AN EXAMINATION OF DOMESTIC WASTEWATER TREATMENT ALTERNATIVES FOR SMALL COMMUNITIES - A CASE STUDY

by

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CHAPTER I

INTRODUCTION

1.1. Introduction

A major factor influencing the health of a small community where public sewers are not available is the proper disposal of domestic wastewaters. Many diseases such as typhoid fever, dysentry, and various types of diarrhea are transmitted from one person to another largely due to improper disposal of human wastes. Hence, every effort should be made to properly dispose of all domestic wastewater so that no opportunity will exist for contamination of water or food.

1.2. The Problem

Apparently the need for disposal of domestic wastewater can be best met by the discharge of these wastes to an adequate community wastewater system. However, this is not always possible, especially in regard to small and isolated communities. In such instances, installation of individual wastewater systems becomes necessary.

1.3. Individual Wastewater Systems and Their Evaluation

The three commonly available individual wastewater systems are

1. Septic tanks
2. Stabilization ponds
3. Package plants, which use the extended aeration modification of the activated sludge treatment process.

The first step in evaluating these three systems is to compute the associated capital, operating and maintenance costs and compare them.

In a traditional analysis, this would be adequate. However, the broader interpretation of cost-benefit analysis warrants the inclusion of the non-quantifiable or intangible factors such as social, environmental and political costs resulting in a complete and thorough analysis. Accordingly, in this study the septic tank, the stabilization pond, and the package plant option would be evaluated both with respect to dollars and cents as well as the intangibles. Also, a sensitivity analysis would be done by varying the sizes of the systems and noting the resultant impact on system cost.

1.4. The Case

The case study to be considered is the "Moske's Addition to Cedar Estates." This subdivision is located west of U. S. Highway 77, about seven miles from Junction City in Geary County. Schwab and Eaton, Inc. of Manhattan have prepared a preliminary plat of the subdivision and this would be the base map for this study. The precise location of the subdivision is given in the plat and places it in the southeast quarter of section 32, township 10, range 5. The location of the subdivision is given in Figure 1 and from this it could be seen that the subdivision is right on the banks of the Milford Reservoir. Figure 2 represents the preliminary plat of the Subdivision.

1.4.1. Climate and Topography

Climate influences the function of wastewater treatment processes like stabilization ponds and to a lesser degree, septic tank systems. Therefore, it is required to take into account the climate of Geary County. Extensive surveys of Geary County have been done by the U. S. Department of Agriculture - Soil Conservation Service and their observations on climate are noted below (1)

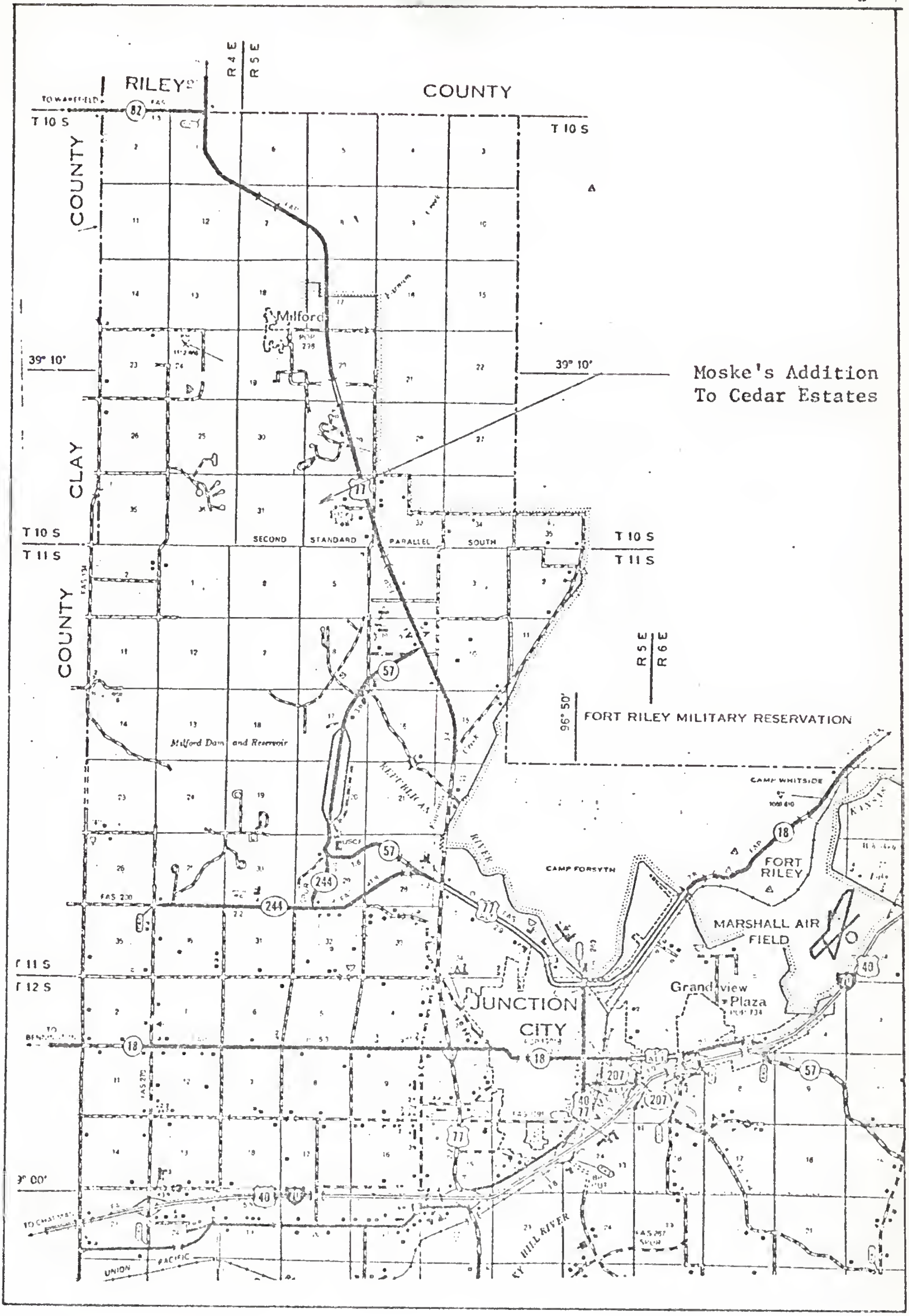

Fig. 1. Location of the subdivision.

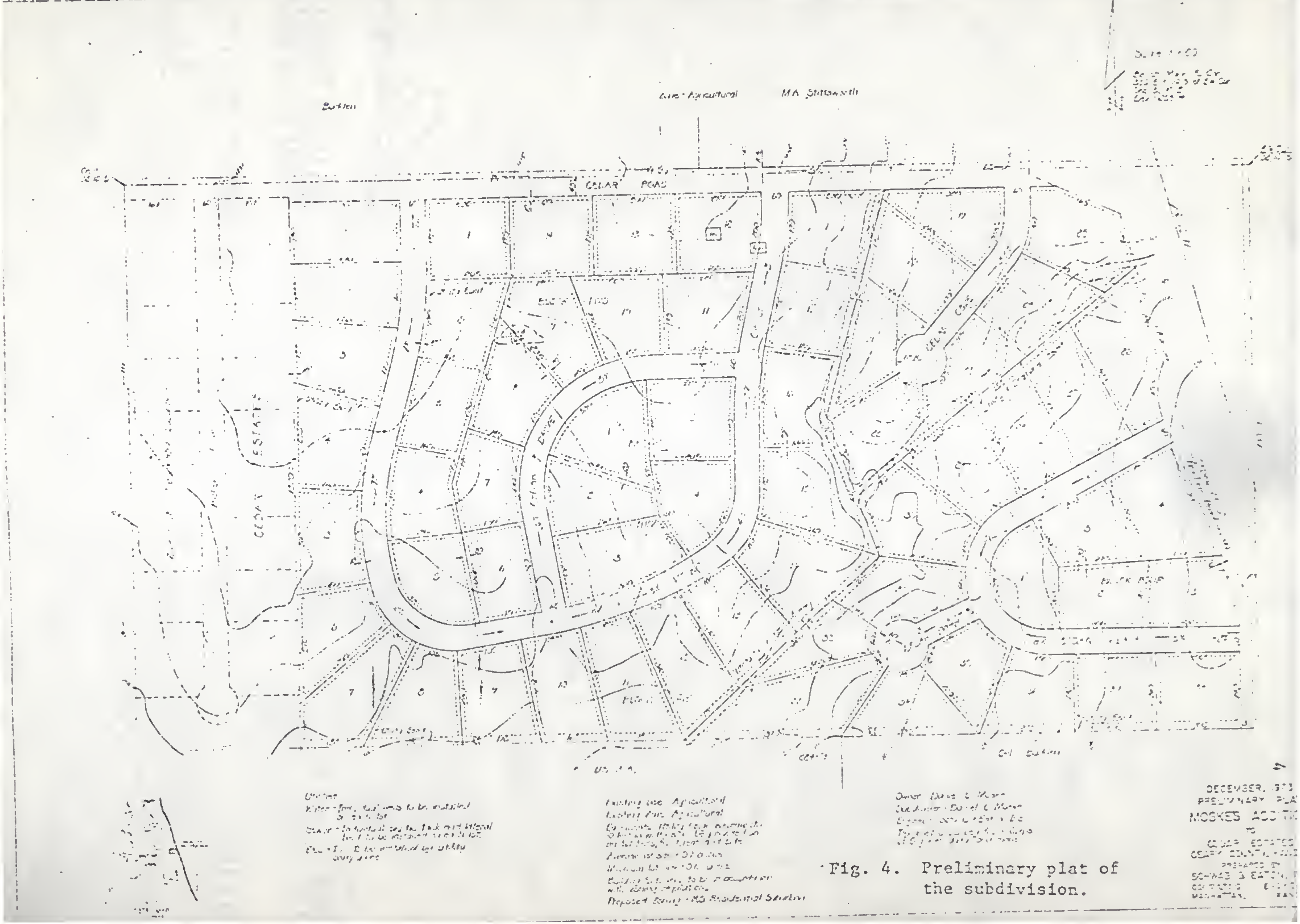

Fig. 4. Preliminary plat of the subdivision.

Geary County has a typical continental climate. The summers are hot, and the winters are moderately cold.

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The precipitation is heaviest in the early part of the summer. Most of it falls during severe thunderstomrs. The winters are dry, clear and open. In summer, the rate of evaporation is high and the relative humidity is low.

The same study has the following data on temperature and precipitation.

TABLE 1

TEMPERATURE DATA OF GEARY COUNTY

SEASON	MEAN TEMPERATURE	REMARKS
Winter	31.3°F	Based on a 96 year
Spring	54.7°F	record, through 1955
Summer	77.8°F	
Fall	57.5°F	
Year	55.3°F	

Source: USDA, SCS, Soil Survey - Geary County, (1959).

The average precipitation based on a 96 year record through 1955 is 31.55 inches per year.

The general topography of the subdivision is characterized by gentle relief and low rolling hills. There are a number of natural gullies and a natural pond. The vegetation is sparse consisting of cedar trees and grass.

The Milford Reservoir offers a magnificent view to most sections of the subdivision.

1.4.2. Soil Characteristics

The location of the subdivision on the banks of the Milford Reservoir significantly affects the soil properties. This is apparent in that the soils occuring here are predominantly sandy and silty loams and clays.

The U. S. Department of Agriculture - Soil Conservation Service (1) has conducted a comprehensive soil survey of Geary County and the following observations relate to this survey. Three major soils are identified and they are:

1. Hastings Silty Clay Loam - 0-8% slope. Soils of this textural class have 27-40% clay and less than 20% sand.
2. Shellabarger Sandy Loam - 4-20% slope. Soils of this textural class have 50% sand and less than 20% clay.
3. Farnum fine sandy loam - 1-4% slope.

Test drilling for water has been done at the subdivision by the Blue Valley Drilling Company of Blue Rapids. Table 2 represents the log of one such drilling. The main point to be noted from this log is that rock layers do not appear until about 27 feet at a minimum. This is a significant criteria in designing septic tank effluent disposal facilities and will be covered in the subsequent section. The log also noted that water was not encountered until about 60'00",which again is a major consideration in the design of a septic tank effluent disposal system.

Finally, the percolation rate of the soil is to be considered. Unfortunately, no percolation tests have been run for the subdivision yet. However, percolation data for the adjacent lots was made available through the owner

TABLE 2

LOG OF SOUTH WELL AT THE SUBDIVISION

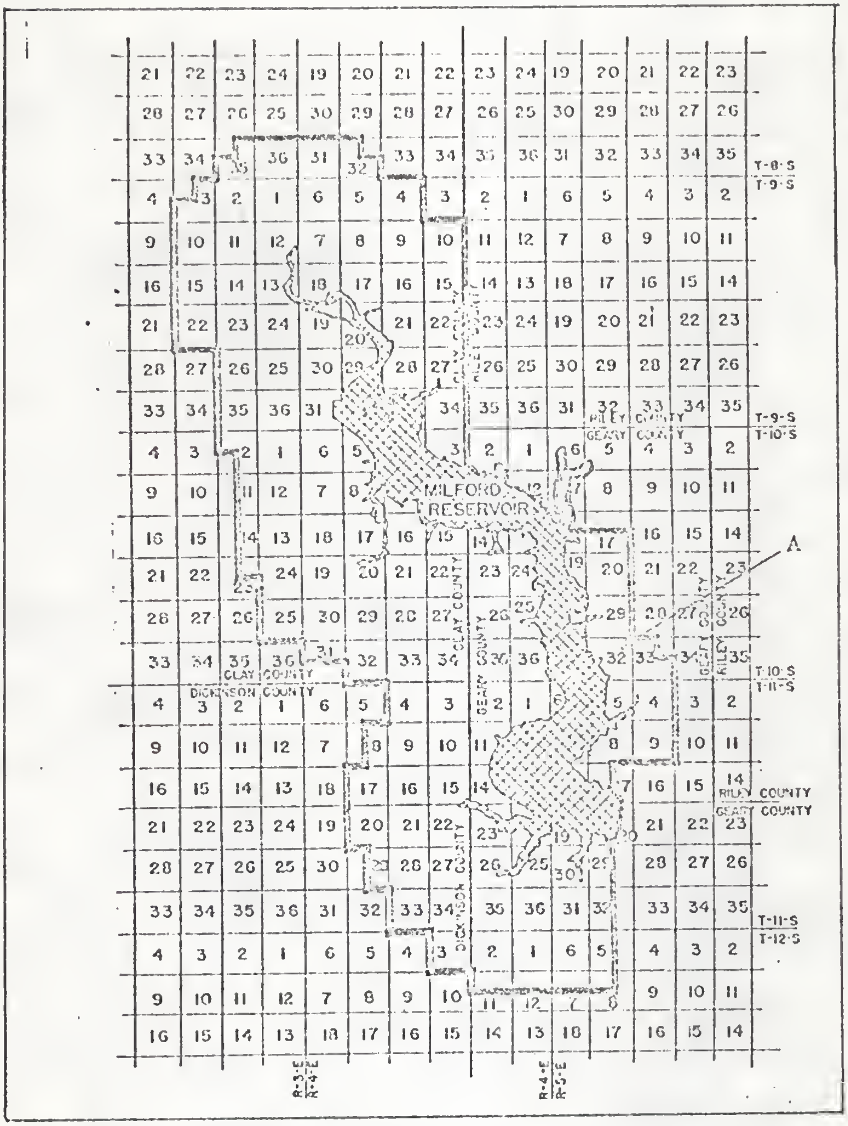
DISTANCE FROM GROUND LEVEL (IN FEET)	TYPE OF MATERIAL ENCOUNTERED
0-4	Brown clay
4-10	Sandy clay
20-27	Brown clay
27-50	Yellow clay and rock layers
50-75	Blue rocks and clay layers
75-90	Layers of white rock and blue shale
90-110	Rock and shale layers
110-130	Layers of rock and some shale

Mr. Daniel Moske and according to this source the percolation rate of the soils in the subdivision is about one inch in 40 minutes which is a poor rate. Yet, in the absence of better data, this rate would be used for design purposes in this study. Nevertheless, it is recognized that percolation rates can vary from lot to lot tremendously and hence actual percolation tests need to be run for each lot before the designs could be finalized.

1.4.3. Zoning Regulations

The subdivision presently comes under the Agricultural Zone; however the developer has proposed a change from Agricultural Zone to Residential Suburban Zone.

The most crucial zoning aspect, though is the fact that the subdivision falls within the boundaries of the Milford Reservoir Sanitation Zone as defined by the Kansas State Board of Health; Figure 3 represents the Sanitation Zone



A Moske's addition to cedar estates.
Source: Kansas State Department of Health, Sanitation Zone Regulations, October, 1971.

Fig. 3. Milford Reservoir Sanitation Zone.

for the Milford Reservoir. The purpose of the Sanitation Zone Act has been defined in the Statutes of Kansas (2); K. S. A. 65-184 is quoted below to clarify the purpose of the Act.

> The purpose of this act is to regulate and control development of areas of the state surrounding certain impoundments of water to prevent pollution of such impoundments, to assure sound and economical development and maintenance of healthful and sanitary conditions so that the state will realize maximum benefits therefrom, and the health, safety and well-being of the people of the state will be protected.

It is now necessary to understand the exact meaning of the Sanitation Zone; this definition is provided in K. S. A. 65-185, Sec. (a).

> The term "sanitation zone" means the land within an area designated and described by regulation of the state board of health under the provisions of this act, <u>no portion of which is located more than three (3) miles from the water line of the conservation pool of any existing or proposed state or federal reservoir having a surface area of its conservation pool of more than one hundred (100) acres</u> (emphasis author's) - - - - - - - - - - - - - - - - - -
> -

Lot sizes are defined in K. S. A. 65-185, sec (j)

> The term "lot" means (1) <u>any premise of less than three (3) acres used or intended for a single family dwelling</u> (emphasis author's) - - - - - - - - - - - - - - - - -
> -
> All such premises shall be placed prior to the construction of buildings or facilities thereon.

The relevancy of the above definitions to the subdivision could be now pointed out. First, the subdivision falls within three miles from the water line of the Milford reservoir and hence in the Sanitation Zone according to the first definition. The minimum lot size in the subdivision is 0.75 acres and the average lot size is 0.9 acres on which single family dwellings are proposed. So, according to the second definition platting becomes mandatory before any construction could begin.

Implementation of the regulations of the Sanitation Zone Act is the responsibility of the "reservoir sanitation officer". This person may be the county engineer or any other person designated by the County Commisioners. The state board of health gives the final approval for such appointments. State control and involvement is further evidenced by the following citation from K. S. A. 65-189C.

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> If the sanitation plan contains plans for a sewage system or a water supply to serve two or more lots or services, the reservoir sanitation officer shall submit the preliminary engineering study and sanitation plan to the department for review and approval prior to his approval. The board of County Commissioner's of the county in which the plat is located shall not approve the plat until approval of the sanitation plan related thereto is received from the reservoir sanitation officer.

Thus, it is apparent that the major burden of controlling and regulating the development within Sanitation Zones is borne by the State. This is quite rational as the State Board of Health has a greater fund of knowledge and expertise to draw on in complex decision making situations. However, expediency of decisions could be assured by a more expanded responsibility to the local government units. Nevertheless, any reform in decision making is to be expected to originate with the Board of Health.

1.5. Design Population and Flow

The design population is based upon the proposed residences in the subdivision. Hence it is necessary to first compute the number of lots proposed in the Preliminary Plat.

By reference to the Preliminary Plat (Fig. 2),

No. of lots in Block 1 = 38

No. of lots in Block 2 = 14

No. of lots in Block 3 = 5

No. of lots in Block 4 = 5

Total lots = 62

Assuming that a typical three bedroom residence is built on each lot, and thus taking three persons per lot,

Total number of people that could be accomodated in the subdivision = 62 x 3 = 186

The per capita, average, daily domestic wastewater flow for design purposes is recommended as 100 gallons by the Kansas Health Department. Peak flows will be higher than this and a typical ratio provided by a standard source (3) is 2.25:1 for residential wastewater flows. Average daily flow is used to design treatment units and peak flow is used to design sewers, pumps etc.

Using a population figure of 186 and an average per capita flow of 100 gallons per day,

Average flow = 186 x100

= 18,600 gallons or say 20,000 gallons

Therefore peak flow, using a ratio 2.25:1,

= 20,000 x 2.25

= 45,000 gallons per day.

CHAPTER II

SEPTIC TANK SYSTEM

2.1. Process Description

The septic tank is simple to construct, requires a minimal amount of care and attention to operate and is one of the most economical wastewater treatment systems for individual household units. However, the treatment is not fully completed in this process. As a result, the tank effluent may contain pathogenic bacteria and it is septic and putrescible.

Figure 4 represents a simple, yet typical septic tank design. Wastewater enters the tank through the inlet pipe and is retained in the tank as quiescently as possible for a period of about 24 hours. During the retention period sedimentation removes 60 to 70% of the suspended solids (4). Most of these solids accumulate at the bottom of the tank as sludge; but some of the solids float to the top to form scum. Anaerobic bacteria digest both the sludge and the scum to form gases and liquids. Thus a volume reduction of the sludge takes place. Also, pathogenic organisms are largely or completely destroyed in the sludge.

The septic tank effluent is generally treated and disposed of in a Tile Disposal Field which is also termed as the soil-absorption field or simply a Tile Field. This system makes use of open jointed clay or concrete pipes to apply the effluent to the soil. Filtration will remove suspended matter, and anaerobic bacteria stabilize the organic matter in the effluent. The soil should be permeable and naturally well drained if this system is to operate efficiently. Heavy clays and limestone formations are unsuitable

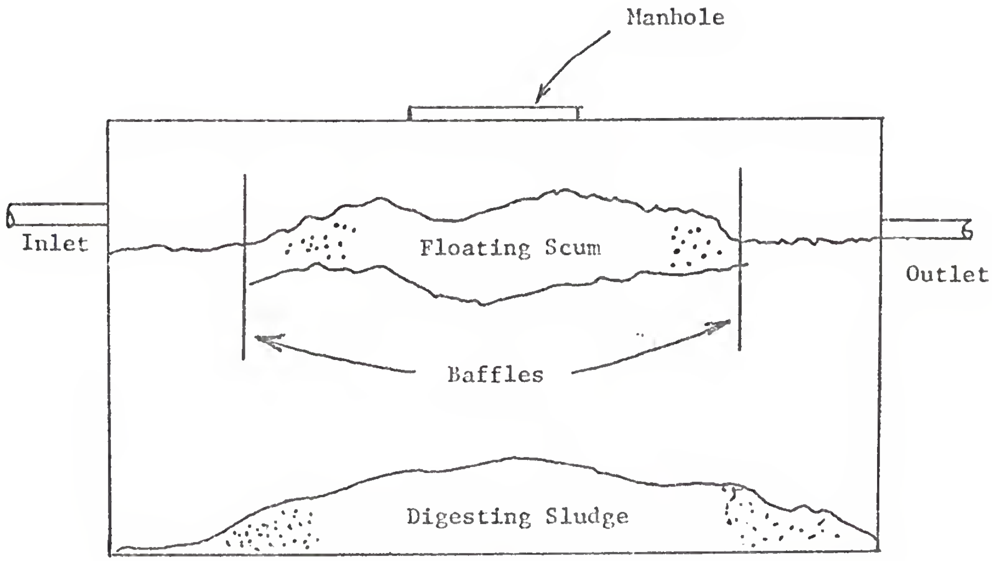


Fig. 4. Typical Septic Tank Design

for tile fields as the former have low permeability and the latter may allow sewage to enter through the fissures and pollute groundwater supplies.

2.2. Advantages and Disadvantages

Some of the advantages and disadvantages associated with septic tanks are briefly listed. These have to be taken into consideration when making an overall system assessment in addition to economics.

Advantages

1. The septic tank is economical to build and operate.
2. Maintenance required is minimal.
3. The septic tank system is well suited for phased construction.
4. The septic tank has given satisfactory service, when built and operated properly, leading to its general acceptance by health officials and the public.
5. There are no power requirements.

Disadvantages

1. The ground water pollution potential of the septic tank is high especially when poorly built and operated.
2. Maintenance required is minimal, yet in the absence of such maintenance regularly the system can fail quickly and become a health hazard.
3. Switch over to a central sewer system is not easy.
4. The septic tank system demands good soil conditions at site and a deep water table.
5. The individual lots on which septic tanks are built require a large area for an efficient system.

2.3. Design of Tank

The regulations for septic tank location, construction and operation in Kansas are set out in a manual of recommended standards (5). This manual is based on a Public Health Service Study (6) and so both the manuals will be used for design purposes.

2.3.1. Tank Capacity

The capacity of a septic tank is recommended by the State. Table 3 gives the recommended minimum capacities.

The liquid capacities recommended allow for the use of all modern household appliances, including garbage grinders. Also, there is a recognition in these standards that liberal tank capacity is important from a functional standpoint. As observed in Section 1.5, three bedroom houses are proposed.

Referring to Table 3, tank capacity for a three bedroom house = 900 gallons or 120.32 cu. ft.

The shape of the tank is unimportant for a given capacity and depth (5). However, it is recommended that the smallest plan dimension of the tank be at least 2'-00". Liquid depth may range between 2.5 ft. to 5.0 ft. Select a liquid depth of 3 ft.

Hence, area needed $= \frac{120.32}{3}$

$= 40.32 \text{ ft.}^2$

Provide a tank of clear dimensions 8 ft. x 5 ft. x 3 ft.

Actual capacity provided = 8 x 5 x 3 = 120 cu. ft. or 898 gallons.

Since this is close to 900 gallons, the capacity provided is adequate.

The design details are given in Figure 5.

TABLE 3

LIQUID CAPACITY OF TANK (GALLONS)

NUMBER OF BEDROOMS	RECOMMENDED MINIMUM TANK CAPACITY	EQUIVALENT CAPACITY FOR BEDROOM
2 or less	750	375
3	900	300
4*	1,000	250

*For each additional bedroom, add 250 gallons.

Kansas Department of Health, "A Manual of Recommended Standards for Locating, Constructing and Operating Septic Tank Systems for Rural Houses" Bulletin No. 4-2 Topeka, June 1973.

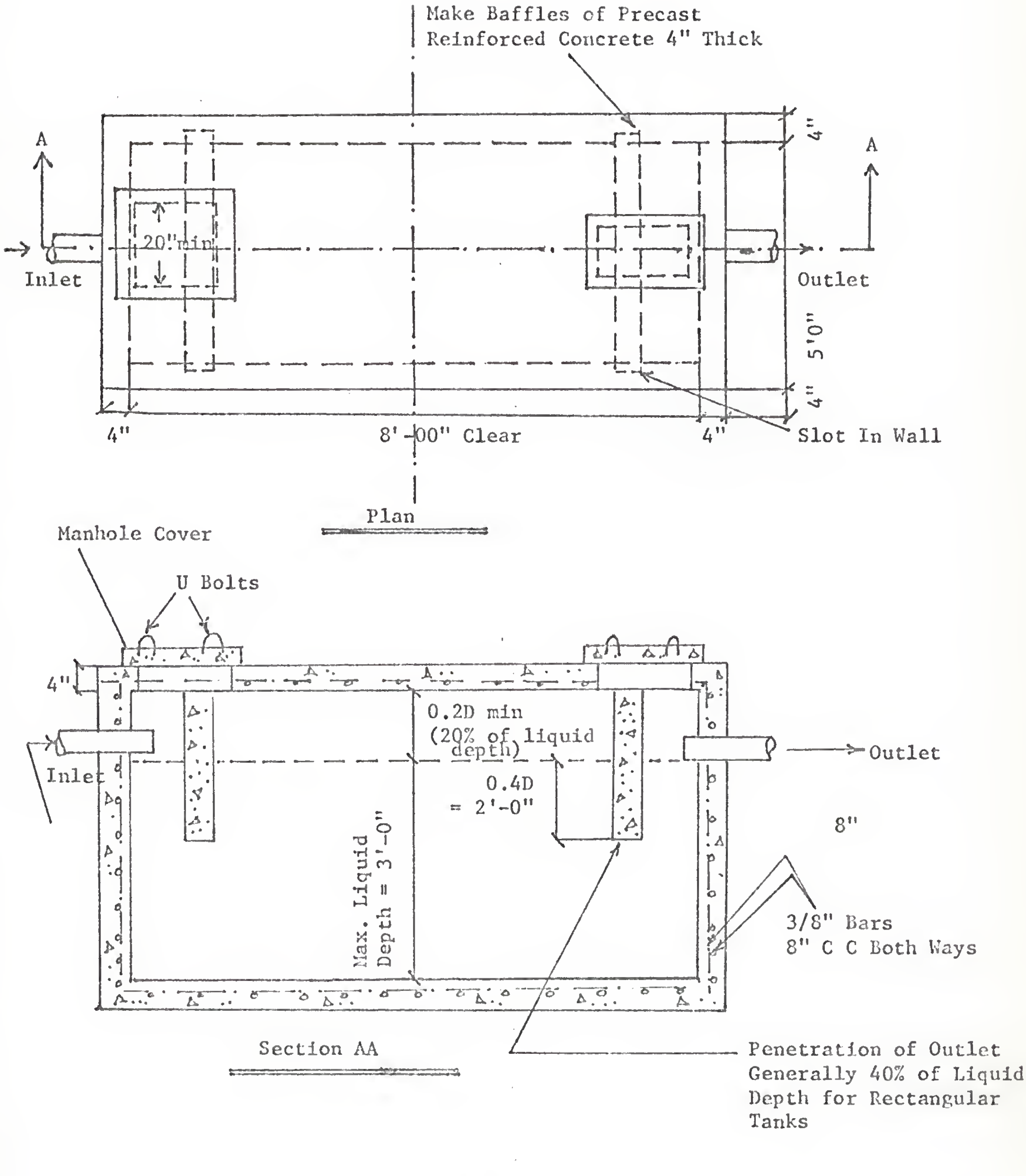


Fig. 5. Septic Tank Design Details

2.4. Design of Tile Field

Soil properties play the critical role in the design of a tile field. The two most important conditions to be met are:

1. The percolation should be within acceptable ranges as shown in Table 4.
2. The maximum elevation of the groundwater table should be at least four feet below the bottom of the absorption trench. Rock formations or other impervious strata should be at a depth greater than four feet below the bottom of the trench.

In order to know whether the subdivision meets the recommended percolation and groundwater conditions, it is necessary to recall the information mentioned in Section 1.4.2. Accordingly, it is seen that the percolation rate of one inch in 40 minutes falls within an acceptable range as shown in Table 4. However, it is recognized that this percolation rate is a poor rate. Secondly, the groundwater as indicated by test wells is at a considerable depth, namely 60 feet. Finally, rock layers do not occur as per the test drilling log until about 27 feet, thus the second recommended standard for groundwater table and underlying rock formation is adequately satisfied. Under these conditions, the tile field can now be sized.

2.4.1. Sizing the Tile Field

Referring to Table 4, it is seen that for a percolation rate of one inch in 45 minutes the required trench bottom area per bedroom is 300 sq. ft. Since the percolation rate for the subdivision is one inch in 40 minutes, 300 sq. ft. per bedroom is slightly on the higher yet safer side. Hence provide 300 sq. ft. per bedroom.

TABLE 4

ABSORPTION AREA REQUIREMENTS FOR PRIVATE RESIDENCES

(Provides for garbage grinder and automatic sequence washing machines)

PERCOLATION RATE (Time required for water to fall 1 inch in minutes)	REQUIRED ABSORPTION AREA, IN SQ. FEET PER BEDROOM[1] STANDARD TRENCH[2]
1 or less	70
2	85
3	100
4	115
5	125
10	165
15	190
30	250
45	300
60[3]	330

[1]In every case, sufficient area should be provided for at least three bedrooms.

[2]Absorption area for standard trenches is figured as trench bottom area.

[3]Unsuitable for absorption systems if over 60.

Kansas Department of Health, "A Manual of Recommended Standards for Locating, Constructing and Operating Septic Tank Systems for Rural Houses" Bulletin No. 4-2 Topeka, June 1973.

Total trench area needed per residence = 300 x 3

= 900 sq. ft.

Current design practice for tile fields recommends trench widths ranging from 18 in. to 24 in. Select a trench width of 24 inches.

Therefore,

$$\text{Total length of trench needed per residence} = \frac{900}{2}$$

$$= 450 \text{ ft.}$$

The recommended individual lateral length is a maximum of 100 ft. But, it is preferred to have shorter laterals up to a length of 60 ft. only. So, provide 50 ft. long individual lateral. Number of individual laterals needed

$$= \frac{450}{50}$$

$$= 9$$

The recommended minimum depth of the trench is 18 inches. Provide a depth of 20 inches. The distance between trenches depends on trench widths and the recommended values are shown in Table 5. Using Table 5, it can be seen that for a trench width of 2 ft., the minimum distance between laterals is 6.5 ft., provide this minimum spacing. A serial distribution arrangement is used as this gives the best possible loading.

Minimum total area for the tile field could now be calculated

Minimum area needed = 9 x 6.5 x 50 = 2925 sq. ft. or say 3000 sq. ft.

An emergency tile field area has to be set aside so that if the original one fails there is space for a second. Provide an area of 3000 sq. ft. for this.

Hence, total tile field area needed = 3000 x 2 = 6000 sq. ft.

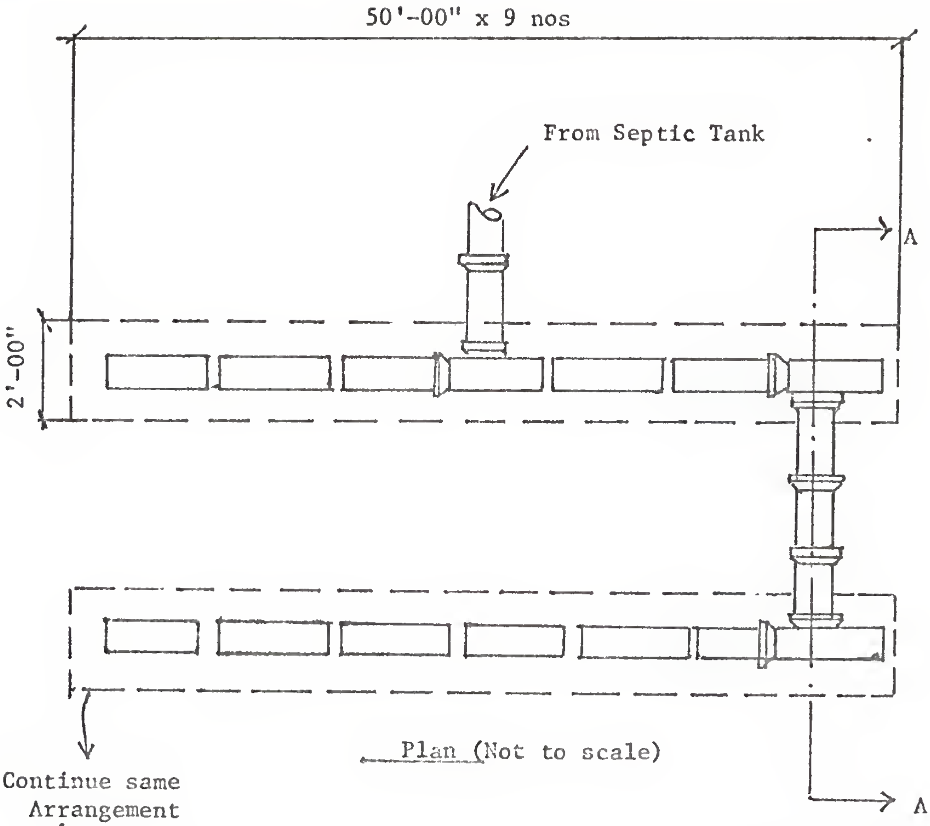
Agricultural drains of 4 in. diameter with open joints are to be used to load the tile field with wastewater. Design details of the tile field are shown in Figure 6.

TABLE 5

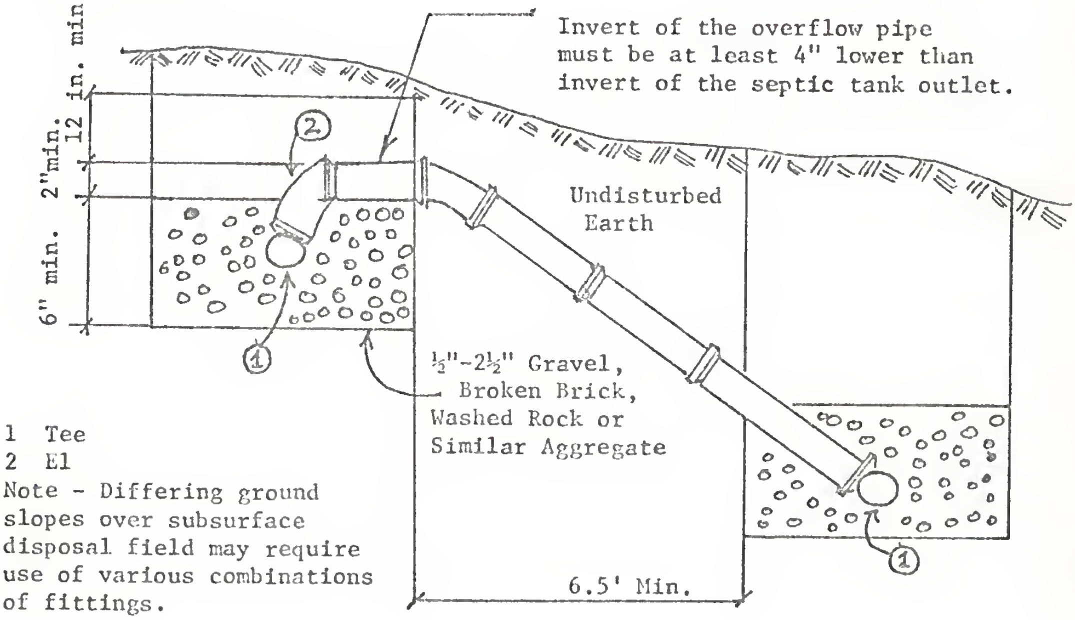
DISTANCE BETWEEN TRENCHES

TRENCH WIDTH (INCHES)	MINIMUM DISTANCE BETWEEN CENTERLINE OF TRENCHES-(FEET)
12 to 18	6
18 to 24	6.5
24 to 30	7.0
30 to 36	7.5

Kansas Department of Health, "A Manual of Recommended Standards for Locating, Constructing and Operating Septic Tank Systems for Rural Houses" Bulletin No. 4-2 Topeka, June 1973.



Plan (Not to scale)



Section AA (Not to scale)

Fig. 6. Tile Field Design Details

2.5. Sludge Cleaning Interval

Regular cleaning of septic tanks is a must if the system is to operate efficiently. Otherwise, the sludge and scum may escape the tank with the effluent which results in a clogging of the tile field. When this happens wastewater may either back up in the plumbing system or it may break through the ground surface creating a health hazard. The solution to the tile field clogging involves not only cleaning the septic tank but also the possible construction of a new tile field.

The rate at which sludge and scum accumulate vary widely. For example, according to one manual (5), in one case out of 20, the tank will reach the point where it should be cleaned in less than three years. Other tanks of similar capacity may be used for much longer periods before it is necessary to clean them. However, the best way to know whether cleaning is necessary is to inspect the sludge accumulation in the tank; according to the same manual the allowable sludge accumulation for a 900 gallon capacity tank, with a liquid depth of three feet is four inches from the bottom of outlet device to top of sludge. We can also compute the cleaning interval as below:

Wastewater flow per residence per day = 3 x 100

= 300 gallons

Hence, yearly wastewater flow per residence = 300 x 365

= 109,500 gallons or say

110,000 gallons

Assume a Total Suspended Solids (T. S. S.) Concentration of 300 mg/l

75% of the T. S. S. is Volatile Suspended Solids (V. S. S.) = 225 mg/l.

Primary sedimentation in septic tank is assumed to have 60% removal efficiency (3).

Hence T. S. S. removed $= 0.60 \times 300$

$= 180$ mg/1.

V. S. S. removed $= 0.60 \times 225$

$= 135$ mg/1.

Anaerobic digestion destroys some of the V. S. S. in the septic tank. Assume a 30% reduction in V. S. S; this is a reasonable assumption as the efficiency of V. S. S. removal can at best of temperature conditions approach only 40 to 50%

Hence, V. S. S. destroyed $= 135 \times 0.3$

$= 40.5$ or

40 mg/1.

Hence, T. S. S. trapped and remaining $= 180 - 40$

$= 140$ mg/1.

The amount of sludge can now be calculated for various conditions of V. S. S. destruction.

1. 30% V. S. S. destroyed (60% T. S. S. trapped)

   Amount of sludge per year = 140 (0.110 MG)(8.34)

   = 128 lb. per year.

   Assuming a typical 4% solids concentration,

   $$\text{Volume of sludge} = \frac{128}{62.4 \times 0.04} = 51 \text{ cu. ft. per year.}$$

   The capacity of a 900 gallon septic tank = 120 cu. ft.

   $$\text{Hence retention time} = \frac{120}{51} = \underline{2.35} \text{ years.}$$

2. No V. S. S. destroyed (60% T. S. S. trapped)

   Amount of sludge per year = 180 (0.11 MG)(8.34)

   = 165 lbs.

Assuming 4% solids, vol. of sludge $= \frac{165}{62.4 \times 0.04} = 66$ cu. ft.

Hence retention time $= \frac{120}{66} = \underline{1.82}$ years.

3. 100% T. S. S. trapped.

Amount of sludge per year = 300 (0.11 MG)(8.34)

= 275 lbs.

Assuming 4% solids again, vol. of sludge $= \frac{275}{62.4 \times 0.04}$

= 110 cu. ft.

Hence, retention time $= \frac{120}{110}$

= 1.09 yrs.

Thus, the sludge accumulation interval before cleaning can vary from a low of 1.09 years under the worst treatment conditions to a high of 2.35 years under the most favorable environment. Two years is a safe sludge cleaning interval.

2.6. Cost Estimates

The cost details used here were obtained by contacting the Manhattan Septic Tank Company.

2.6.1. Cost of Tank

Cost of a precast, 900 gal. capacity tank =	$70.00
Excavation, transportation, and installation =	180.00
Total =	$250.00

26.2. Cost of Tile Field

Total land cost for 6000 sq. ft. needed (Section 2.4.1) @ $1000 per acre = 138.00

Material costs for 4 in. pipe @ \$0.30/L.F.	= 450 x 0.30
for 450 ft	= \$135.00
Trenching, installation and backfilling	= 450 x 1.30
@\$1.30/L.F. for 450 ft.	= \$585.00
Total for tile field	= \$858.00
Grand total for tank and tile field	= 250 + 858
	= \$1108.00

Maintenance involved lies in cleaning the tank every two years to remove the sludge. In Manhattan area, the cleaning expenses run to about \$50.00 per cleaning. Since cleaning interval is shown to be two years,

$$\text{Annual cleaning cost} = \frac{50}{2} = \$25.00.$$

CHAPTER III

STABILIZATION POND SYSTEM

3.1. General

A stabilization pond is a large shallow pond in which wastes are added at a single point and the effluent removed at one single point. The ponds are generally two to four feet deep, the reason for the shallowness being weed control. But deeper ponds, 10 to 20 feet, have been used quite successfully. Another factor in favor of shallowness, is that, the shallower the pond for a given waste, the greater will be the surface area. Mixing by wind currents is more effective generally in the case of shallower ponds.

3.2. Process Description

A schematic diagram given in Figure 7 represents the process of a stabilization pond (7). The treatment occuring in the pond is biological and aerobic bacteria, algae to a lesser extent and anaerobic bacteria help achieve this. The algae use CO_2 as a carbon source and together with the nutrients NH_3 and PO_4 in the pond synthesize cell mass. Light energy is needed as this process is photosynthetic. Algae release O_2 in this process and aerobic bacteria supplement it with O_2 from reaeration in order to convert organics in the wastewater to more CO_2, and H_2O. In winter time, when there is ice cover, the pond will turn anaerobic and then anaerobic bacteria act on the organics to yield CH_4, NH_3 and CO_2. The BOD (Bio-Chemical Oxygen Demand) reduction depends primarily on climatic conditions and warm, sunny weather accelerates decomposition and photosynthesis resulting in greater removals. B. O. D. reductions in the summer usually exceed 95 percent (3). A case study in Kansas (8) gives an average B. O. D. removal of 85%.

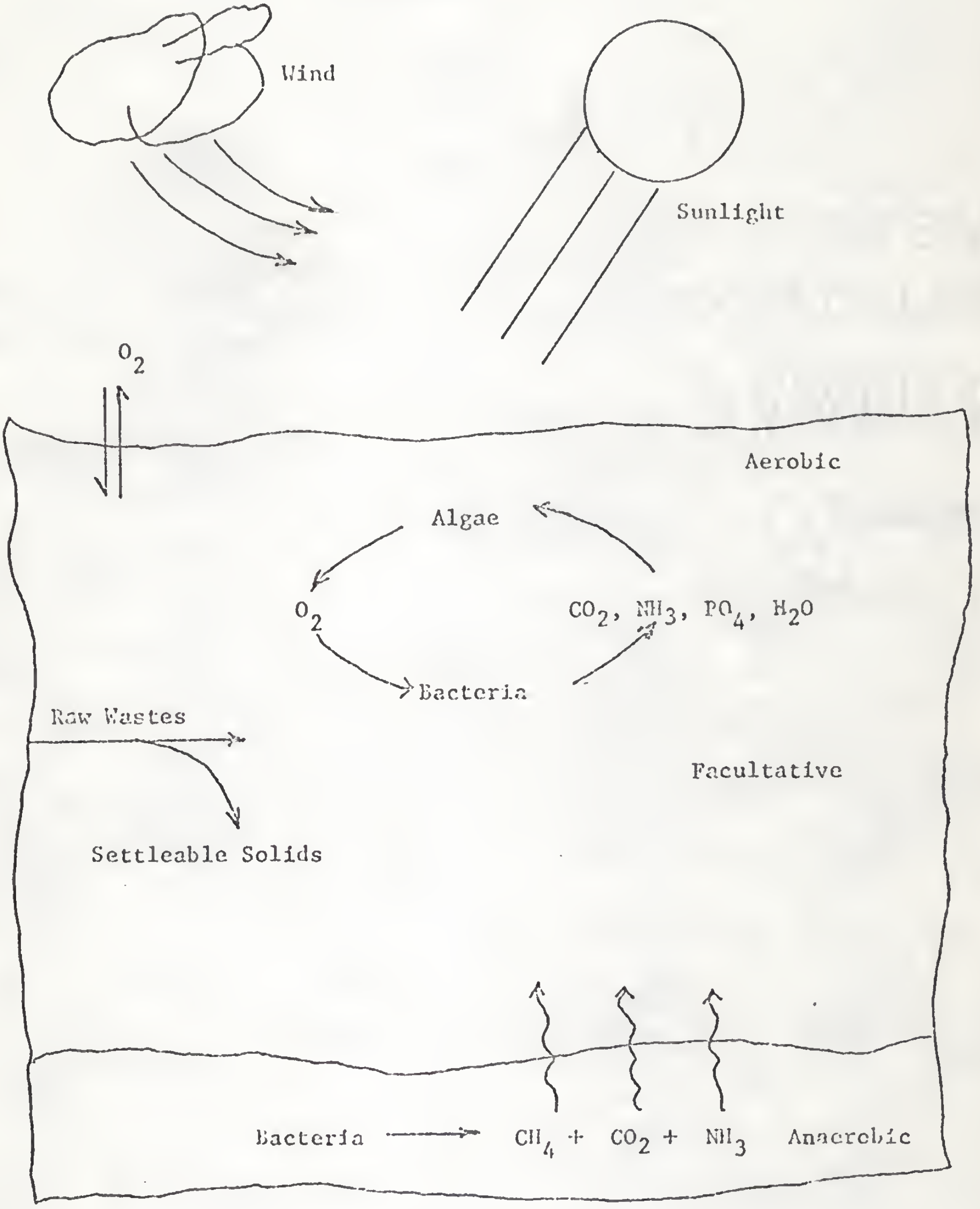


Fig. 7. Process of Stabilization Pond.

3.3. Advantages and Disadvantages

The advantages and disadvantages of stabilization ponds are briefly listed below. It is necessary to use these factors in addition to economics in decision making.

Advantages

1. The construction cost is low.
2. Minimum operation and maintenance cost.
3. Ideal for small outlying subdivisions.
4. Gives reliable treatment.
5. There are no power inputs.

Disadvantages

1. Effluent disposal is a problem, especially algae removal.
2. Poor maintenance can create odor problems.
3. Land area needed is rather large.
4. Stabilization pond along with the sewer system is not quite suitable for phased construction.

3.4. Design of Stabilization Pond

3.4.1. Calculation of B. O. D. and S. S. Contributions

Assuming a 100 gallon per capita average flow, the following values for B. O. D. and S. S. are fairly typical for domestic wastewaters (3).

B. O. D. = 200 mg/l or 0.17 lbs./capita/day.

S. S. = 240 mg/l or 0.20 lbs./capita/day.

Installation of garbage grinders in homes, which is assumed, would increase B. O. D. by about 30% and S. S. by about 60% (3).

Revised B. O. D. = 0.17 x 1.3 or

0.22 lbs./capita/day

andrevised S. S. = 240 x 1.6

= 384 mg/1

say about 300 mg/1

= 0.25 lbs./capita/day

For designing the stabilization pond, then, a B. O. D. contribution of 0.22 lbs./capita/day and a S. S. contribution of 300 mg/1 will be used.

3.4.2. Sizing the Pond

The State of Kansas has the following design guidelines (8).

1. B. O. D. bonding - 35 lbs./acre/day maximum.

2. Detention time - A minimum of 90 days for raw wastewater.

Using these standards, the stabilization pond can now be sized.

Total B. O. D. contribution = 186 x 0.22

= 40.92 lbs./day

Hence area needed @ 35 lbs./acre/day $= \frac{40.92}{35}$

= 1.169 or

1.20 acres

= 52,272 sq. ft.

To check detnetion time, assume a depth range of 3 ft. to 5 ft. or average 4 ft.

Hence, liquid volume of pond = area x depth

= 52,272 x 4

= 209,088 cu. ft.

Discharge into the pond is 20,000 gallons/day

Hence detention time $= \frac{\text{volume}}{\text{discharge}}$

$= \frac{209,088}{20,000/7.5}$

= 78.4 days.

Since the minimum detention time is 90 days, the volume has to be increased by either varying area or depth. The maximum depth of five feet can not be exceeded, therefore increasing the area is called for.

The area is increased to 1.4 acres.

New area provided = 1.4 acres or 60,984 sq. ft.

Detention time after increase in area = $\frac{60,984 \times 4}{20,000/7.5}$

= 91.5 days.

Hence, O.K.

Providing a pond of 330 ft. x 185 ft. would yield a surface area of 61,050 sq. ft. > 60,984 required. Hence, O.K. The average depth of pond is four feet (4'-00"), with provision for a five feet maximum.

3.4.3. Time to Fill Up The Pond

S. S. contribution/capita/day = 0.25 lbs./c/day

Total solids for the subdivision = 186 x 0.25

= 46.5 lbs./day

= 16,972 lbs./yr.

1 acre at 1 ft. depth = 43,560 cu. ft.

Assuming a sludge density of 65 lbs./cu.ft.

Volume of sludge = $\frac{16,972}{65}$ = 261 ft.^{3}/yr.

Therefore, time needed to fill pond to a depth of 1 ft. = $\frac{43,560}{261}$

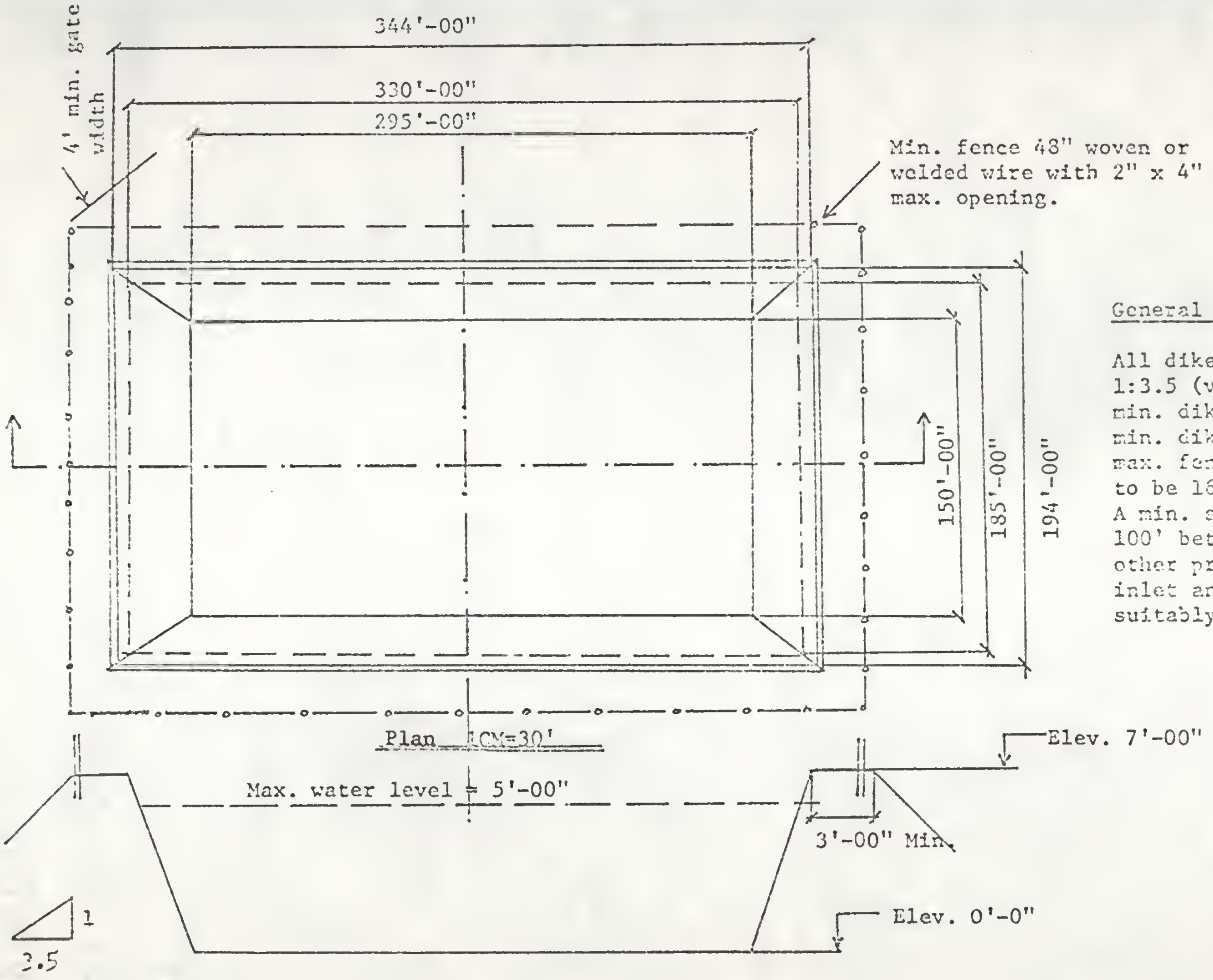
= 167 years.

Design details of the stabilization pond are given in Figure 8.

3.5. Cost Estimates

Surface area needed for the tank = 1.4 acres.

Additional land will be needed for the embankment and also the buffer area which is controlled by the minimum separation distance of 100 feet in Kansas. Buffer area is computed as shown next.



General Notes

All dike slopes to be 1:3.5 (ver. to horiz.)
min. dike berm width 3'
min. dike free board 2'
max. fence post spacing to be 16.5'
A min. separation of 100' between pond and other property provide inlet and outlet suitably.

Section AA (Not to scale)

Fig. 8. Design Details of Stabilization on Pond.

The clear dimensions of pond as shown in Section 3.4.2 = 330 ft. x 185 ft.

Providing 100 ft. space in all directions,

$$\text{additional area needed} \begin{array}{l} (330 + 200) \times 100 \times 2 \\ + 185 \times 100 \times 2 \end{array}$$

$$= 106{,}000 + 37{,}000$$

$$= 143{,}000 \text{ sq. ft. or } 3.28 \text{ acres.}$$

Hence total land needed = 1.4 + 3.28

= 4.7 acres, app.

Therefore at \$1000.00 an acre, land cost 4.7 x 1000 = \$4700.00

Excavation needed = 1.2 x 43,560 x 5 = 261,360 cu. ft. = <u>9680 cu. yds</u>.

The excavation and other costs used below were obtained from Manhattan Septic Tanks.

Cost of excavation including embankment	= 9680 x 0.65
@ \$0.65/cu. yd.	= <u>\$6292.00</u>
Cost of Bentonite clay for sealing of	= 55 x 10
pond @ \$55/ton, 10 tons	= <u>\$550.00</u>
Cost of fencing including installation	= 1092 x 4.00
etc. complete @ \$4.00/ft.	= <u>\$4368.00</u>
Cost of inlet and outlet structures	= 800 + 1200
Lump sum	= <u>\$2000.00</u>
Total capital cost for pond	= <u>\$17,910.00</u>

Maintenance of a stabilization pond involves periodic mowing of grass on embankments so as to keep the water free of weeds, for insect control, and general operation. It is very hard to come up with exact costs, but some approximations need to be made to get a general idea.

Assume that four hours a week are needed to maintain the pond. Neglecting a winter lapse of three months, maintenance is then needed for 9 months every year.

Therefore, total manhours needed for maintenance = 9 x 4 x 4

= 144 hours

Assuming an hourly wage of \$5.00/hour,

Total wages needed = 144 x 5

= \$720.00/yr.

CHAPTER IV

PACKAGE PLANT SYSTEM

4.1. General

The general term "package plant" is applied to plants which are pre-engineered and make use of standardized equipment. Package plants may be partly fabricated in the factory and then assembled and erected at site or they can be completely assembled in the factory in which case they just need to be transported to site to be erected readily. The materials used for package plants vary - concrete, steel and of late fibre glass have been used. Aerobic or anaerobic process could be utilized to treat wastewaters, but for the purpose of this study an aerobic system will be considered.

4.2. Process Description

The extended aeration process is a modification of the activated sludge process and gives B. O. D. efficiencies in the range of 85 to 95%. Figure 9 represents the process. As shown in Figure 9, wastewater enters the aeration tank after some sort of screening and is aerated for a period of 24 hours. Vigorous mixing in the aeration tank is achieved by supplying compressed air through diffusers. Aerobic bacteria utilize the oxygen supplied and break down the organics to more stable compounds and the mixed liquid flows into a settling tank where it is detained for about four hours. The settled sludge is then returned to the aeration tank again to be further broken down. The repeated recirculation of sludge, and the extensive aeration period ensure the breaking down of decomposable solids into stable inert matter. Very little sludge is produced in the process because of

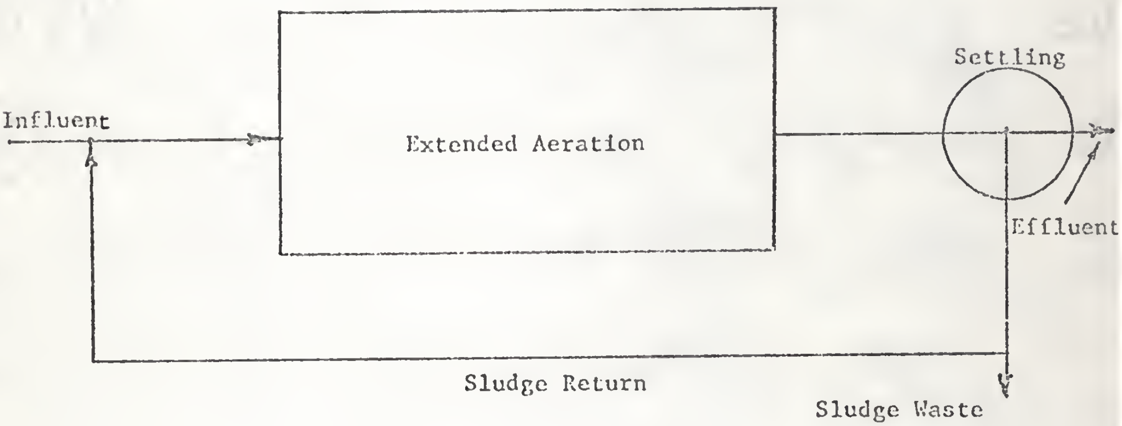


Fig. 9. Extended Aeration Process.

recirculation and this helps to eliminate sludge handling equipment, digesters etc.

4.3. Advantages and Disadvantages

Advantages

1. Overcomes soil limitations and makes possible development on lands where septic tanks can not function efficiently.
2. The effluent produced is clear, odorless and easily disposable either by stream or surface discharge.
3. Extended aeration can handle shock loadings without becoming upset.
4. Very reliable treatment is possible.
5. Ideally suitable for small developments.

Disadvantages

1. The initial capital costs are high.
2. Skilled and regular maintenance is essential.
3. Maintenance costs can be high as there are power inputs for aeration.
4. Public officials have yet to accept the system on a wide basis.

4.4. Design of Package Plant

4.4.1. General

Basically, there is very little design work involved in the case of a package plant as far as this study is concerned. The only design parameters to be known are the design flows and the required degree of treatment. Once, these are known, it is just a question of looking up the various manufacturer's catalogs and selecting the best plant to meet the treatment needs in an economical way.

4.4.2. Manufacturers Contacted

Since the data on package plants is not readily available in the standard literature, letters were sent to manufacturers requesting information. A list of manufacturers contacted and a sample letter are shown in Appendix 1 and 2-B respectively. Unfortunately, the responses were poor; yet one or two companies gave good information. A national study on package plants also was consulted in order to get a clearer idea (9).

4.4.3. Selection and Capital Costing

The first package plant to be considered on the basis of data supplied by the manufacturer is the NPS Batch-Treatment System (10) supplied by the Cromaglass Corporation. The system uses the extended aeration principle to treat wastewaters and gives more then 85% solids and B. O. D. removal. The subdivision needs a 20,000 gpd plant. Referring to the catalog, it is found that model G.C.6 has a capacity of 21,000 gpd. Since this is the model coming closest to the required capacity of 20,000 gpd, select this. The installed price of the plant varies from a minimum of \$25,000 to a maximum of \$28,000 because of varying job site conditions, construction practices etc. So, the average price is around $\frac{25,000 + 28,000}{2}$ or \$26,500 which can be rounded off to around \$30,000 on the average.

The second company to be contacted was Smith and Loveless which sells more than 95% of the package plants sold in Kansas. Since cost data were not given in the regular catalog (11), it became necessary to call the company and get the cost data.

Smith and Loveless markets a package plant under the trade name of "Model V Treatment System." The cost of the system is broken down into an aeration cost and clarification cost as below.

Aerator of 20,000 gpd capacity (Model 12 CA 20)	=	$10,500.00
Clarifier for above (Model 10 C 191)	=	10,500.00
		$21,000.00

Additional costs would be necessary for transporting, erecting, and landscaping needs. Add a lump sum of $9,000.00 to the equipment cost. Hence, the erected cost of the equipment for 20,000 gpd capacity = $30,000.

The national study (9) has a listing of different sized package plants and their list prices. For a 20,000 gpd plant the mean list price is about $14,000 and the maximum list price is about $18,000. Under the worst conditions, the highest list price is therefore $18,000.00. Adding $9,000.00 again, for installation charges, total cost would be $27,000.00 or approximately $30,000.00 on the average.

Apart from the above approximations, discussions about package plants with Dr. Schmid of the Civil Engineering Department at Kansas State University tend to favor $30,000.00 as a good average figure for a package plant of about 20,000 gpd capacity. Accordingly, the capital cost for a package plant of 20,000 gpd will be taken as $30,000.00 in this study.

4.4.4. Operation and Maintenance

Operation cost involved is primarily the power cost associated with running the air compressor. The following calculations show how power costs are computed. Motor horsepower used in running compressors is typically 3. Since aeration is on a 24 hour, continuous basis,

$$\text{power consumed per day} = \frac{3\ \text{H.P} \times 746\ \text{Watts/H.P} \times 24\ \text{hrs.}}{1000\ \text{watts}}$$

$$= 53.7\ \text{kwh/day}$$

$$\text{say } 55\ \text{kwh/day}$$

Kansas City Power and Light was contacted for information on power costs and it is found that 2.5 cents/kwh is a good average value.

Hence, power costs per day @ 2.5 cents/kwh = $\frac{55 \times 2.5}{100}$ = \$1.38

Therefore, annual power costs = 1.38 x 365

= \$503.70 or \$504.00.

Maintenance of a package plant involves periodic checkups of the air compressor and blower unit. Scum removal, effluent sample collection for analysis and other related jobs. Assuming one man day per week for the above jobs,

Total man-hours needed 52 weeks/year x 8 hours/week

= 416 hrs/year

Assuming a skilled wage rate of

\$5.00/hour at a minimum, total wages per year 416 x 5

= \$2080.00/yr.

Total O & M costs = Power + Maintenance Costs

or

= 504.00 + 2080.00

= \$2584.00/year.

CHAPTER V

SEWER SYSTEM

5.1. General

The first procedure in the design and costing of a sewer system is the approximate calculation of sewer lengths needed to service the subdivision. The preliminary plot of the subdivision contains information about lot sizes and easements, which enable the rough computation of sewer lengths. As far as laterals are concerned the actual length will vary according to the proximity of the house to the sewers, however a typical 50 ft. per house could be used as an average figure.

Cost data were provided to the author by Dr. Larry Schmid of Kansas State University. Since the cost data are about two years old, a 10% increase is assumed to bring them to current status.

5.2. Calculation of Sewer Lengths

The subdivision consists of four major blocks which are farther subdivided into lots. Approximate sewer lengths needed are calculated on a block by block basis and then summed up. Table 6 represents the computations.

Laterals needed for 62 hours @ 50 ft/house = 62 x 50

= 3100 ft.

Provide 4 in. dia. laterals.

5.3. Manholes

Kansas State Health Department guidelines recommend that manholes should be provided at every change in grade or allignment. The maximum

TABLE 6

CALCULATION OF SEWER LENGTHS

BLOCKS	LOTS	LENGTH IN FEET
One	1-7	1350
	8-11	925
	12-14	650
	15-22	975
	23-32	900
	33-38	1300
Two	1-8	1400
	9-14	600
Three	1-5	850
Four	1-5	900
All Blocks	All lots total = 9,850 or say 10,000 ft.	

recommended spacing is approximately 400 ft. for sewers 18 in. in diameter or less; approximately 600 ft. for larger sewers. Table 7 represents the computations done to know the number of manholes needed. The total number of manholes needed comes to 38.

5.4. Selection of Sewer Diameter

The two major design considerations in selecting a suitable pipe diameter are

1. The sewer should be able to carry the expected peak flow as an open channel, that is with half full flowing condition.
2. Minimum velocity of flow must be self-cleansing that is, the flow should be fast enough so as not to cause deposition of solids inside the sewer which may eventually lead to the clogging of the system.

The Kansas Department of Health has laid down the following guidelines for sewer selection.

1. The minimum size of sewers to be 8 in.
2. Graded to give velocities of at least 2.0 ft./sec. for pipes flowing one-half full.
3. The minimum grade for a 8 in. pipe is 0.4%.

Selecting a 8 in. dia. pipe as the first choice, in light of the above guidelines, the flow capacity could now be computed.

Using the nomograph based on Manning's formula for circular pipes flowing full with $n = 0.013$ (3),

flow capacity of 8 in. dia. pipe laid to a slope of 0.4% = 0.75 cu. ft./sec.
= 484,704 gal./day

TABLE 7

MANHOLES REQUIRED

Block No.	Total length of sewer in ft.	Manholes needed @ 400 ft. spacing	Manholes for allignment change	Total
One	6100	16	4	20
Two	2000	6	2	8
Three	850	3	3	6
Four	900	3	1	4
			Total =	38

Hence, flow capacity of 8 in. dia. pipe acting as an open channel flowing 1/2 full = 484,704 x 1/2

= 242,352 gal./day

The projected peak flow for the subdivision (Section 1.5) = 45,000 gal./day. Therefore an 8" sewer is actually oversized for the projected flow. However, this is the legal minimum needed and also future additions might need the additional built-in capacity.

In special cases, it might be possible to reduce the pipe diameter to 6 in. at the most. To provide an alternative sewer system at a lower cost assume that a 6 in. dia. pipe is another choice. Flow capacity of a 6 in. dia. pipe will be checked next.

Flow capacity of a 6 in. dia. pipe laid to a slope 2.5% or giving a flow velocity of about 3.6 ft./sec. = 0.70 cu.ft./sec.

= 452,390 gal./day

Hence, open channel flow in a 6 in. dia. pipe = 452,390 x 1/2

= 226,195 gallons

Thus, the required flow of 45,000 gpd could be easily accomodated in a 6 in. dia. pipe. This 6 in. pipe can serve future additons up to a maximum of 226,195 gallons. To calculate the number extra additions the sewer can handle devide (226, 195-45,000) by 45,000 or 4 similar subdivisions could be added without overloading the system. The additional number of people that could be accomodated = 186 x 4

= <u>744</u>.

Eventually the subdivision could reach a total population of 186 + 744 or 930 and still be safely serviced by a 6 in. dia. pipe.

5.5. Cost Estimates

Cost estimates will be done for both 8" and 6" diameters to see the difference in cost and hence to select a more economical system.

5.5.1. Eight Inch Pipe

Assuming a polymer pipe, cost of 8 in. dia. pipe	= \$2.40/L.F.
Excavation, backfilling, and alligning pipe etc. complete	= \$1.50/L.F.
Total cost for 8" dia. pipe in situ	= \$3.90/L.F.
Therefore, total cost of sewer system @ \$3.90/L.F. for	
10,000 ft. (Section 5.2)	= 10,000 x 3.90
	= \$39,000.00
Cost of 4 in. dia. polymer lateral	= \$1.60/L.F.
Excavation and other charges	= \$1.50/L.F.
Total	= \$3.10/L.F.
Therefore, total cost of laterals @ \$3.10/L.F. for	
3100 ft. (Section 5.2)	= 3100 x 3.10
	= \$9610.00
Average cost of a standard manhole	= \$400.00
Therefore total cost of manholes @ \$400.00/one for	
38 nos. (Section 5.3)	= 400 x 38
	= \$15,200.00
Hence total for system	= 39,000 +
	9,610
	15,200
	= \$63,810.00
Add 10% for inflation as prices used are 2 years old	= \$6,381.00
Grand total at current price	= \$70,191.00

5.5.2. Six Inch Pipe

For a 6 in. dia. system everything remains the same except the price of the 6 in. dia. piping which is calculated next.

6 in. dia. polymer pipe cost		= \$1.90/L.F.
Excavation, backfilling etc. complete		= \$1.50/L.F.
	Total	= \$3.40/L.F.
Total cost for 10,000 ft. @ \$3.40/L.F.		= 10,000 x 3.40
		= \$34,000.00
Total cost for laterals, as before		= \$9,610.00
Total cost for manholes, as before		= \$15,200.00
	Hence, total	= \$58,810.00
Add 10% to bring the total to current level		= \$5,881.00
	Amount total at current price	= \$64,691.00

Hence by using a 6 in. dia. pipe instead of a 8 in. dia. pipe a savings to the order of 70,191 - 64,691 or \$5,500.00 could be had. Since the economic savings are so small, it is better to stick with the 8 in. dia. pipe line which is legal.

Hence an 8 in. dia. pipe will be used for the sewer system.

CHAPTER VI

COST EFFECTIVENESS ANALYSIS OF ALTERNATIVE SYSTEMS

6.1. General

Analysis of the three wastewater treatment systems will be done by utilizing the standardized cost effectiveness methodology outlined by Kazanowski (12). Using this method has some of the following advantages.

1. It includes both quantifiable and nonquantifiable or intangible criteria.
2. It emphasizes subjective ranking of the criteria.
3. It provides a means of updating and continual feedback in each step of the procedure.
4. It accounts for uncertainity in data.
5. It organizes the information of a problem into meaningful steps.

The main aim of cost-effectiveness analysis is not so much to give the correct answers always as to subject the various alternatives to a vigorous probing and thus help the decision maker in getting a clear picture of the pros and cons of the options.

Kazanowski (12) sums up this as below:

> A cost-effectiveness evaluation is deemed good if it is derived in conformance with state-of-the art techniques. Whether its conclusions are subsequently proven right or wrong is immaterial. Its purpose is to clarify complex interrelationships between choices, and thus generate a rational consensus for action.

6.2. Prerequisites

Kazanowski (12) outlines the following prerequisites necessary to be met before an actual cost-effectiveness analysis could be launched.

1. Common goals, purpose or mission of the systems.
2. Alternative means of meeting the goals must exist.
3. Constraints for bounding the problem must be discernible.

Conceptual relation among these three steps could be graphically put as shown in Figure 10. As shown in Figure 10, the constraints are important as they impose limitations within which framework the evaluation has to be based. Costs, time, technical capability, manpower inputs are constraints, to name only a few. Goals come next in sequence but are equally important for without clear and explicit goals it is difficult to evaluate the alternatives. A detailed discussion of a goal formulation strategy for Urban Water Resources is indicated in a recent study (14). Finally, alternatives have to be identified and all possible and relevant alternatives have to be taken into account to make the analysis complete.

6.3. Modified Standardized Methodology

Poporich et al. (13) have applied a modified version of Kazanowski's standardized methodology to the analysis of solid waste management in the city of Tuscon, Arizona. The analysis of the wastewater treatment systems for the subdivision is very similar to the analysis done by Poporich et al., consequently the modified version of the standardized approach to cost effectiveness is adopted in the ensuing discussion.

The following ten steps from the skeleton of Kazanowski's approach in the modified version:

1. Define the system goals or objectives.

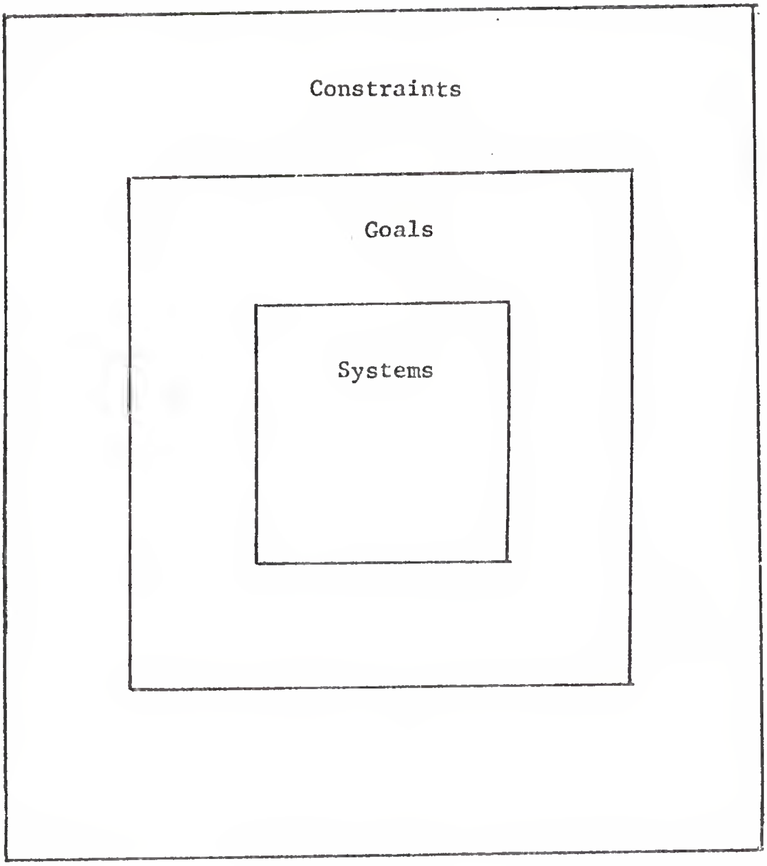


Fig. 10. Conceptual Relation Between Constraints.

2. Identify the systems specifications for achievement of these goals.
3. Establish measures of effectiveness that are to relate system performance to the accomplishment of goals.
4. Select a fixed cost or fixed effectiveness approach for evaluating systems.
5. Develop distinct alternative systems.
6. Determine capabilities of alternatives in terms of measures of effectiveness.
7. Generate an array of systems versus criteria.
8. Analyse merits of alternatives by ranking criteria, not weighting them.
9. Perform sensitivity analysis on all foregoing steps and feedback.
10. Document rationale and assumptions.

The above steps are for guidance only, so the order of them is not a rigid one. For the present study, however, the above order will be followed.

6.4. Application of Methodology

6.4.1. Goals.

The goals of any wastewater treatment scheme are given in general terms by the Kansas State Health Department (5). These goals are assumed to suffice for this study and they are given below:

1. To provide for an orderly method of wastewater disposal.
2. Protection of drinking water.
3. To provide for the disposal of wastewaters in such a way as to prevent public health hazards.
4. To prevent pollution of lakes or rivers.

5. To operate and maintain the wastewater disposal system at a reasonable cost.

6. To generate community and local support and concern in the system.

It may hardly be possible to achieve all of these goals at one and the same time because of conflicts among some. For example, maintaining the operating the system at a reasonable cost could not be possible if pollution of lakes and rivers is to be totally prevented. If on the other hand, some laxity is allowed as far as pollution goes, the cost might come down to a reasonable level. This obviously points out the inevitability of trade-offs which have to be made before a final decision is taken. But the cost-effectiveness approach does not so much concern itself with attainment of all of the goals as the search for a compromise or even a "satisfying solution. The effectiveness of the system finally chosen, of course, would be judged by how well the goals are satisfied.

6.4.2. System Specifications

Once the goals are enunciated, the next procedure is to define the systems specifications or requirements, and these are nothing but the constraints that bind the problem. Such constraints include legal, financial, technical and institutional aspects of the systems. For the case under study, the system specifications may be as follows:

1. Comply with federal laws, especially the "Federal Water Pollution Control Act Amendments of 1972." Since, the Kansas State Health Department bases its recommendations on Federal guidelines, compliance in the federal laws is ensured if the state guidelines are followed.

2. Comply with the state laws. Kansas State Department of Health

has indicated that secondary treatment standards have to be met for all newly constructed facilities. This means the treatment should yield an effluent having the following parameters before it could be discharged into a water body.

1. Suspended solids $\leq$ 30 mg/1
2. B. O. D. at 5 days $\leq$ 30 mg/1
3. Fecal coliform count $\leq$ 30 MPN/100 ml

3. Meet the Geary County sanitation laws. Since the county bases its recommendations on state guidelines, compliance with state guidelines should take care of this.

4. Stay within reasonable cost levels especially since the developer dealt within this study does not have extensive financial resources enjoyed by the larger development corporations. It is to be expected that the lowest priced system would be attractive for the developer.

5. Meet the present treatment load which is at 100 g/capita/day is around 20,000 gpd.

6. Have some flexibility for any future expansion as the reserve capacity is essential for future additions.

7. Have flexibility for phased development, in other words the initial cost of the system should justify the returns to the developer, so that there are no undue financial risks and burdens imposed.

8. Have compatibility with present institutional setups in terms of construction, operation and maintenance.

6.4.3. Measures of Effectiveness

Measures are necessary to evaluate the alternative systems in terms of the stated goals as well as specifications. To some extent, measures have been identified in the previous stage as specifications - for example "reasonable cost" is specification and measure. Traditional analysis focuses all its thrust on cost measures. In this study, in addition to cost measures noncost measures of effectives also will be considered.

Cost measures of effectiveness include

1. Capital costs for land and equipment.
2. Maintenance cost.
3. Operation cost.
4. Expansion costs for future land needs, equipment needs etc.

Noncost measures of effectiveness include ground water and surface water pollution hazards, visual and esthetic pollution or impact, reliability of operation, ie. the possibility of breakdown, future expansion possibilities, and staged development. At these time, none of these intangible are listed in any order of time, but these will be ranked later. It is also recognized that this list may not cover all the possibilities. However, as compared to traditional analysis this is a broader approach. The nonquantifiable criteria will be ranked on a scale, such as excellent, fair, and poor. This procedure allows consideration of these terms with the measures of effectiveness that are tangible.

6.4.4. Fixed-Cost or Fixed-Effectiveness

The circumstances under which either fixed-cost or fixed-effectiveness approach is to be preferred is discussed very lucidly by Kazanowski in his study (12). Suffice it to say that the fixed-effectiveness approach appears to be the most likely method to deal with real world situations. It is recognized that the developer's funds are restricted and so it would seem that a fixed cost approach is more appropriate. However, this is not possible because the effectiveness specifications are pretty much regulated by the law, and also for the protection of the environmental quality it is undesirable to vary effectiveness. Hence in this analysis the fixed-effectiveness approach will be utilized. Numerous instances (16) of individual disposal system failures justify this approach.

6.4.5. Alternative Systems

As mentioned in Section 1.3, the alternative systems to be analyzed for cost effectiveness are

1. Septic Tanks
2. Stabilization Ponds

and 3. Package Plants.

6.4.6. Capabilities and Merits of Each Alternative

Before capabilities and merits of any system can be evaluated, it is necessary to first obtain a clear picture of the associated costs. For this reason, the object of the first part of this discussion will be to compute the annual system costs of each alternative system based on the computations done in Chapters II, III, IV and V. For annual cost comparisons, it is necessary to assume an interest rate which represents the cost of the capital. Assuming this interest rate as 7% per year and assuming a capital recovery period of 20 years.

Capital recovery factor = 0.09439 (17). This factor will be used in the computations of amortisation cost of the systems.

6.4.6.1. Annual Cost of Septic Tank

As calculated in Section 2.6,

Cost of septic tank and tile field	= \$1108.00/house
Therefore amortized annual per house	= 1108.00 x 0.09439
	= \$104.58
Annual maintenance cost (Section 2.6)	= \$25.00
Total annual cost per house	= 104.58 + 25.00
	= \$<u>129.58</u>

6.4.6.2. Annual Cost of Stabilization Pond

As calculated in Section 3.5,

Total capital cost of stabilization pond	= \$17,910.00
Hence total amortized annual cost	= 17,910.00 x 0.09439
	= \$1690.52
Total annual operating and maintenance cost (Section 3.5)	= \$720.00
Hence total annual cost	= 1690.52 + 720.00
	= \$2410.52
Annual cost per house	$= \frac{2410.52}{62}$
	= \$<u>38.88</u>

It is to be noted that the sewer system cost has to be added to the stabilization pond construction cost to get the entire system cost. Sewer system cost will be computed subsequently.

6.4.6.3. Annual Cost of Package Plant

As calculated in Section 4.4.3,

Total capital cost of the package plant	= \$30,000.00

Hence, total amortized annual cost = 30,000.00 x 0.09439

= \$2831.70

Operating and maintenance costs per year = \$2584.00

Therefore total annual cost for the system = 2831.70 + 2584.00

= \$5415.70

Hence annual system cost per house = $\frac{5415.70}{62}$

= \$87.35

Again, the above does not include sewer costs.

6.4.6.4. Annual Cost of Sewer System

As calculated in Section 5.5.1,

Total capital cost of an 8 in. dia. network = \$70,191.00

Therefore amortized annual cost = 70,191.00 x 0.09439

= \$6625.33

Operations and maintenance costs for a system of this size were found to be negligible after a discussion with the consulting engineers and others. Hence they can be neglected without a significant impact on system cost.

Hence system cost per house per year = $\frac{6625.33}{62}$

= \$106.86

A summary of the various system costs is given in Table 8. The various system costs without and with sewers are given with the idea of bringing out the major cost influence of sewers on total system costs.

6.4.6.5 Discussion of System Capabilities and merits.

The septic tank system is capable of meeting the wastewater treatment demand of the subdivision at a reasonable cost level. The cost level is in fact the lowest among the three systems as shown in Table 8. The primary

TABLE 8

SUMMARY OF SYSTEM COSTS

SYSTEM	TOTAL CONSTRUCTION COST $	AMORTIZED COST $		O & M COSTS $		TOTAL COSTS $	
		ANNUAL	$/HOUSE	ANNUAL	$/HOUSE	ANNUAL	$/HOUSE
Septic Tank	68,696.00	6,484.22	104.58	1,550.00	25.00	8,034.22	129.58
Sewer System	70,191.00	6,625.33	106.86	-	-	6,625.33	106.86
Stabilization Pond Without Sewer System	17,910.00	1,690.53	27.27	720.00	11.61	2,410.53	38.88
Package Plant Without Sewer System	30,000.00	2,831.70	45.67	2,584.00	41.68	5,415.70	87.35
Stabilization Pond With Sewer System	88,101.00	8,315.86	134.13	720.00	11.61	9,035.88	145.74
Package Plant With Sewer System	100,191.00	9,457.03	152.53	2,584.00	41.68	12,041.03	194.21

Note - Houses in the subdivision = 62

reason as can be seen is that a septic tank system does not need a central sewer system. Another distinct advantage enjoyed by this system is that to finance septic tank systems, the developer need not have a large working capital. Even though in Table 8, the total cost of the system is $68,696.00, this amount need not be spent all at once as the developer need provide septic tanks only to the completed houses. The septic tank, then is eminently suited for phased construction. Financing is a very real problem for the developer and a study on fringe area sanitation (18) gives 13 items of difficulties to be faced in providing sanitation facilities for small fringe communities.

Coming to the less appealing side now, the disadvantages of septic tanks have also to be mentioned. The biggest danger lies in the septic tank tile fields failing and creating nuisance conditions. There are numerous cases to prove this (16). Also, when drinking water wells are proposed on the same lot where the septic tank is located, which is the case with the subdivision under study, water contamination becomes a distinct possibility.

To sum up, it can be said that the septic tank is a simple, economical system if it is built and operated according to sound engineering practices. It is a system which does not impose undue financial burdens on the developer and is thus attractive to him. But, from the long range point of view, the septic tank is less desirable as the waste contamination hazards and nuisance potential are really great. Hence, the septic tank system can never be counted on for long range solutions - under the best conditions it can be a temporary, interim measure.

The second system, the stabilization pond, could again handle the wastewater treatment demand of the subdivision. The main point in favor of this system is its extreme low cost. As shown in Table 8, the system without sewers is only $38.88 per house. But since the sewer system is necessary to operate the pond the sewer cost of $106.88 has to be added giving a total system cost of $145.74 per house, which is slightly higher than the septic tank system cost.

But the stabilization pond despite its very low capital and operating cost presents major problems. Large land requirements and odor nuisances are of course real problems. Apart from these, there is the problem of effluent disposal. At present, there is considerable evidence in numerous instances (18) to show that the effluent form stabilization ponds may fail to meet the suspended solids and coliform bacteria criteria for effluent to be discharged into water courses. One may try to use filters and chlorination to combat these problems, but this will again add to the cost and thus the economic advantage enjoyed by these ponds is lost. The effectiveness of algae removal and chlorination is yet to be demonstrated on an economical basis (18).

To sum up, although, an extremely economical system, the stabilization pond is an inappropriate system as far as this subdivision is concerned especially for long term solutions. This is in a great measure due to effluent discharge problems associated with the system and the site limitations.

The third disposal system, the package plant is capable of meeting the wastewater treatment needs of the community admirably. The advantages of the system are considerable. The package plant can yield extremely high quality effluents in a consistent manner unlike stabilization ponds which depend on factors beyond human control such as sunlight and temperature for the quality of treatment. The land requirements are low and nuisance conditions such as odor and visual pollution are practically non-existent in comparison with the other two systems. The package plant is well suited for modular construction, that is, if the community doubles, another plant could be added and so on.

On the other hand, the system is the costliest to build and operate. For example a working capital of $30,000.00 for the plant and another $70,191.00 for the sewer line totalling $100,191.00 is essential to build the system. The system total cost is $194.21 per house and the operating cost is $41.68 per house. Financing, thus presents major headaches, especially the capital expenditures are critical. But, the system as discussed could be the best solution on a long term basis if capital problems are overcome.

6.4.7. Systems Versus Criteria Array

The summary of the previous discussions and computations is given in Table 9. This table represents an array of monetary as well as non-monetary or intangible considerations. A direct comparison between the alternative systems as to their effectives in meeting the criteria is thus shown in a clear format.

6.4.8. Sensitivity Analysis

6.4.8.1 General

Any analysis dealing with systems extending into the future always has many uncertain factors. It is therefore necessary to have continuous feedback by varying the assumptions and criteria of individual systems. The three primary areas of uncertainities are in:

1. Determination of goals.
2. Identification of criteria.
3. System capabilities in terms of criteria chosen.

TABLE 9

SUMMARY OF CAPABILITIES OF EACH SYSTEM

NO.	SYSTEM CRITERIA	SEPTIC TANK	STABILIZATION POND	PACKAGE PLANT
1	Ground water pollution possibility	Great	None	None
2	Surface water pollution possibility	None	Fair	Minimal
3	Land pollution possibility	Fair	Minimal	None
4	Total system cost in dollars per house including sewers where needed	129.58	145.74	194.21
5	Operating and maintenance cost in dollars per house	25.00	11.61	41.68
6	Financing possibilities	Excellent	Fair	Fair
7	Ease of phased development	Great	Small	Small
8	Reliability of operation	Fair	Fair	Excellent
9	Long range feasibility	None	Fair	Excellent
10	Visual and esthetic pollution possibility	Fair	Great	Minimal

The goal determination process is a dynamic one. Although it is unlikely that the state may drop the environmental goals, it may relax some of them. For example, the state may decide that drinking water contamination by septic tanks is only a long term possibility if proper precautions are taken. This will then mean meeting a demand only for the time being, thus ignoring fixed effectiveness approach. A fixed-cost approach will be needed in such a case-e.g. with only so much money available how to provide the best septic tank system at a fixed cost.

The second area of uncertainity is in establishing criteria. For example, long range implications which have been included in this study such as future expansion possibilities are uncertain at this time. At this time, it is beyond our knowledge to foresee the future housing demand in the area. Of course, a housing market study could to some extent remove this uncertainity. But, this aspect is beyond the scope of this analysis. Secondly, financing is again an area of uncertainity especially with a small developer whose resources are limited. A financial analysis is called for to remove this uncertainity. Lastly, groundwater pollution criteria ranks high in this analysis. Though this is a possible risk, it is difficult to predict exactly how large this risk is unless actual on-site studies are conducted.

The third area of uncertainity, the determination of the capabilities of each alternative, is not a static one. For example, take the case of the sewer system. The sewer system exerts the major cost influence on the total system cost of the stabilization pond and package plant alternatives. As shown in Table 8, the sewer system costs \$106.86 per house out of the \$145.74 per house for the stabilization pond and \$194.21 per house for the package plant. Speaking in percentage terms, the sewer cost represents 73% and 55% of the total system cost for the stabilization pond and the package plant respectively. Thus the total system lost is extremely sensitive to the sewer system cost.

6.4.8.2. Reduction of Sewer Size

To see whether any considerable savings could be achieved by reducing the sewer diameter to 6 in. instead of 8 in., calculations were made in Section 5.5.2. The savings turned out to be only $5,500.00 which represents the savings in reduced material costs; the labor costs not undergoing reductions. Any reduction beyond this, say to a 4 in. dia. sewer, would be totally unacceptable to the state health authorities and this is primarily due to the concern of clogging effects in such small diameter pipes.

That the cost of the sewer system is extremely sensitive to the development density is shown in an article by Dajani and Gemmell (19). Dajani and Gemmell have found that the minimum network cost occurs at around 50 persons per acre and beyond this point the costs are increasing on either side of the scale. In other words, large scale wastewater systems would incur the diseconomies of scale dictated by low density developments.

6.4.8.3. Impact of Density Increase

To show how sensitive the density function can be to sewer costs is shown in the following computations -

Assuming that the number of lots would be doubled in the subdivision from 62 to 124, the number of people accomodated @ 3 persons per lot would be 124 x 3 = 372. Since the average lot size for the 62 lots was 0.9 acres as shown in the preliminary plat, doubling the number of lots would reduce the average lot size by 50% or to

0.45 acres or 19,602 sq. ft. This lot size rules out septic tanks for single family houses because the minimum lot size required by state law (19) for on-site septic tanks is 20,000 sq. ft.

The next alternative would be to consider stabilization ponds. But it has been shown in Section 6.4.6.5 that stabilization ponds may fail to meet secondary treatment effluent criteria for discharging into water courses. Since the subdivision under study is on the Milford Lake shores and the lake being the only discharge point available, stabilization ponds would be unacceptable to the health authorities. Only when the problems of suspended algae removal and disinfection are overcome, can stabilization ponds be considered for this subdivision.

The only alternative remaining, then, is the package plant. This alternative can meet the secondary treatment effluent criteria quite reliably; but the cost was found to be critical when 62 lots were being served. To see how costs behave when 124 lots are concerned, it is necessary to consider a new design.

Population for 124 lots	= 124 x 3
	= 372
At 100 gal per capita per day, Flow	= 372 x 100
	= 37,200 gallons
Rounding off, Flow	= 40,000 gallons

Referring to the EPA study on package plants,

The cost of a 40,000 gpd package plant on the higher side	= $30,000.00 app.
Adding 50% of above, for installation and misc. expenses	
Total cost	= 30,000 + 15,000

or $45,000.00

Amortization cost of above per house $= \frac{45,000 \times 0.09439}{24}$

$= \underline{\$34.25}$

Operation and maintenance costs per year per house for a 20,000 gpd plant as in Table 8 = $41.68

Assume additional power costs @ $100.00/year (9).

Hence additional power cost/house $= \frac{100}{124} = \$0.80$

Hence total O & M costs for 40,000 gpd plant = 41.68 + 80

= 42.48

The system cost without sewers, then is 34.25 + 41.68 or $75.93. To this the sewer costs need to be added and the sewer costs are calculated below.

The peak flow at a ratio of 2.25 to 1 is 40,000 x 2.25 or 90,000 gal.

Since an 8 in. dia. pipe at a slope of 0.40% can safely carry a max. flow of 242,352 gal/day, (Section 5.4). The 8 in. sewer network is O.K.

The length and cost of laterals will only double as the houses are doubled in number.

Cost of 8 in. dia. network as before (Section 5.5.1) = $39,000.00

Cost of manholes as before (Section 5.5.1) = $15,200.00

Cost of laterals will be twice in magnitude = $ 9,610.42 x 2

= $19,220.00

Total sewer system cost = 39,000 + 15,200 + 19,200

= $73,420.00

Adding 10% for inflation as before,

current system cost = 73,420 + 7,342

= $80,762.00

Hence amortized annual cost per house $= \dfrac{80{,}762 \times 0.09439}{124}$

$= \underline{\$61.48}$

Hence, total system cost for the package plant and the sewer including O & M expenses = 34.25

42.48

61.48

= $138.21/house

This means that the package plant alternative becomes very feasible if the development density is increased from 62 lots to 124 lots. In such a case the difference in cost between the package plant alternative and the septic tank alternative is only 138.21 - 129.58 or $8.63 which is very small relative to the advantages of the former alternative as discussed earlier. Again, the problem of large initial expenditures (45,000 + 80,762) of about $126,000.00 for the system as a whole is to be faced.

6.4.8.4. Impact of Size Increase

A final sensitivity analysis is done by increasing the size of the development without changing the housing density, i.e. the lot sizes remaining the same and observing the impact on the package treatment plant system cost.

The starting point is with 62 lots,

Cost of the package plant system for the above size (Table 8) = $<u>194.21</u>/house

Doubling the development yields 62 x 2 = 124 lots

Since the density remains the same, sewer system costs for above (Table 8) = $106.86/house

As has been shown in the previous sensitivity example, the increased flow due to doubling of size could still be handled by an 8 in. dia. pipe. Hence the above sewer cost is justified.

The size of package plant for the 124 lot development as shown earlier = 40,000 gpd

Capital cost for the above as shown earlier = \$45,000.00

Hence amortization cost $= \frac{45{,}000.00 \times 0.09439}{124}$

= \$34.25/house

Operation and maintenance as shown earlier = \$42.48/house

Hence total system cost incl. sewers = 106.86 + 34.25 + 42.48

= \$183.59/house

So, there is a savings at this point due to economies of scale. To see whether the economies of scale continue with increasing size, again double the development size yielding 124 x 2 = 248 lots.

Population served by 248 lots = 248 x 3

= 744

Total flow @ 100 gal/c/d = 744 x 100

= 74,400 gal or say 75,000 gal.

Peak flow @ 2.25:1 ratio = 168,750 gal.

Hence, an 8 in. dia. system will prevail as it can carry up to 242,352 ga. (Section 5.4).

Therefore the cost of the sewer system at constant density = \$106.86/house

Capital cost of a 75,000 gpd plant (9) = \$50,000.00

Adding 50% of above for installation and other costs as earlier = 50,000.00 + 25,000.00

Total cost = \$75,000.00

Hence amortization cost $= \frac{75,000 \times 0.09439}{248}$

$= \$28.55$/house

Operation and maintenance costs for the previous plan $= \$42.48$/house

Assume additional power costs @ \$100.00/yr. (9)

Therefore additional power cost/house $= \frac{100}{248} = \$0.40$, app.

Adding this to above, total O & M cost for 75,000 gpd plant

$42.48 + 0.40$

$= \$42.88$

Hence total system cost incl. sewers $= 106.86 + 28.55 + 42.88$

$= \$178.29$/house

This shows that at 75,000 gpd, the economies of scale are still on. To understand whether the economies of scale will continue, the size of the development will be doubled again from 248 to 496 lots.

Population served by 496 lots $= 496 \times 3$

$= 1488$

Total flow @ 100 g/c/d $= 1488 \times 100$

$= 148,800$ gal.

or say

150,000 gal.

Peak flow @ 2.25:1 ratio $= 337,500$ gal.

An 8 in. dia. pipe is no more sufficient as it can carry only 242,352 gal. maximum (Section 8). Hence a 10 in. dia. pipe, which is the next immediate size is selected and the capacity checking is done next.

Flow capacity (3) of 10 in. dia. pipe laid to a slope of 0.3% and acting as open channel = 388,000 gal.

Hence a 10 in. line is O.K.

Since the diameter is changed from 8 to 10 in. the system costs will be increased and are computed next.

Since there is a 8 fold increase from 62 to 496 lots the total length of the sewer system will also increase 8 fold from 10,000 ft. (Table 6) to 10,000 x 8 = 80,000 ft.

Material costs for 80,000 ft. of 10 in. line @ 2.80/ft = 80,000 x 2.80
= $224,000.00

Excavation, back filling etc. complete for above @ $1.50/ft. = 80,000 x 1.50
= $120,000.00

Since the development has grown 8 times from 62 lots to 496 lots, the cost of laterals will also increase 8 times, i.e. from $9,610.00 (Section 5.5.1) to 9610.00 x 8 or $76,880.00

Similarly the cost of manholes will increase from $15,200.00 to 15,200.00 x 8 or $121,600.00.

Hence total system cost	= 224,000.00
	120,000.00
	76,880.00
	121,600.00
	542,480.00

Adding 10% of above for inflation	= 542,480.00
Total system cost	54,248.00
	= \$596,728.00
Hence sewer amortization cost	$= \frac{596{,}728 \times 0.09439}{496}$
	= \$113.60/house
Capital cost of the package plant for 150,000 gpd cap. (9)	= \$90,000.00
Adding 50% of above as before towards installation etc., total cost	= 90,000 + 45,000
	= \$135,000.00
Hence amortization cost	$= \frac{135{,}000 \times 0.09439}{496}$
	= \$25.62
O & M cost for previous plant	= \$42.88
Assume addition power cost @ \$100.00/year.	
Therefore, additional O & M cost/house = $\frac{100}{496}$	= \$0.21, App.
Hence present total O & M cost	= 42.88 + 0.21
	= \$43.09
Hence total system cost incl. sewers	= 113.60 + 25.62 + 43.09
	= \$182.31/house

Hence, at this point the economies of scale cease, primarily because the network expenses have increased extremely rapidly.

The results of the package plant cost variations due to increasing sizes of development but at constant density are shown in Figure 11.

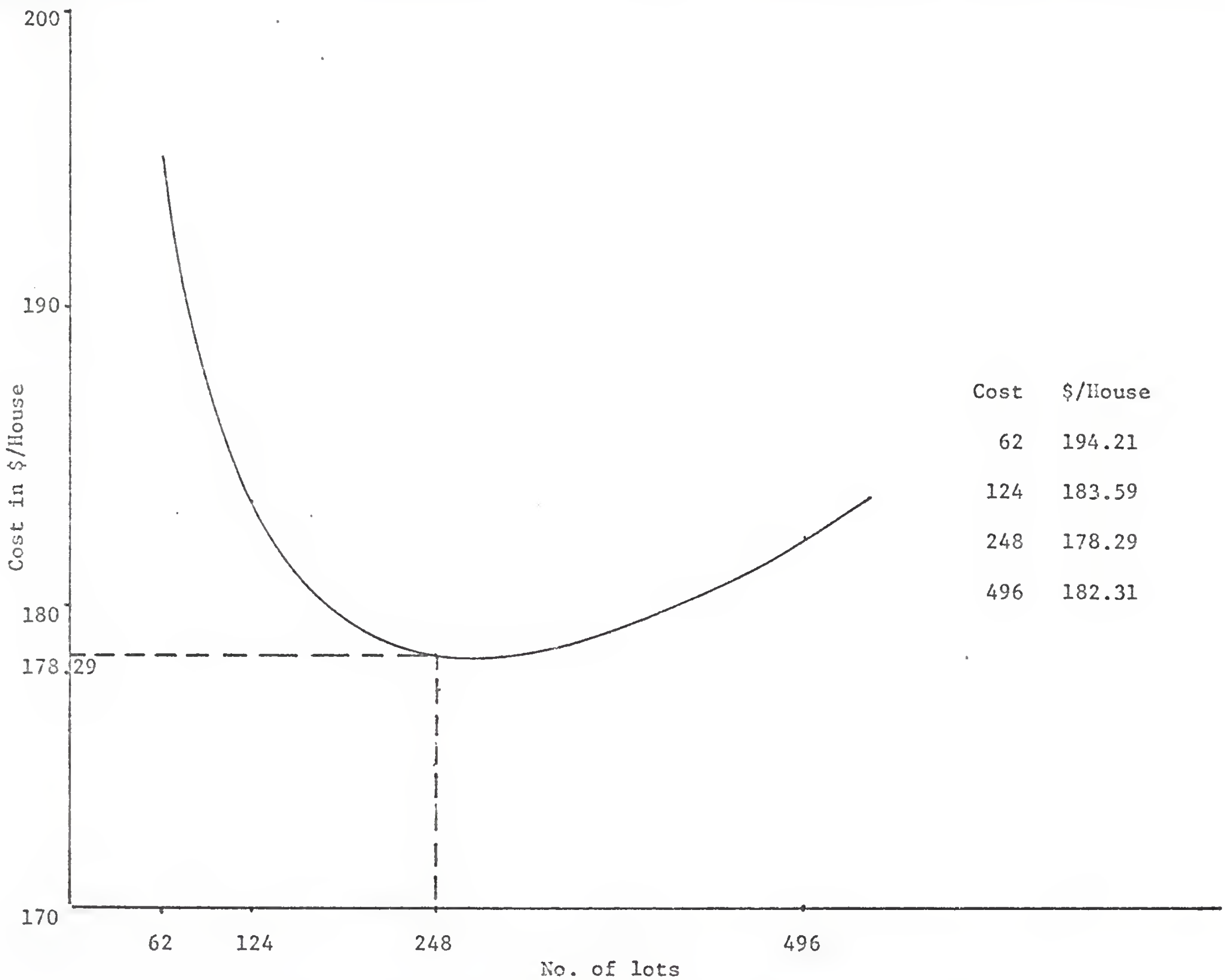


Fig. 11. Variation of Package Plant Cost on House with Increasing Development Size or Constant Housing Density.

Thus, the package plant cost per house is a minimum for development size of 248 lots, while the lots are not changed in size. As shown in the first sensitivity analysis when lots are doubled in size reducing the size of an average lot by 50% the cost of the package plant per house gets still lower. Only in the latter case does the package plant become competitive with the septic tank because increasing the development without increasing housing density tends to nullify economies of scale by increased sewer costs.

6.4.9. Assumptions

Several assumptions have been either made or implied in this study to simplify the analysis and these are noted as follows. First, although cost data used has been made as current and realistic as possible, changing technology and economic conditions may vary the costs and the results in the future. Secondly, the assumptions about the effluent disposal problems of stabilization ponds may be altered in the future as technological advances may make better control and treatment possible. This changing technology concept applies to package plants as well; for example today the capital and operating costs of these plants are high as noted in this study. The future may well change this trend as mass production and innovation in package plant technology could possibly bring the prices down. Thirdly, the septic tank method, even though the cheapest system, may not be the adequate solution if actual percolation tests on the site should give poor results. Since, actual percolation results were not available simplifying assumptions have been made giving fairly good percolation values. Finally, enough financial analysis could not be done within the scope of this study. All these assumptions would have to be fully explored in an actual study.

6.5. Summary and Conclusions

6.5.1. Ranking and Decision Maker

Since distinctly different evaluation measures are used in this analysis, it is important to decide which of these are most helpful in selecting a most satisfactory alternative. For example are groundwater pollution possibilities more important than capital costs, or short term needs more important than long range possibilities and so on. It is up to the decision maker to decide this kind of ranking or order of importance of measures or criteria. The decision maker in this case may be the developer or a consultant retained by him. In this analysis, environmental issues were placed at the top of costs because the federal and state laws stress environmental protection and also the author's perception and social preferences indicated that even though costs are important, the environment can not be regulated. Ranked below environmental issues and costs were other general criteria such as phased development possibilities, financing aspects, long range feasibility and so on. The specific ordering of the array in Table 9 was done by looking at the following questions oriented towards maintaining a balance between man and his environment:

1. What are the environmental hazards posed by each alternative?
2. What is the capital and O & M cost of each alternative?
3. What are the financing possibilities?
4. What are the chances for phased development?
5. What are the long term implications.

This is only one opinion and a different decision maker could argue otherwise. For example, the developer might be interested in short term needs only and also might not consider environmental needs as the most important ones. In such a case, a least cost approach would prevail. However, it is hoped that this analysis has shown the applicability of the cost effectiveness methodology to the problem of wastewater disposal in a small community.

6.5.2. Conclusions

This study, whether through actual analysis or through personal interviews with the developer, the consulting engineers and the academic advisors has led the author to conclude the following.

1. The septic tank system definitely is the most economical alternative if soil conditions and groundwater conditions permit the use of the system and also if the system is built and operated according to sound engineering principles.
2. The stabilization pond alternative is the next best choice in terms of economy. But as for this subdivision goes, it is not desirable as the pond effluent may fail to meet the secondary effluent standards and consequently could not be discharged into Lake Milford. Future developments in effluent treatment might change this conclusion, hence further study is necessary to check this.
3. The package plant alternative is the costliest among the three systems considered for the development size considered (62 lots). However, if the lots are doubled without increasing the subdivision size the package plant becomes very competitive with the septic tank.
4. The total system cost of stabilization pond and package plants is extremely sensitive to sewer costs. Sensitivity analysis indicates that decreasing the sewer diameter to 6 in. does not afford considerable savings.

5. Economics of scale result if subdivision size is increased at constant housing density for package plants and the minimum occurs at the development size of 248 lots. Beyond this point economies of scale disappear primarily to an increase in sewer diameter from 8 in. to 10 in.
6. Additional financial analysis should be done to see whether funds could be provided to the developer to help in meeting the large capital expenditures involved in a package plant system, should one be decided upon. Especially needed is a housing market study to know the housing demand.
7. Continuous monitoring of wells should be done to check ground-water contamination possibilities if septic tank system is chosen.
8. The package plant system should be further researched both in its technical and economical aspects as it holds considerable promise for the future.

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APPENDIX 1

LIST OF PACKAGE PLANT MANUFACTURERS CONTACTED

1. Chicago Pump, Hydrodynamics Division, 622 Diversey Parkway, Chicago, Illinois, 60614.
2. Clow Her-o-Flow Corporation, P. O. Box 223, Florence, Kentucky, 41042.

*3. Cromar Company, Cromaglass Division, Williamsport, Pa., 17701.

4. Dravo Corporation, 1 Oliver Plaza, Pittsburgh, Pa., 15222.
5. Harsco Corporation, Can Tex Industries Division. P. O. Box 340, Mineral Wells, Texas, 76067.

*6. Nepture Meter Company, P. O. Box 612, 1965 Airport Road, Corvalis Oregon, 97330.

*7. Smith and Loveless, Ecodyne Corporation, Lenexa, Kansas, 66215.

* Only these companies were kind enough to respond.

APPENDIX 2

SAMPLE LETTER

January 17, 1974

Dear Sir:

My name is G. P. Pai. I am a student at the Kansas State University working on the Master's program in Regional and Community Planning.

My Master's thesis is entitled "Examination of domestic wastewater treatment alternatives for small communities" and this study involves a comparison of septic tanks, oxidation ponds and packaged-prefabricated plants as used in treating the domestic wastewater of small communities such as subdivisions, mobile home parks, motel complexes and similar establishments.

As you may know, the published data on packaged-prefab. plants is rather diffused and hard to find and it is in this connection I need your help. I am requestiong you to be good enough to send me your latest information on these packaged plants including associated cost data. Please allow me to stress here that the cost data is important for my analysis and so would you please give the cost figures that are fairly typical as well as guiding if not very precise. Also kindly allow me to state here that all the information you supply is for my academic purpose only and not for publication.

Lastly, if you charge for any of the above, please note that you can bill me for it. I am thanking you in advance for your kind cooperation. Thank you again.

Very sincerely yours,

Mr. G. P. Pai.

APPENDIX 3

LIST OF PERSONS INTERVIEWED

1. Mr. Daniel Moske, Cedar Estates, Junction City, Kansas. (General Information on the Moske Addition to Cedar Estates).
2. Mr. John Bailey, Division of Environmental Health - Kansas State Department of Health (Regulatory Aspects).
3. Mr. Mike Butler, Schwab & Eaton, Inc., Manhattan (Preliminary Plat of the Moske Addition).
4. Dr. Larry Schmid, Civil Engineering Department, Kansas State University (Package Plant and Sewer System Discussion).
5. Dr. O. W. Bidwell, Agronomy Department, Kansas State University (Soils of Geary County).

Note: The subject of interviews is given in parentheses against the corresponding person interviewed.

APPENDIX 4

SITE VISITS

1. Visit to Cedar Estates, Junction City, Kansas, March, 1974.
2. Visit to Package Plant at the Walnut Grove Development in Manhattan, Kansas, March, 1974.
3. Visit to Package Plant at the Timber Creek Development in Manhattan, Kansas, March, 1974.

EXAMINATION OF DOMESTIC WASTEWATER TREATMENT

ALTERNATIVES FOR SMALL COMMUNITIES - A CASE STUDY

by

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A major factor influencing the health of a small community where public sewers are not available is the systematic and proper disposal of domestic wastewaters. In the absence of a good wastewater disposal system, many diseases like typhoid fever, dysentry and various types of diarrhea could be transmitted from one person to another and thus make community health unsafe.

In the past, small and isolated communities have been primarily served by septic tanks. Although when built and operated according to sound engineering principles septic tanks have provided satisfactory service, there have been numerous failures resulting in health hazards and nuisance conditions. With continuing population growth and suburbanization, the septic tank system as a wastewater disposal system needs to be reconsidered in terms of reliability and safety.

In addition to the septic tank system, this study considers the stabilization pond and the package plant as alternatives to domestic wastewater disposal. To compare the three systems realistically and conveniently a case is chosen which is the Moske's Addition to Cedar Estates near Junction City in Geary County.

The septic tank, the stabilization pond and the package plant are first designed according to standard engineering principles for the Moske's Addition with a starting population of 186 persons. Next, the three system costs are approximately estimated. Kazanowski's Cost Effectiveness Methodology is applied in the final comparison of the three systems to give equal consideration to the dollars and cents aspects as well as the intangibles. Finally, a sensitivity analysis is done by varying the sewer size, by varying the housing density and by varying the subdivision size.

It is found that in terms of capital cost, short term needs, financing convenience and phased construction, the septic tank system appears to be an adequate answer. However, due to groundwater contamination hazard and other problems, this system has to be carefully reconsidered. The stabilization pond compares favorably with the septic tank in terms of cost; yet is unsuitable for this subdivision due to the problems of effluent disposal. The third system, the package plant has the highest capital and operating costs among the three systems. Despite this high cost, the package plant could be the most desirable system to meet long term needs reliably. But as far as a small developer is concerned, the high working capital needed to obtain a package plant system is rather discouraging.

Sensitivity analysis reveals that a reduction of sewer diameter from 8 in. to 6 in. does not appreciably reduce sewer system costs. The most drastic cost reduction occurs when the housing density is doubled from 62 to 124 lots without increasing the subdivision size in which case the package plant becomes very competitive with the septic tank. This reduction in cost is attributed primarily to the reduction in sewer costs illustrating the fact that the total system costs are extremely sensitive to sewer costs. The final sensitivity analysis reveals some economies of scale in the package plant system with increasing subdivision size at constant housing density and these cease at 248 lots or about 744 persons after which the system costs go up due to an increase in sewer costs. However, despite the economies of scale with increasing subdivision size at constant housing density the package plant costs are found incompetitive with septic tank costs.