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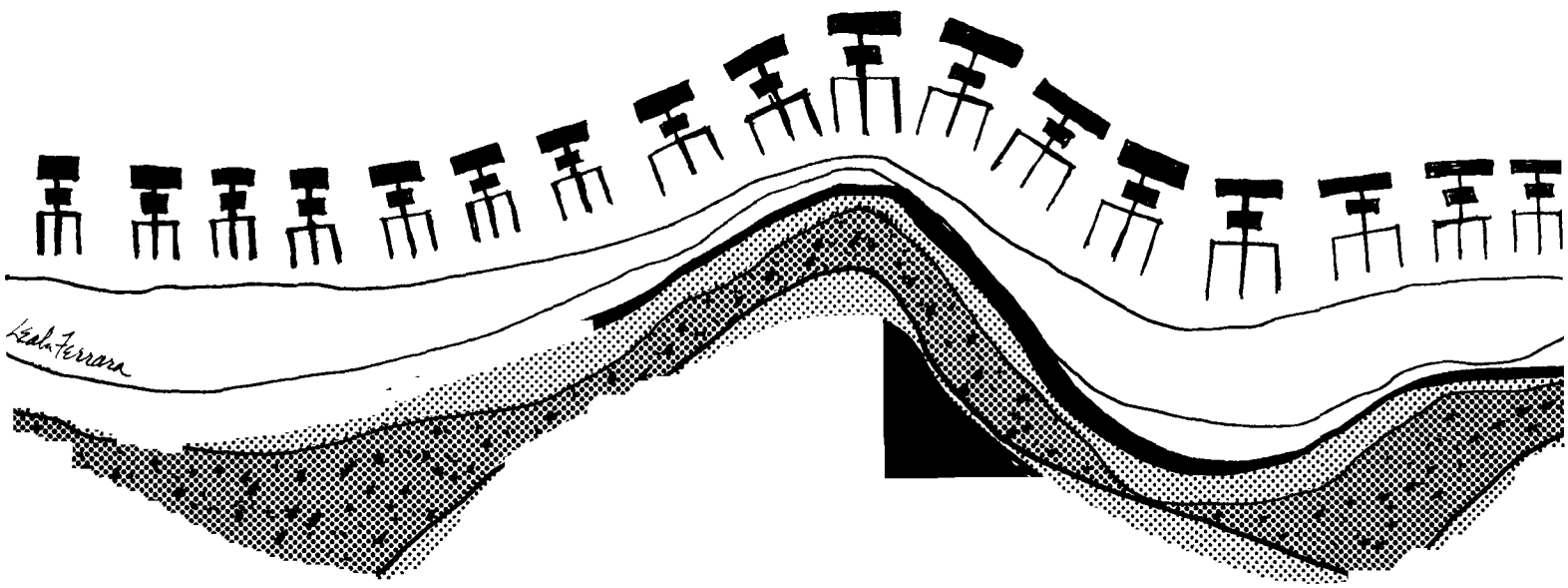
Final-Generic Environmental Impact Statement Wastewater Management In Rural Lake Areas



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Supporting the
Generic Environmental Impact Statement
for
Wastewater Management in Rural Lake Areas

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MUNICIPAL FACILITIES BRANCH
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DISCLAIMER

These Technical Reference Documents have been reviewed by the Water Division, U.S. Environmental Protection Agency, Region V, and approved for publication. Approval does not signify that the contents necessarily reflect the views and policies of the U.S. Environmental Protection Agency, nor does mention of trade names or commercial products constitute endorsement or recommendation for use.

FOREWORD

The Construction Grants Program, authorized in Title II of the Federal Water Pollution Control Act of 1972, provides federal grants to municipalities and other eligible recipients for the purpose of planning, designing, and constructing wastewater facilities. The objectives of the Act and of the Construction Grants program "is to restore and maintain the chemical, physical, and biological integrity of the Nation's waters."

Region V administers the Construction Grants program for the states of Ohio, Indiana, Illinois, Michigan, Wisconsin, and Minnesota.

In the mid-1970's we became aware of a class of grant applications that had the following characteristics:

- the communities to be served by the proposed facilities were developed around lakes in rural areas,
- the communities had a high proportion of seasonal residents,
- the proposed facilities were centralized sewers and treatment plants, and
- the costs per household were among the highest seen by the Region.

As a class, these applications presented one of the most complex sets of environmental, social, and technical issues that the Region has faced.

In 1977 we selected seven representative projects to study in detail through the preparation of environmental impact statements. These seven case studies provided the factual basis for subsequent preparation of Region V's Generic Environmental Impact Statement for Wastewater Management in Rural Lake Areas, the final version of which was published in January 1983.

The Technical Reference Documents that make up the document you are reading now constitute an intermediate step between the seven case studies and the Generic EIS. The many issues, methods, and alternatives that were recognized during preparation of the case studies are addressed in depth in these Technical Reference Documents. The Region reviewed these reports during preparation of the Generic EIS. These documents contain much valuable information that could not be included in the Generic EIS because of space limitations. However, they should be viewed as working papers, not final EPA positions. Some of the information has been outdated by the Municipal Construction Grants Act of 1981 and by new regulations implementing the Act. The Region is publishing the complete set of Technical Reference Documents to make all of our work available to the public and to provide additional background on the issues and recommendations presented in the Generic EIS.

PREPARERS

These Technical Reference Documents were prepared by the staff of WAPORA, Inc., Chevy Chase, Maryland. Mr. Gerald Peters, Jr. was WAPORA's Project Manager and Principal Investigator. Mr. Eric Hediger was WAPORA's Assistant Project Manager. Four of the Technical Reference Documents were prepared by the Clean Water Fund, under the direction of Mr. Larry Silverman.

The U.S. Environmental Protection Agency's Project Officers were Mr. Theodore Rockwell and Mr. Jack Kratzmeyer, of Region V's Environmental Impact Section. Mr. Alfred Krause, Region V's Small Waste Flows Specialist, also provided invaluable guidance.

WAPORA's and the Clean Water Fund's project staff, their areas of expertise, and Technical Reference Documents for which they were principally responsible are listed below:

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PART ONE

TECHNOLOGY ALTERNATIVES

CHAPTER 1

FACT SHEETS

FLOW REDUCTION - TOILET TANK DAMS FACT SHEET 1.1

October 1980

DESCRIPTION

Toilet tank dams are devices designed primarily to save water in existing toilets by reducing the amount of water per flush. They are inserted in the tank on each side of the drain to retain excess water not required for flushing. The dams, which are made of plastic, fiberglass, or rubber-coated metal, are flexed and vertically inserted in the toilet tank to form the "dams" that retain water. In California's pilot water conservation program (12), dams were the most successful of all toilet tank devices used for saving water. Public acceptance was quite favorable.

COMMON MODIFICATIONS

Dam devices can be modified to provide a "dual flush" for existing toilets. By removing a small semi-circular section at the base of the unit and then quickly depressing and releasing the handle, a reduced flushing action occurs. By holding down the handle, the water in the tank drains and a full flushing action occurs.

TECHNOLOGY STATUS

Toilet tank dams are fully developed and simply constructed (28). Various municipal codes would need to be changed if the dams reduced the flush volume below that required (53).

APPLICATIONS

This type of flow reduction device can be used to save fresh water, thereby lowering one's water and sewer bill and extending the life of an on-site disposal system or prolonging fresh water availability, especially in times of drought. Annual savings are reported to range from 4.4% to 7.5% of a family's total water consumption (indoor and outdoor) (53).

LIMITATIONS

California's pilot water conservation program (12) indicates that one of the problems with the dams is the need for double flushing to ensure proper operation. However, proper adjustment of the dams alleviates this problem. In some cases, inefficient or low-volume toilets do not allow for use of dams.

TYPICAL EQUIPMENT/NO. OF MFRS.

There are many types of devices and at least 15 manufacturers (12).

PERFORMANCE

Toilet tanks often supply more water than is needed for efficient operation of the toilet bowl and trap. Dams take advantage of this fact by reducing the water used while allowing enough flush water to adequately remove solids (28). In a test of 13 devices, the average reduction of flushing water was 30% (12).

RELIABILITY

Of the 13 devices tested in the California pilot study, 12 offered a warranty ranging from one year to the lifetime of the fixture (12). Owing to the corrosion-free materials used for these devices, the units themselves should not create problems of reliability. Adjustment problems, as mentioned under "limitations," might be the only problem to affect reliability.

ENVIRONMENTAL IMPACT

Toilet tank dams have a beneficial impact. For a small cost, about 30% of the water normally used for flushing can be saved; this reduction affects the size of transmission systems or postpones the need for expansion.

REFERENCES

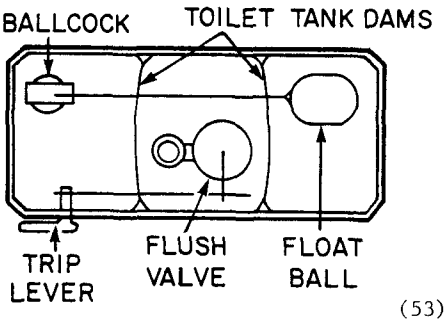
12, 16, 53, 28.

FLOW REDUCTION - TOILET TANK DAMS FACT SHEET 1.1

October 1980

DIAGRAM

TOILET TANK DAMS



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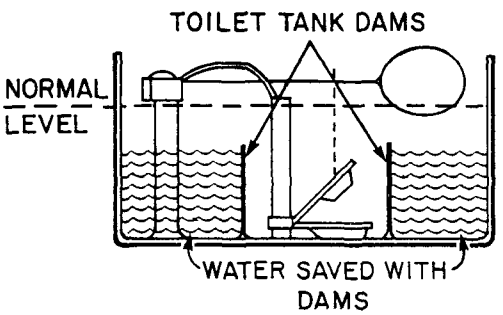


(53)

ENERGY NOTES

No hot water is saved. However, water demand and wastewater generation changes affect energy requirements for treatment, supply, and disposal (12).

COSTS

1980 dollars; ENR = 3260. Toilet tank dams cost from $5 to $9 for a set to fit one toilet. No maintenance is required, no tools are required for installation, and installation takes less than 5 minutes (12).

FLOW REDUCTION - SHALLOW TRAP TOILETS FACT SHEET 1.2

October 1980

DESCRIPTION

Shallow trap (low flush) toilets, similar to conventional toilets but with a smaller tank, are designed to flush with less water because of the shallow trap. With a smaller elevation to overcome, the siphoning action begins with less water clearing the bowl (in the same manner as a conventional toilet). In many localities, low flush toilets are required by plumbing codes for new installations (53, 61).

TECHNOLOGY STATUS

Shallow trap toilets are fully developed and commercially available from all major toilet fixture manufacturers (53).

APPLICATIONS

Shallow trap toilets function and have similar applications as do conventional toilets. They may be used in association with collection and centralized treatment or with on-site systems. Because of their public acceptance, they are widely used except where water supply limitations or on-site sewage disposal limitations dictate greater reductions in water use.

LIMITATIONS

With the lower flows from shallow trap toilets, the possibilities of solids settling out of the waste stream appear greater; however, studies have proven this to be wrong. Hydraulic calculations need to be made if the toilets are to discharge to an infrequently used sewer with flat grades (17).

Difficulty in using additional water saving devices (such as dual flush or dam devices with shallow trap toilets) has been reported (16).

TYPICAL EQUIPMENT/NO. OF MFRS.

Shallow trap toilet/9 (53, 61).

PERFORMANCE

Flow per use = 3.5 gal. (53, 13, 61). Performance of shallow trap toilets has been acceptable as seen from the plumbing codes that now require their use (e.g. Suburban Washington and California). In a test program, shallow trap toilets were installed in 6 houses; 83% of the respondents found the performance of the toilet acceptable and would recommend it to others (16).

RELIABILITY

In the areas of California and suburban Washington where code requirements call for shallow trap toilets, no major problems have been reported (73). The plumbing code requirements have been in effect since 1973 in the Washington area and since January, 1978 in California, which indicates the length of time that this type of toilet has given reliable service on a large scale.

ENVIRONMENTAL IMPACT

A positive environmental impact results from the use of shallow trap toilets due to the conservation of water and energy resources without a capital expenditure greater than conventional toilets.

REFERENCES

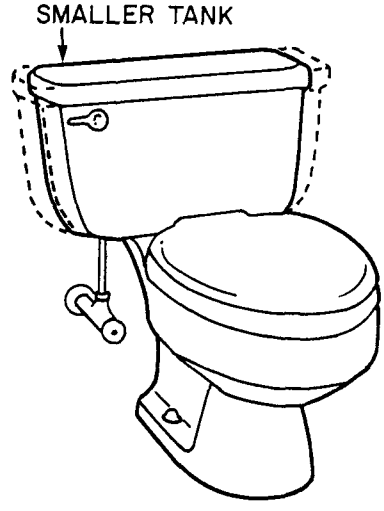
53, 61, 17, 43, 16, 28, 13, 73, 78, 74.

FLOW REDUCTION - SHALLOW TRAP TOILETS

FACT SHEET 1.2

October 1980

DIAGRAM



(53)

SHALLOW TRAP TOILET

ENERGY NOTES

No hot water is used for toilet flushing and subsequently no energy savings are realized by the homeowner in this area. Rural areas that pump water from wells realize an energy savings from reduced pumping costs due to the reduction in water from the shallow trap toilet.

COSTS

1980 dollars; ENR Index = 3260. Capital costs for low flush toilets are comparable to costs for conventional toilets.

OPERATION AND MAINTENANCE

Shallow trap toilets are similar to conventional toilets in construction and installation. In addition, their maintenance requirements do not vary greatly, even though the trapway is smaller, which means greater possibility of clogging (53).

When retrofitting with some models, the separation between the back of the toilet tank and the wall is six to nine inches, which may be aesthetically or physically unacceptable in some applications (78).

FLOW REDUCTION - AIR ASSISTED TOILETS FACT SHEET 1.3

October 1980

DESCRIPTION

This type of low flush toilet uses compresssed air and a small volume of water (2 quarts) to discharge waste from the toilet. The flushing action begins by pushing the flush handle that turns on the water and opens a flapper valve in the bottom of the bowl. When the washing action is complete, the flapper valve closes, the bowl fills up and the waste is in the chamber below the bowl. At this time a burst of air ejects the waste into the usual wastewater line (50, 52). The required compressed air is supplied by an air compressor. The forceful flushing action aids the transport of the waste where limited heads exist.

COMMON MODIFICATIONS

Air assisted toilets can be down discharging or rear discharging with rounded or elongated front. The toilets may also be fitted with a vandal-proof lid.

TECHNOLOGY STATUS

The air assisted toilet is fully developed and commercially available (53).

APPLICATIONS

The air assisted toilet, similar in appearance to conventional toilets, is satisfactory for homes with conventional sewers or alternative wastewater systems. Additional applications include recreation facilities, schools, and airports (50). Used in conjunction with holding tanks, these toilets achieve significant reductions in bacteria and nutrient content of wastewaters disposed on-site at costs competitive with other no- or low-discharge toilets and with minimal change in user acceptance.

LIMITATIONS

A number of engineers, architects, sanitarians, and supervisory personnel reported no complications in septic tanks or in the pipes preceding the septic tanks due to low flush toilets (52).

When two toilets on different floors are retrofitted with an air assisted toilet, the major difficulty likely to be encountered is supplying the required compressed air to each toilet (78).

TYPICAL EQUIPMENT/NO. OF MFRS.

Air assisted toilets and compressors/1.

PERFORMANCE

By using only two quarts of water per flush, the air assisted toilet reduces water consumption by 90% over conventional toilets (53, 50). Household water reduction is about 40%. The toilets have been approved by the Uniform Plumbing Code (UPC), the International Association of Plumbing and Mechanical Officials (IAPMO), Basic Plumbing Code of the Building Officials and Code Administrators International (BOCA), and Standard Plumbing Code of the Southern Building Code Congress International (SBCCI) (50). The performance to date has been good.

RELIABILITY

In a demonstration project resident reactions indicate that air assisted toilets perform as well as conventional toilets (78). They have been successful at highway rest stops, ski areas, Army Corps of Engineers lakes, and other installations (53, 52).

ENVIRONMENTAL IMPACT

As with other water conservation devices, the air assisted toilet is beneficial due to the water savings.

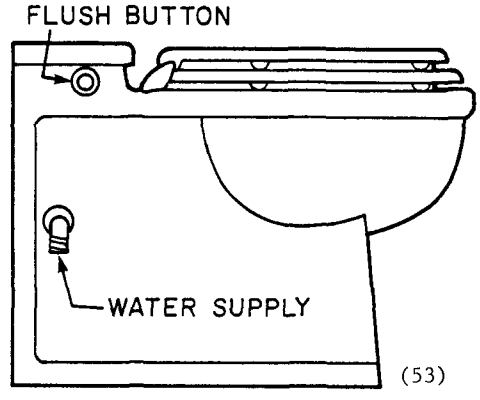
REFERENCES

50, 53, 78, 52, 3.

FLOW REDUCTION - AIR ASSISTED TOILETS FACT SHEET 1.3

October 1980

DIAGRAM



AIR ASSISTED TOILET

ENERGY NOTES

Air assisted toilets require a compressor that uses about 0.0016 kwh/toilet use. The yearly power consumption per residence would therefore be 5-15 kwh, which is minimal (78). Energy savings result from reduced water supply and wastewater treatment requirements.

COSTS

1980 Dollars; ENR Index = 3260. Capital costs for air assisted toilet systems range from $800 to $1,000. This cost includes the toilet and the air compressor. Installation costs are expected to be about $120 or 4 hours of a plumber's time (3).

OPERATION AND MAINTENANCE

During a recent demonstration project, only minor maintenance by the homeowner was necessary after the initial "bugs" were worked out. After six months of operation, the oil and air filters of the air compressor were both fine. No other maintenance was required on the system (78), which normally requires no additional maintenance than does a conventional toilet system, and possibly less. Electrical costs would be less than $1/year.

FLOW REDUCTION - LOW FLOW SHOWERS FACT SHEET 1.4

October 1980

DESCRIPTION

Three of the major types of low flow showers are 1) low flow shower heads, 2) restrictors, and 3) air assisted showers. Low flow shower heads resemble conventional shower heads but they reduce the use of water by various restriction devices such as fixed or variable orifices. Variable orifices are convex and made of rubber so that as the pressure increases, the size of the orifice decreases. Restrictors, external and internal, can be installed with existing showers. The internal restrictor fits within the shower head while the external device is a separate section of pipe that attaches between the water supply pipe and the shower head. Low flow shower heads and restrictors reduce water consumption for conventional showers from 3.0 - 10.0 gpm to 1.5 - 3.0 gpm.

In contrast, air assisted showers, which use only 0.5 gpm, operate by mixing air with water at the shower head to give a full-stream effect. Air assisted showers can be installed in existing shower stalls or with new construction.

COMMON MODIFICATIONS

Additional water can be saved by cutting off showers while soaping or shampooing. A cut-off valve on some low flow shower heads allows the water to be turned off temporarily without the need for adjusting hot and cold water.

TECHNOLOGY STATUS

Low flow shower heads and restrictors are fully developed technologically and readily available commercially (53, 17). Air assisted showers are fully developed, patented and available commercially through one manufacturer.

APPLICATIONS

Low flow shower heads are applicable to residential and institutional water conservation. Restriction devices cost less than shower heads and would be more applicable for community-sponsored distribution programs where some of the devices may never be installed. Air assisted showers would be used for maximum water (and energy) savings. Applications of the various low flow showers are a function of their cost and the amount of flow reduction desired. Both restrictors and shower heads operate at low pressures (20 psig to 40 psig) whereas the air assisted shower operates at a minimum of 35 psig. The low flow shower heads perform better than the restrictors in this pressure range.

LIMITATIONS

A drawback of the inexpensive restrictors is that they limit flows only to 3 gpm; a much greater savings can be realized when the flow rate is reduced to 2 gpm or less. However, unacceptable showers result if these inexpensive flow restrictors limit flow to 2 gpm, especially at pressures below 50 psig (73). This problem is less likely using low flow shower heads than using restrictors. Air assisted showers require plumbing changes so that the air and water can be mixed at the shower head. On the other hand, low flow shower heads and restrictors can be installed by the homeowner without any major plumbing changes.

TYPICAL EQUIPMENT/NO. OF MFRS.

Shower Heads/25

External Restrictors/13

Internal Restrictors/20

Air assisted Showers/1
(61, 12, 55)

PERFORMANCE

California's Energy Resource Conservation and Development Commission (ERCDC) tested 13 shower heads against a standard of a maximum flow rate of 2.75 gpm for line pressure between 20 and 45 psig and 3.0 gpm for a 45 to 80 psig line pressure. Of the 13 devices tested over a range

of temperatures, 5 complied and 4 did not. The other 4 were marginal, either not complying for particular temperatures or at high pressures (>74 psig). The greatest flow rate measured was 3.4 gpm in any of the tests.

Of the internal restrictors tested, only 4 out of 34 met the ERCDC criteria; the external restrictors performed much better. Eight out of 9 met the established criteria satisfactorily, although 5 had difficulty in conforming at the lowest temperature tested (76°F). A flow rate of 3.3 gpm was the highest of any of the 5 that did not conform at the lower temperature (12).

The manufacturer of air assisted showers claims that an average (4-person) household can save 88% of the shower water consumed yearly, thus reducing the shower sewage contribution by an equal amount. Energy consumed by the air assisted shower is reported to be 86% of that consumed by conventional showers (55).

RELIABILITY

Since restrictors, shower heads and air assisted showers operate, to some extent, by forcing the water through a smaller opening, the possibility exists for dirt or rust to clog the openings. The air assisted shower manufacturer recommends that a replaceable or cleanable filter precede the shower head if dirt or rust is present in the water supply system. Air assisted showers rely on electricity; shower heads and restrictors do not. The use of electricity and moving parts in the air assisted shower motor indicates a lower reliability than shower heads and restrictors, which have no moving parts and do not require electricity.

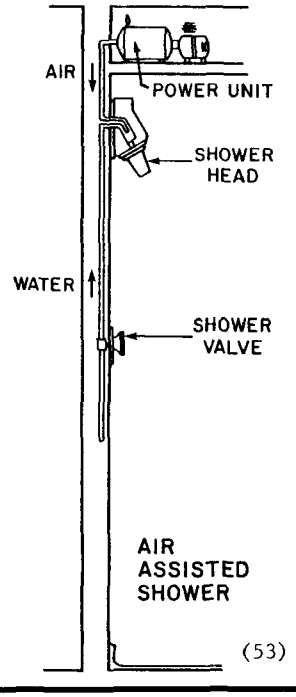
ENVIRONMENTAL IMPACT

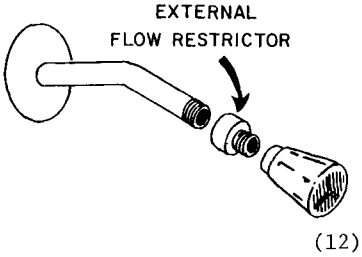
Because of the water volume and heating energy savings, low flow shower heads and air assisted showers offer more of an environmental benefit than other flow conservation devices. The greater benefit is due to the increased energy savings as a result of using less hot water. Although no reduction in pollutants is realized by these flow reduction devices, the lower hydraulic loading on a soil absorption system can increase the potential for the soil to remove these pollutants more effectively.

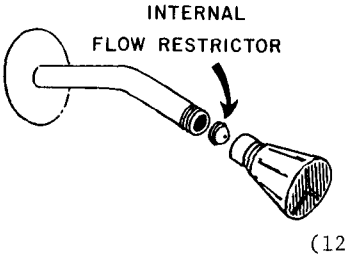
REFERENCES

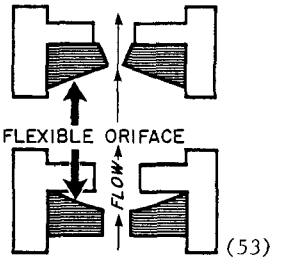
61, 12, 53, 28, 73, 80, 55, 54, 58, 21, 57.

DIAGRAM









FLOW REDUCTION - LOW FLOW SHOWERS FACT SHEET 1.4

October 1980

ENERGY NOTES

The following table compares the various shower conservation devices with a conventional shower and indicates the water and energy consumption of each. The assumed flow rates vary for different line pressures and other individual circumstances. Hot water heater efficiencies also vary depending on size, type, and other site-specific conditions. The object of the table is to show relative differences, not absolute values.

	Flow rate (GPM)	Water used (1,000 gal/yr)	NATURAL GAS			HEATING OIL			ELECTRICITY		
			Million Btus per yr	1000CF per yr	Dollars per yr	Million Btus per yr	Gallons per yr	Dollars per yr	Millions Btus per yr	KWH per yr	Dollars per yr
Conventional shower	4	29.20	15.62	15.32	60	15.62	113	111	14.74	4319	172
Shower with restrictor	3	21.90	11.70	11.48	45	11.70	84	83	11.69	3425	137
Low flow shower head	2	14.60	7.80	7.65	30	7.80	56	55	8.65	2534	101
Air assisted	0.5	3.65	1.95	1.91	8	1.95	14	14	4.08	1195	48

Assumptions

1) 4-person household, 1-five minute shower per person per day
2) Btu's required = lbs water (ΔT) (Btu/lb water°F) ÷ efficiency & standby loss, ΔT = difference in shower water temperature and temperature of water entering water heater in °F (ΔT = 105°F 55°F = 50°F).
3) Gas and oil water heaters: Efficiency = 80%, standby loss = 3.17%
4) Electrical water heaters: Efficiency = 100%, standby loss = 2.56 million Btu's per year
5) 1 CF gas = 1019 Btu's
6) 1000 CF gas costs $3.939
7) 1 gallon oil = 138690 Btu's
8) 1 gallon oil costs $0.978
9) 1 KWH = 3413 Btu's
10) 1 KWH costs $0.04

COSTS

1980 dollars; ENR Index = 3260. Costs for low flow shower heads are in the range of $5 to $20. Most restrictors cost from $1 to $5, and air assisted showers cost about $260 for all necessary materials. Installation requires approximately 4 hours of a plumber's time which costs $120 at $30/hour.

FLOW REDUCTION - CHEMICAL TOILETS FACT SHEET 1.5

October 1980

DESCRIPTION

Chemical toilets are self-contained units that store human body waste in a storage chamber along with either chemicals for odor control or enzymes for decomposition of waste. Some models use no water for flushing, are portable, and are emptied directly into a conventional toilet or a treatment facility. Flushing models use fresh or recirculated water to wash the waste into the storage container. A foot-operated pump controls the flushing action. A variety of chemical toilets is available, including models manufactured for permanent indoor residential use.

COMMON MODIFICATIONS

The number of uses prior to pumping is a direct function of the storage container size (and the reservoir size for freshwater flushing models). The capacity can be extended by using additional holding vaults. Indoor installations are available with manual, electric (AC or DC) or compressed air flushing (56).

TECHNOLOGY STATUS

Chemical toilets are fully developed technically. The chemicals used are biodegradable and do not harm septic tanks or wastewater treatment plants (53, 56).

APPLICATIONS

Chemical toilets are designed primarily for recreational, marine and industrial applications. Installations include airplanes, trains, buses, campgrounds, trailers, and others with portable and permanent uses. Residential year-round applications require a sizeable storage vault to reduce the pumping costs associated with removing the waste from the site.

LIMITATIONS

The main problem with chemical toilets for home use (other than possible code restrictions) is the need for disposal of the chemical-waste mixture. The toilet system serves as a temporary storage with disposal still necessary. The odor of the chemical might present problems indoors, but go unnoticed in an outdoor environment. Chemical toilets do not make allowances for grey water disposal and are not designed to receive any wastes other than excreta.

TYPICAL EQUIPMENT/NO. OF MFRS.

Chemical toilets/7 (53).

PERFORMANCE

United States Forest Service tests (35) on three models of chemical toilets indicate that the models are satisfactory for forest service recreation sites. Although the testing was not directly applicable to home use, it did indicate that the toilets' performance would be satisfactory.

RELIABILITY

Because of the high operation and maintenance (chemicals and pumping), the reliability would be a function of user care and upkeep. The foot-operated type chemical toilet requires no energy to operate and, therefore, functions during power failures as well as in areas where there is no electricity.

ENVIRONMENTAL IMPACT

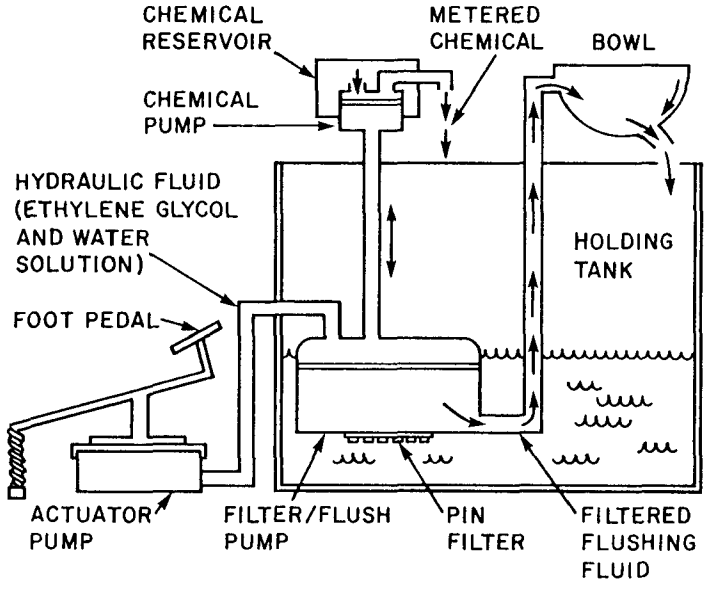
The environmental impact is beneficial from the standpoint of water conservation. However, an adverse impact is caused by the need to haul the waste away from the site for treatment and disposal. The possibility of leakage and spills exist. Use of chemical toilets on residential sites over sensitive groundwater resources will substantially reduce bacteria and nitrate loads in household wastewaters.

REFERENCES

53, 56, 35.

DIAGRAM

CHEMICAL TOILET



(56)

ENERGY NOTES

Manually operated toilets require no outside energy source for operation, however, the other types would require energy in some form depending on the type of installation (electric - AC or DC - or compressed air). Energy must be considered in the number of pumpouts (including hauling) required yearly, the ultimate treatment and disposal of the stored waste, and the manufacture of the required chemicals.

COSTS

1980 dollars; ENR Index = 3260. Capital costs for chemical toilets vary depending on size and application. The range of these costs is $700 to $1,900. Additional storage tanks would increase the costs.

OPERATION AND MAINTENANCE

Operation and maintenance costs are greater for chemical toilets than for conventional toilets. The major considerations are adding the chemicals and pumping the chemical wastes. For example, an indoor year-round chemical toilet that provides up to 1,000 uses for a family of three may have to be pumped five or six times per year. At $50 per pumping, the annual costs could be from $250 to $300 per year plus chemical costs. One gallon of chemical is required for 1,000 flushes, implying five to six gallons of chemical per year, which would cost from $42 - $50 and would bring the total annual operation and maintenance costs to $292 - $350 in this example. The use of a storage vault, the number of users, and the local cost of pumping would all affect the annual operation and maintenance costs.

SEPTIC TANK MODIFICATIONS FACT SHEET 2.1

October 1980

DESCRIPTION

1) Multiple Tanks or Chambers: An improvement to the conventional septic tank is the use of two or more compartments within a septic tank or two or more septic tanks in series. The purpose of the additional chamber(s) is the reduction of solids carry-over into the drainfield along with the prevention of short circuiting.

2) Baffles: Baffles inside the septic tank provide for proper settlement of solids and retention of scum, and prevent short circuiting. The inlet baffle directs the sewage downward and prevents the scum layer from clogging up the inlet pipe. The outlet baffle helps to retain sludge and scum in the tank thus preventing drainfield clogging.

3) Septic Solids Retainer: This outlet device is designed to allow fewer solids out of the tank by using a bell-shaped outlet pipe equipped with an insert to keep scum and solids in the tank.

COMMON MODIFICATIONS

1) Compartmentalization can be achieved by adding additional septic tanks or by using a baffle in one tank with a horizontal opening below the scum level to allow clarified effluent to flow into the second compartment.

2) Baffles can vary in location, shape, and size but should follow recommended criteria in an accepted reference (48).

3) A septic solids retainer can be installed in combination with either of the above two modifications for increased solids retention.

TECHNOLOGY STATUS

1) Available research data indicates that two-compartment tanks provide an extra degree of suspended solids removal (76). Some state guidelines recommend dual compartment septic tanks (62) which indicates the acceptance of the technology.

2) Baffles are well accepted modifications of septic tanks and are required in many areas.

3) The septic solids retainer is a relatively new device designed to keep more solids in the septic tank and out of the drainfield.

APPLICATIONS

1) Multi-compartment septic tanks can be used with newly constructed systems when solids are anticipated to be a problem, or to lessen the solids load on the drainfield even if excessive solids loading is not expected.

2) Baffles should be an integral part of all new septic tanks. Inlet and outlet baffles can prevent septic tank drainfield problems and should therefore be replaced if damaged or missing from existing tanks.

3) Septic solids retainers may be retrofitted to existing septic tanks or installed in new ones.

LIMITATIONS

1) If a single tank is divided into compartments or if multiple tanks are used, adequate ventilation is necessary to allow the escape of gases produced by the anaerobic decomposition process (48). The material used to divide a septic tank into compartments should not be subject to the corrosive nature of the septic tank environment.

2) Baffles are subject also to corrosion and should be sized and placed according to recommended guidelines. Fiberglass and acid-resistant concrete are suitable materials; metal baffles corrode easily.

3) The septic solids retainer may have a tendency to clog the outlet due to its restrictive nature. Regular septic tank maintenance, however, should avoid this problem. A septic solids retainer by itself cannot rehabilitate a failed system.

TYPICAL EQUIPMENT/NO. OF MFRS.

1) Multiple tanks or built-in baffles would be constructed by the local septic tank supplier.

2) Baffles can be built-in by the septic tank supplier or installed by the contractor or homeowner with materials supplied locally.

3) There is only one manufacturer for the septic solids retainer.

PERFORMANCE

1) In the research available, some authorities suggest that multiple tanks provide better treatment than single tanks. Although no studies have been extensive enough to provide definitive conclusions, many septic tank guidelines suggest the use of more than one chamber. Additional capacity along with walls or baffles appear to provide a greater opportunity for solids to settle out and thus reduce their presence in the soil absorption system (71, 8, 85, 96).

2) Baffles constructed of the proper materials have proved to be an effective aid to the septic tank process that retains scum and prevents short circuiting.

3) In the testing performed by the National Sanitation Foundation, the septic tank with the septic solids retainer removed 30 to 35 percent more solids than the control septic tank.

DESIGN CONSIDERATIONS

1) The first compartment (or tank) should be between 1/2 and 2/3 of the total required volume (32, 48, 96).

2) "The inlet baffle must be submerged at least 6 inches below the liquid level but must not penetrate any deeper than 20% of the liquid depth...the bottom of the outlet baffle must be submerged at a point which is equal to 40% of the liquid depth of the tank." Both baffles should extend 20% of the liquid depth above the liquid level for scum storage (48).

3) The design considerations for the septic solids retainer is handled by the manufacturer.

RELIABILITY

1) The reliability of dual tanks is the same as for a single tank provided proper connections and adequate ventilation are provided. Increased solids storage adds to the reliability of the overall system.

2) The reliability of baffles is related to proper placement and choice of materials. Outlet baffles must allow for gaseous exchange with the remainder of the tank.

3) The septic solids retainer has no moving parts and is made of a non-corrosive high density polyethylene which should provide reliable service although no long-term data is available.

ENVIRONMENTAL IMPACT

By improving the operation of conventional septic tank soil absorption systems without added operational costs, a positive environmental impact will be achieved in the form of reduced failure rates of subsurface soil absorption systems.

APPLICATION TO CLUSTER SYSTEMS

Cluster systems that operate by using individual septic tanks followed by a community treatment process benefit from the additional treatment provided by multiple chambered septic tanks, baffles, and the septic solids retainer.

SEPTIC TANK MODIFICATIONS FACT SHEET 2.1

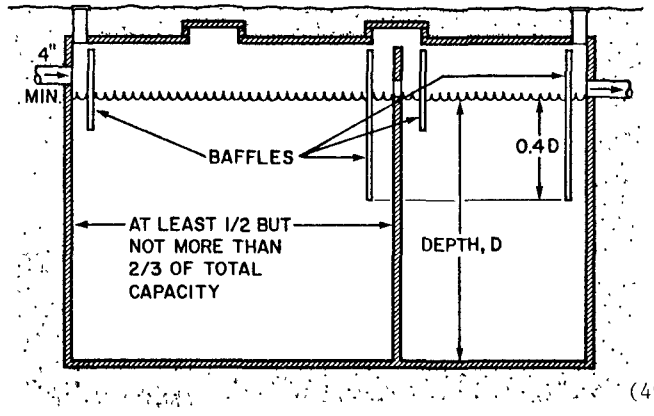
October 1980

REFERENCES

8, 71, 48, 85, 76, 32, 60, 96, 24, 62, 49.

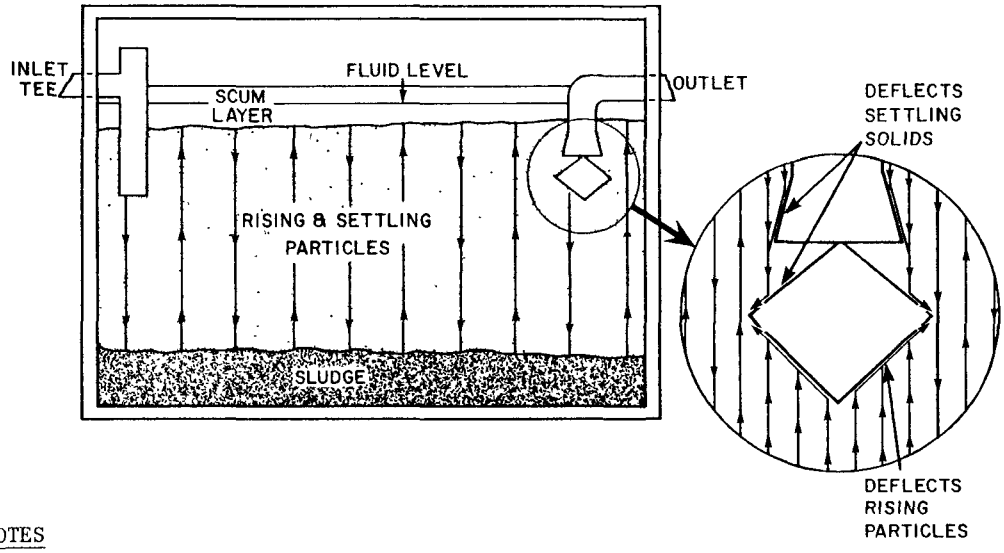
DIAGRAM

COMPARTMENTED SEPTIC TANK WITH BAFFLES



(49)

SEPTIC SOLIDS RETAINER



(60)

ENERGY NOTES

None of the three modifications discussed requires energy input after initial installation.

COSTS

1980 dollars, ENR Index = 3260

1) Capital costs for an additional septic tank would range from $350 to $750.
2) Many septic tanks are built with the baffles included. The costs of installing them should not exceed the cost of building a water-proof wall to section off a septic tank into compartments.
3) Septic solids retainers cost about $30 each with bulk discounts available.

OPERATION AND MAINTENANCE

There are no operation and maintenance requirements after installation of any of the modifications mentioned above.

GRAVITY DISTRIBUTION FOR SOIL ABSORPTION SYSTEMS FACT SHEET 2.2

October 1980

DESCRIPTION

Gravity distribution of septic tank effluent to a soil absorption system is the most common and least energy intensive method of applying wastewater to subsurface drainfields. Many methods of effluent distribution exist using various appurtances, some of which are described below.

1) Parallel Distribution is the most widely used method of gravity distribution for soil absorption systems. Both trench and bed designs fall into this category. Trenches are usually one to five feet deep and one to three feet wide. Perforated distribution piping is laid on top of a six-inch-deep layer of washed, crushed rock or gravel. Additional gravel is placed on top of the piping followed by material which keeps the dirt backfill from penetrating the rock. A series of these trenches constructed parallel to each other comprise the entire system. Beds differ from trenches in that the excavation is wider than three feet and contains several lines of distribution piping. Absorption beds require less room than trenches. Distribution lines in beds should be placed from four to six feet apart and from 1½ to three feet from the sides of the excavation.

2) Distribution Boxes are used between the septic tank (or other treatment unit) and the drainfield distribution lines to divide the effluent evenly among all distribution lines. The box has one inlet and multiple outlets, one for each line in the distribution network. All the outlets should be at exactly the same elevation and slightly below the inlet elevation.

3) Serial Distribution is a method of loading a drainfield so that the first trench is used to capacity before the second trench is loaded. This is accomplished by using plastic fittings or drop boxes at the head or the middle of each trench. Plastic fittings join distribution lines such that the opening to the second trench is at a higher elevation than the capacity level of the first trench. Because the outlet to subsequent trenches is at a higher elevation than the "capacity level" of the trench being filled, the subsequent trenches are not filled until the first trenches are fully utilized.

4) Drop Boxes are round or square boxes with removable covers containing an inlet, distribution lateral(s), and an overflow. The boxes are used for serially loaded trenches distributing wastewater through the distribution laterals until the water level reaches the overflow elevation. The overflow line then becomes the inlet for the next drop box in the series.

5) Dosing Siphons are gravity-operated devices that deliver wastewater to a soil absorption system in periodic slugs instead of at a constant trickle. A dosing siphon contains a chamber that fills up with effluent from the treatment unit to a predetermined level and then empties, dosing the entire drainfield.

6) Alternating Drainfields involve two drainfields that operate independently of each other but receive effluent from a common tank through a diversion valve. The diversion valve can direct the flow to either drainfield so that, while one is in operation, the other is resting.

7) Large Diameter Distribution Tubing is ten inches in diameter compared to four inches for standard distribution pipe. The drainage openings are in the sides of the large pipe to facilitate additional solids settling within the distribution system. An inspection port or a pump is provided at the end of the soil absorption trench to monitor the solids accumulation and to allow the system to be flushed when solids deposition reaches a certain level (1).

8) Vents (or breathers) are installed at the ends of trenches in soil absorption systems to provide a more aerobic environment in the drainfield. Vents consist of a vertical pipe extending from the bottom of the drainfield above ground and covered with a vent cap. The portion of the pipe located in the porous media section of the drainfield is usually perforated so that the liquid level can be observed (64).

COMMON MODIFICATIONS

1) Parallel Distribution can be modified by connecting the ends of the distribution lines with additional perforated piping to form a closed loop system. Septic tank effluent can reach any point in the drainfield by one of several routes. This modification avoids under utilization of drainfields when local blockages occur. Closed loop systems function properly when constructed on a level site and connected to the septic tank through a level cross, tee, or distribution box.

2) Distribution Boxes can be designed to feed two or more trenches or more and can load the trenches from the center or the ends. Distribution boxes can be either round or rectangular and should have a removable cover to observe problems (24, 71). Construction can be concrete, brick or fiberglass and may contain a baffle if high velocities are expected.

3 & 4) Serial Distribution and Drop Boxes can be used with either continuously-ponded or dosed drainfields. For dosed systems, or where inlet velocities are high, a baffle should be used to ensure that the trenches fill before the wastewater flows out of the drop box.

5) Siphon Dosing can be accomplished with either a bell or a V-shape siphon. The amount of volume to be applied can be changed by altering the size of the dosing chamber.

6) A common modification of an Alternating Drainfield involves sizing three separate fields instead of two. Each field is sized to accept 50% of the flow and two are kept in operation continuously while the third is rested.

7) Large Diameter Distribution Tubing with outlet holes located on the sides can drain into a conventional rock-filled trench or can be covered with a special fabric and can leach directly into the soil. The fabric provides filtration of the effluent and keeps the backfill material from entering the pipe while allowing the effluent to leach into the soil (1). A system called the Sheldon Network, which uses small diameter distribution tubing with the holes located on the top of the pipe, utilizes the idea of additional settlement within the drainfield tubing prior to soil absorption (75).

8) Vents can allow air to enter the drainfield by providing a connection to the drainfield from above ground. More air is allowed in the system if an inlet is provided at the end of the trench and an outlet vent is connected to the top of the septic tank. In addition, the outlet vent can be connected to a windpowered turbine on the roof of the house, which would induce fresh air while removing the gaseous build up in the drainfield and the septic tank (1).

TECHNOLOGY STATUS

All of the devices and techniques discussed here are proven and fully developed technologies, with the exception of large diameter distribution tubing which has only been available since 1978.

APPLICATIONS

1) Parallel Distribution has many applications for rural areas where soil characteristics are favorable such that higher cost alternatives are not necessary.

2) Distribution Boxes are used with conventional soil absorption systems to maintain equal flow to all distribution lines. Depending on the number of lines to be fed, the distribution box must be designed for site-specific circumstances.

3 & 4) Serial Distribution networks are used on level ground but are most useful on steep sites where parallel distribution results in overloading the bottom trench. Trenches follow contours of the land so that each trench stays level throughout its length. An advantage of serially-loaded systems is that additional trenches may be readily added to expand the system without disturbing the existing treatment process.

5) Siphon Dosing is used when a more even distribution over the entire drainfield is desired. Periodic dosing allows for resting of the system which creates more aerobic conditions leading to quicker decomposition of the biological mat that forms at the infiltrative surface.

6) Alternating Drainfields are useful wherever regular soil absorption systems are installed, providing enough land area is available. In addition to allowing a year-long resting period every other year, alternating drainfields also provide an emergency backup field in case the field in use should fail.

7) Because the main advantage of Large Diameter Tubing is its capacity to settle out additional solids, a prime site for application is where excessive solids are expected (e.g., a home with a garbage disposal). Systems with large diameter pipe and fabric covering that do not utilize

gravel in the trenches are easier to install, although care must be taken to maintain the proper slope and orientation of the holes. Another application is where an underlying bedrock or groundwater layer prohibits the use of a system that requires more depth for installation.

8) Vents are used with almost any type of soil absorption system. A major advantage to having properly installed vents is the ability to monitor the system by checking the level of the wastewater above the infiltrative surface.

TYPICAL EQUIPMENT/NO. OF MFRS.

Most of the material and equipment needed for the techniques discussed is locally available. Siphons and large diameter tubing are modifications that require specialized equipment usually available through the manufacturer's representative. Residence size siphons are available from at least two manufacturers. Large diameter tubing is available only from one manufacturer.

PERFORMANCE

The performance of any system depends on the site conditions, installation procedures, and user maintenance. If the treatment system is not located on a site with suitable soils or topography, the chances for good performance are poor. Similarly, construction procedures can produce detrimental effects throughout the life of the system, such as the compacting effect of heavy equipment on underlying soils or the overloading of areas of soil absorption systems due to poor distribution box installation. Maintenance of on-site systems is also a major factor in system performance; for example, septic tanks that are not pumped will eventually cause solids to carry over to the absorption field leading to system failure.

1) Parallel Distribution performs well when properly sited, installed and maintained. Pollutant removals are satisfactory with the exception of nitrates and chlorides in coarse aerated soils. Viruses, BOD, suspended solids, bacteria, and complex organic compounds undergo adsorption and biological degradation in the soil so that the groundwater is protected.

2) Performance of Distribution Boxes has been less than adequate. The U.S. Public Health Service discarded this technique because of poor distribution and subsequent under utilization of drainage field capacity (65).

3 & 4) Serial Distribution has proven to be a satisfactory method of effluent disposal. Some states consider Drop Boxes and serial distribution to be the most desirable method to distribute effluent (48).

5) Dosing Siphons perform well enough to be required for soil absorption systems over a certain size in some states (46). They are well accepted because of their satisfactory performance.

6) The performance of Alternating Drainfields is a function of the design, construction, and maintenance of the individual soil absorption systems. No performance problems have been noted with the diversion valves, and other components of the system are the same as conventional soil absorption systems that function well if properly designed, installed and operated.

7) There is not enough information available to judge the performance of Large Diameter Tubing.

8) Vents perform satisfactorily for systems that are not continuously ponded. As a monitoring device, vents operate well if constructed so that the bottom of the trench is visible from the ground surface.

DESIGN CONSIDERATIONS

1) Parallel Distribution systems should not be installed in soils with a coarse sand or gravel texture of percolation rates faster than .1 min/in. Silty clay loams or clay loams with percolation rates min/in should not be used either for conventionally designed trench or bed soil absorption systems. The following table gives guidelines for domestic wastewater application rates for different soil textures and percolation rates (71):

SOIL TEXTURE	PERCOLATION RATE (MIN/IN)	APPLICATION RATE (GPD/SF)
Coarse to Medium Sand	1 - 5	1.2
Fine Sand, Loamy Sand	6 - 15	0.8
Sandy Loam, Loam	16 - 30	0.6
Loam, Porous Silt Loam	31 - 60	0.45

2) Distribution Boxes should be designed so that all outlet pipes are at the same elevation and the invert of the inlet pipe is at least one inch above the invert of the outlets. The water-tight outlet pipes should have equal slopes for five feet after leaving the box, and each trench should be connected individually to the distribution box. The cover for the distribution box should be removable and a baffle should be installed in front of the inlet pipe when the velocity of the influent is expected to be high (49).

3 & 4) For Serial Distribution, Drop Boxes offer many advantages over water-tight pipe for distributing the effluent to individual trenches. Drop Boxes are usually 12 to 18 inches in diameter, and are square. The overflow invert should be at the same elevation as the crown of the distribution laterals or two inches above it so that the full depth of the trenches is flooded. If the overflow invert elevation is near the top of the rock surface in the trench, the entire trench sidewall will be used, maximum hydrostatic head will be developed on the trench bottom, and a constant supply of liquid to the capillary pores in the soil will maximize soil evaporation and plant utilization. The inlet invert may be at the same elevation as the outlet invert or a few inches higher (48, 71).

5) The main criteria for Dosing Siphons is the amount of waste to be delivered to the absorption field. This volume is determined by the size of the dosing chamber. A drainfield should not be dosed more than four times per day. The volume of each dose shall be the greater of 1) the daily wastewater volume divided by the daily dosing frequency or 2) an amount approximately equal to 3/4 of the internal volume of the distribution lines being dosed (32).

6) Alternating Drainfields should follow the design criteria for conventional soil absorption systems for both fields. The diversion valve should be of a corrosion resistant material and should be accessible from the ground surface for the annual changing of drainfields.

7) For the design of drainfields with Large Diameter Tubing the manufacturer's recommendations should be followed. For systems without gravel in the trenches, these recommendations include having approximately 12 inches of cover and being laid on a slope of 0 to ½ inch per foot of length. Also provisions must be made for washing out the solids that build up in the bottom of the pipe (1).

8) Vent Pipes should be a minimum of 4 inches in diameter and can be connected to the distribution pipe at the end of a trench with a fitting or a junction box. The vent should extend at least 12 inches above the finished grade and terminate with a vent cap. Below the connection to the distribution pipe, the vent pipe should extend to the bottom of the excavation with perforated pipe. Vents should be located at least 25 feet from any windows, doors, or air intakes for buildings (106).

RELIABILITY

1) Parallel Distribution is a reliable method of dispersing septic tank effluent to the soil. Systems properly constructed and maintained have provided useful service for more than 20 years.

2) Distribution Boxes are not reliable because of the requirement for keeping all the outlet inverts at the same elevation. Improper backfilling or differential settling over time are often reasons for distribution boxes to favor one line over others, resulting in possible system failure.

3 & 4) The reliability of Serial Distribution of effluent is greater than that for conventionally loaded systems. If the first trenches in the system fail due to clogging, the remainder of the system can still be used. The ability to rest individual trenches by plugging the distribution laterals from the Drop Boxes gives the system an added degree of reliability.

5) The addition of mechanical equipment such as a Dosing Siphon often reduces the reliability of the overall system. However, dosing increases the reliability of the drainfield because failure by clogging is less likely when aerated conditions are established between applications. Overall reliability is therefore increased over systems that are not dosed.

6) Alternating Drainfields are more reliable than systems with only one drainfield. Resting the individual drainfields provides one aspect of the increased reliability while having an emergency back-up field provides another.

7) Although little data exists, the increased solids retention in Large Diameter Tubing provides for a more reliable system.

8) Vents are more reliable for monitoring than for providing an aerobic environment in continuously ponded systems. The air entering vents is more effective in gravity dosed systems.

ENVIRONMENTAL IMPACT

Subsurface disposal systems provide sufficient filtration, adsorption, and microbial degradation of harmful constituents in household effluent. System failures, however, can occur and are related to site limitations or to improper installation, operation, or maintenance. Adverse environmental effects attributed to failures of subsurface disposal systems generally involve the contamination of ground or surface waters. This contamination then may result in public health hazards if drinking water is affected or in accelerated eutrophication if excessive amounts of nutrients are delivered to lakes or streams.

Mitigation and avoidance of adverse environmental impacts depend on information regarding site specific conditions that led to system failures in the community, the use of this information to select repairs and replacement systems and to design new systems.

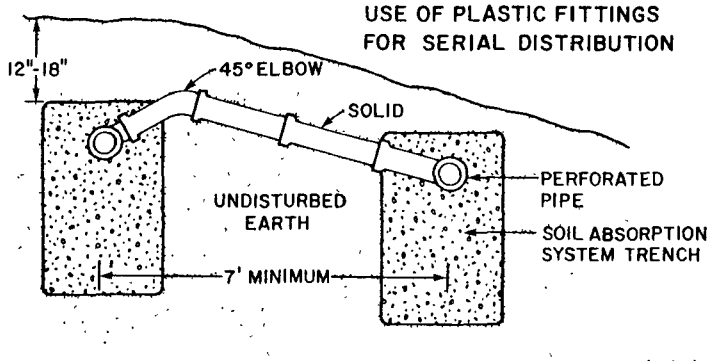
APPLICATION TO CLUSTER SYSTEMS

For large septic tank/soil absorption systems such as those used for multiple residences, gravity systems may not adequately distribute the effluent. However, dosing (gravity or pumped) and alternating drainfields can work well with larger systems. The additional maintenance required for dosed and alternating drainfields becomes more important as the risk for groundwater pollution increases with the size of the system.

REFERENCES

48, 64, 101, 44, 46, 31, 71, 76, 85, 32, 106, 24, 10, 49, 1, 75, 65, 109.

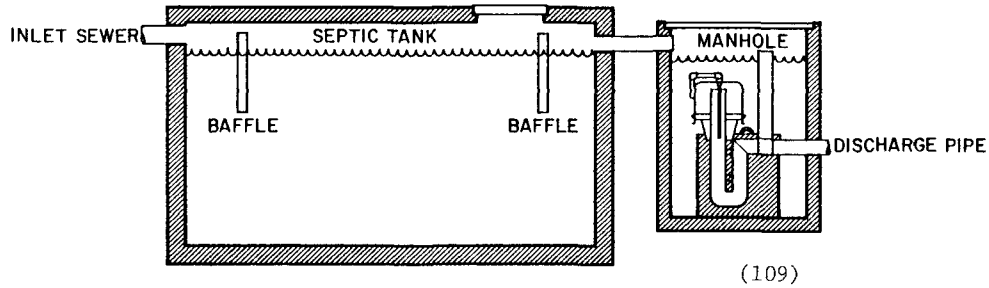
DIAGRAM



(101)

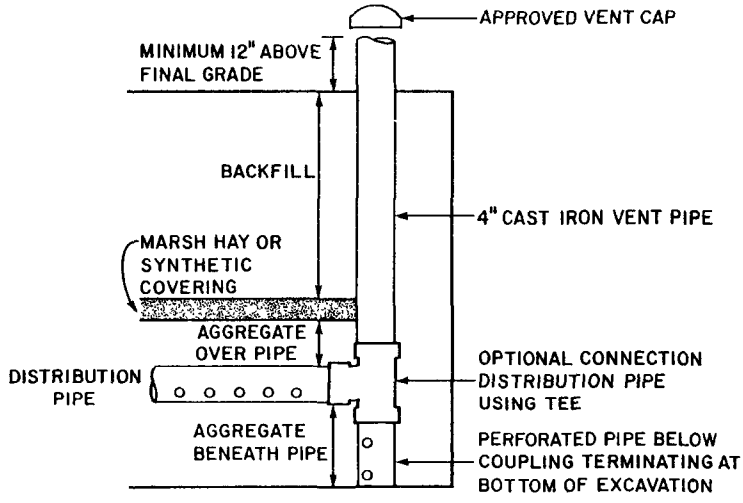
DIAGRAM

SEPTIC TANK FOLLOWED BY GRAVITY DOSING SIPHON



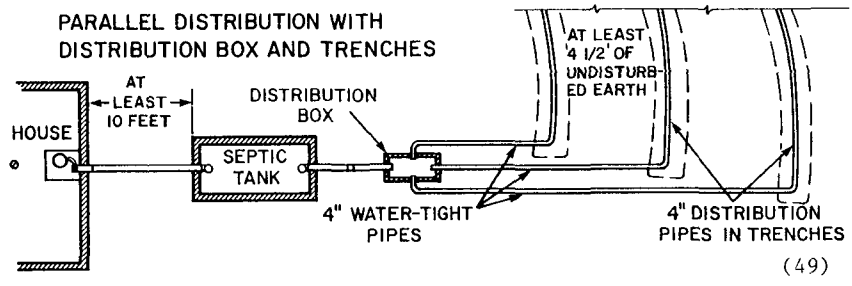
(109)

VENT OBSERVATION PIPE



(106)

PARALLEL DISTRIBUTION WITH DISTRIBUTION BOX AND TRENCHES



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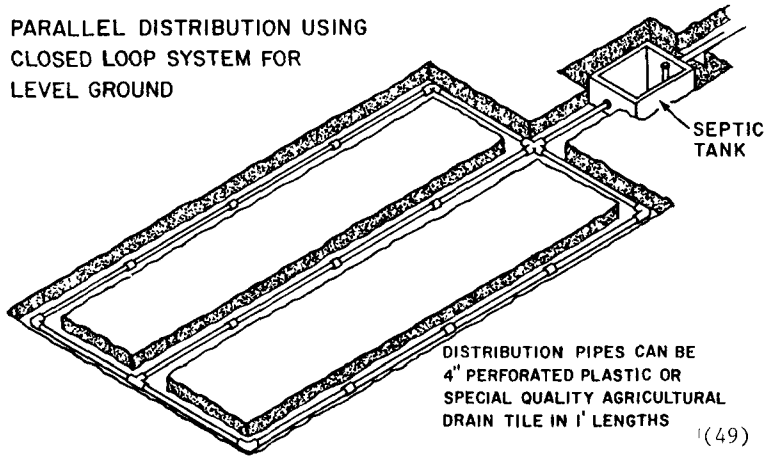

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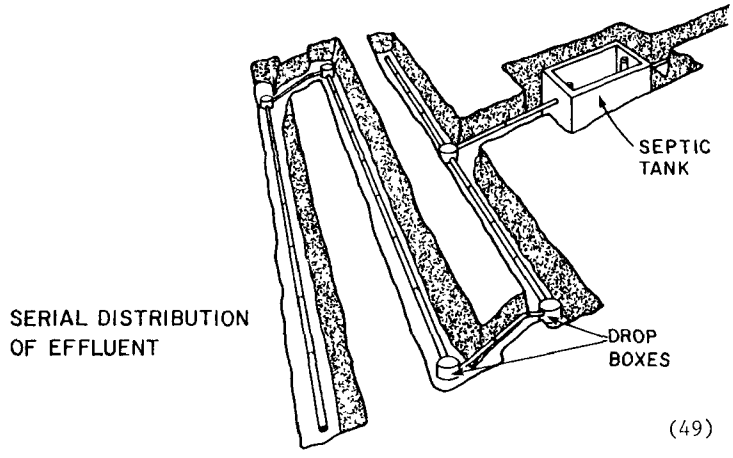


(49)

ENERGY NOTES

None of the systems or devices discussed above requires any energy input after initial installation.

COSTS

1980 dollars; ENR Index = 3260. The costs presented are those that can be expected for the construction of an entire on-site system for a 3-bedroom house unless otherwise noted.

1 & 2)	Parallel distribution with a distribution box	$1,500 to $3,000
3 & 4)	Serial distribution with drop boxes	$1,700 to $3,000
5)	Dosing siphons (siphon and chamber installed)	$ 400 to $ 650
6)	Valves for alternating drainfields	$ 50 to $ 80
7)	Large diameter tubing costs $2.10/LF the total system costs are lower than for a conventional ST/SAS if local gravel costs exceed $10 - $12 per cubic yard (personal communication with Tim Lang, ADS - 8/11/80).	
8)	Vents (each)	$ 75 - $ 100

OPERATION AND MAINTENANCE

Operational costs for all gravity fed systems would be a function of septage pumping every three to five years. An annual cost of $15 to $25 can be expected. Other routine O & M operations could be performed by the homeowner such as turning the valve on alternating systems or cleaning the dosing siphon. The actual infiltrative surface of the drainfield also should be checked periodically, perhaps when the tank is pumped.

SOIL ABSORPTION SYSTEMS - PRESSURE DISTRIBUTION FACT SHEET 2.3

October 1980

DESCRIPTION

Pressure distribution is a method of applying effluent to soil absorption systems using a pump and usually small diameter pipe with drilled holes. The system includes the idea of dosing with uniform distribution of effluent. The pressure maintained at the end of the lateral farthest from the manifold connection is about 1 to 2 psi during operation.

COMMON MODIFICATIONS

Pressure distribution systems with small diameter distribution pipes can be used with deep or shallow installations and can be recommended for use with mound systems. Pumps following septic tanks can be used to pump the effluent to a distribution box (or header) followed by a gravity distribution soil absorption system.

TECHNOLOGY STATUS

Although pressure distribution networks have not been used for very long, they are a fully developed and acceptable method for effluent distribution.

APPLICATIONS

Pressure distribution is useful for any situation where uniform application of effluent is desirable, such as with highly permeable soils to prevent saturated flow and with moderately to slowly permeable soils to retard clogging (96). Another application is for large absorption areas to ensure that the field capacity is utilized fully.

LIMITATIONS

To achieve uniform distribution, more attention must be given to the design and construction of pressure distribution systems than to conventional gravity systems. Also, pump failure would have to be repaired within a short time to keep wastewater from backing up into the house (85).

TYPICAL EQUIPMENT/NO. OF MFRS.

Small pumps (1/3 to 1/2 hp) and 1 - 3 in. PVC piping are supplied locally along with pump chambers. Holes in the PVC pipe are usually drilled by the contractor.

PERFORMANCE

Performance of pressure distribution systems is good providing that the pump is sized properly; undersizing of the pump can lead to hole clogging and sludge accumulation (19). Removing the burrs from the drilled holes prevents solids buildup and reduces head loss across the openings in the laterals.

DESIGN CONSIDERATIONS

Site-specific designs are required for each system depending on the layout of the trenches and the absorption field area requirements. To achieve uniform distribution, the volume of effluent flowing from each hole in the network should be the same. The head losses should therefore be balanced such that 75 to 85 percent of the total head loss occurs crossing the orifice, and the remaining 15 to 25 percent occurs in delivering the wastewater to each hole. The networks usually consist of laterals from 1 to 3 inches in diameter with a larger center or end manifold. The laterals are perforated at the invert with 1/4 to 1/2 inch holes spaced from 2 to 10 feet (71). Dosing volumes should be 10 times the total pipe volume and a minimum of 75 gallons. Dosing frequency should not exceed 4 times/day.

RELIABILITY

Pressure distribution networks are dependent on electricity for operation and are therefore subject to losses of power. The pump itself may fail if not checked periodically for proper operation. An indoor alarm indicating pump failure and/or high water level in the pumping chamber increases the reliability of the overall system. Replacement pumps should be readily available.

The pipe system should be installed so that after the pumping cycle, the wastewater remaining in the pipe drains back to the pumping chamber.

ENVIRONMENTAL IMPACT

Subsurface disposal systems can provide sufficient filtration, adsorption, and microbial degradation of harmful constituents in household effluent. System failures, however, can occur and are related to site limitations or to improper installation, operation, or maintenance. Adverse environmental effects that are attributed to failures of subsurface disposal systems generally involve the contamination of ground or surface waters. This contamination then may result in public health hazards if drinking water is affected or if nutrients are delivered to lakes or streams.

Mitigation and avoidance of adverse environmental impacts depend on information regarding site specific conditions that led to system failures in the community, and the use of this information to select repairs and replacement systems and to design new systems.

APPLICATION TO CLUSTER SYSTEMS

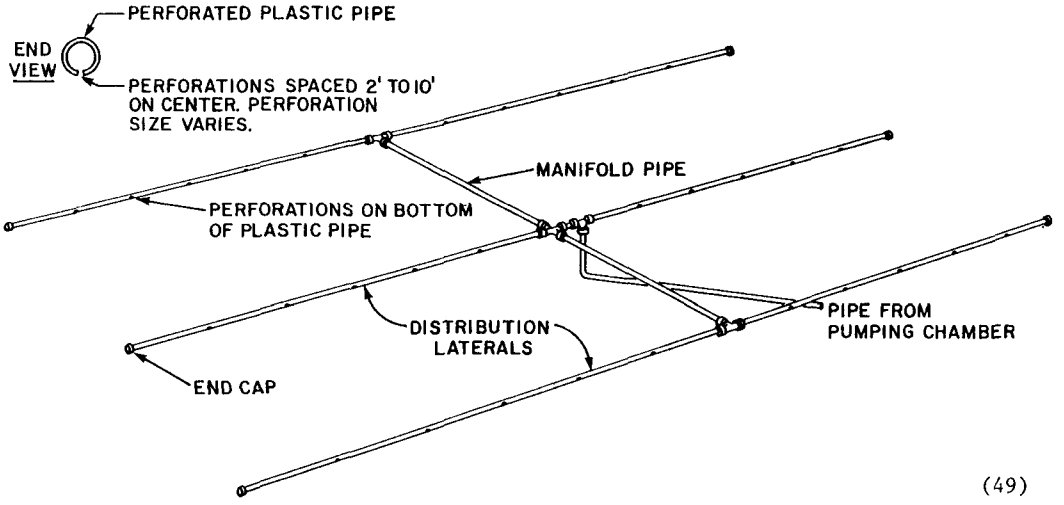
Pressure distribution is readily adaptable to cluster systems where subsurface application of wastewater is used. The advantage of pressure distribution for large fields is that the entire field is used. The idea of using alternating drainfields with pressure distribution has great potential for cluster systems.

REFERENCES

65, 19, 85, 71, 96, 24, 48, 49, 94.

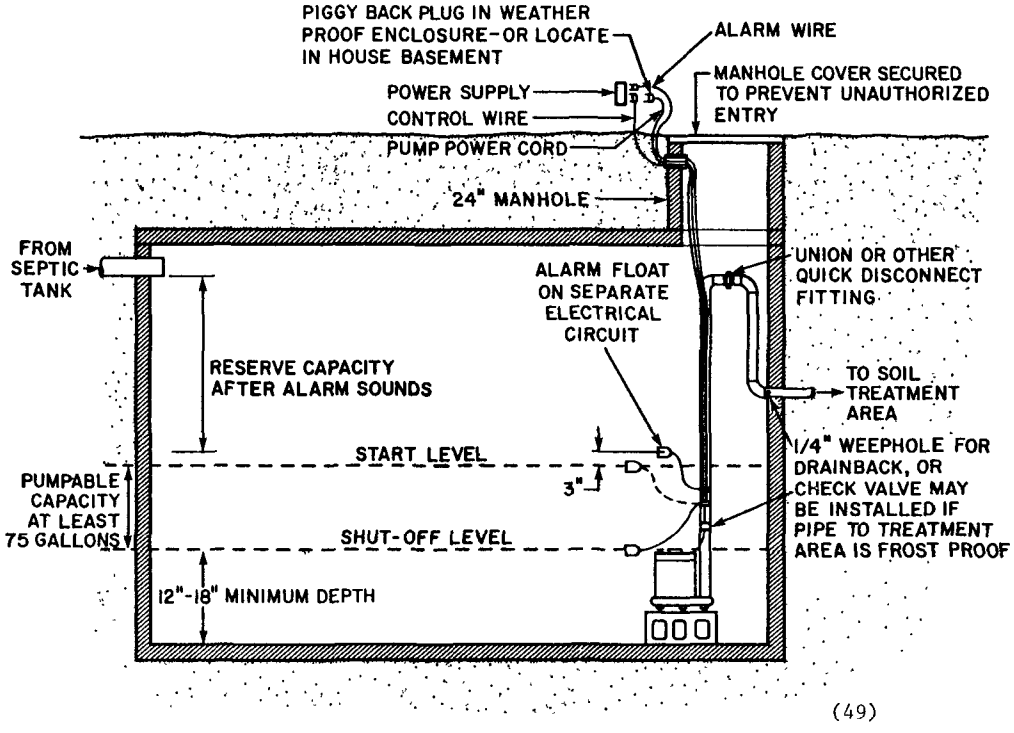
DIAGRAM

LAYOUT OF PERFORATED PIPE LATERALS FOR PRESSURE DISTRIBUTION



(49)

PUMPING CHAMBER FOR
PRESSURE DISTRIBUTION SYSTEM



(49)

ENERGY NOTES

Electricity is required to operate the pump for pressure distribution systems. A 1/3 to 1/2 HP motor is required to operate from 1 to 4 times daily. The amount of energy required is minimal for such small motors and infrequent use varying flow 50 to 250 KWH/yr (94).

COSTS

1980 dollars; ENR Index = 3260. Installed costs for an individual pressure distribution system (assuming a 3-bedroom house) are from $3,000 to $4,000 including septic tank, pump, pump chamber, and soil absorption system. The cost for a pump and pumping chamber to raise the effluent from a septic tank to a gravity fed distribution system is $400 to $600 installed.

OPERATION AND MAINTENANCE

Operating costs include pump maintenance and power along with septic tank pumping (every 3 to 5 years). These costs are in the range of $35 to $75 per year.

SOIL ABSORPTION SYSTEMS FOR HIGH GROUNDWATER OR SHALLOW SOILS FACT SHEET 2.4

October 1980

DESCRIPTION

Soil absorption systems that are designed to deal with high groundwater, shallow soil over rock, or impermeable layers include mounds, shallow placement, and artificial drainage.

Mound Systems consist of a septic tank, a pumping chamber, and pump that lifts the effluent to a raised absorption bed constructed over original ground surface. The raised bed is constructed of a suitable fill material, a gravel absorption area, pressurized distribution pipe, an impermeable soil cap, and topsoil for a vegetative cover.

Shallow Placement Systems are distribution networks that are installed in the upper soil horizons. Distribution pipe may be either gravity (4" perforated) or pressurized (1½" or 2" PVC). The actual depth of installation is dictated by site constraints. The shallowest systems are dug through fill to use original ground surface as the trench bottom. Varying amounts of fill are required for deeper systems.

Artificial Drainage makes sites usable for soil absorption systems that would otherwise be restricted by the presence of groundwater. Artificial drainage is used to lower perched or seasonal water tables to make sites amenable to soil absorption systems; however, regional (year round) groundwater levels should not be altered for this purpose (33). Plume recovery is an innovative method of artificial drainage use in moderately to highly permeable soils where effluent plumes from existing drainfields discharge nutrients to lakes or streams. A shallow recovery well or line of wells intercept plumes and return the mixed effluent and groundwater to lawns or gardens for irrigation.

COMMON MODIFICATIONS

Although Mound Systems are constructed usually using pressure distribution systems, a dosing siphon can be used if enough elevation head is available. Distribution layouts can take the form of trenches or beds (20).

Shallow Placement Systems can use different distribution methods such as trenches, serial distribution, or beds. Shallow systems can be combined with dosing and artificial drains. Depth and shape of trenches can be varied as suitable for individual sites. V-shaped trenches can be used to increase the sidewall surface area. Another technology adaptable to shallow placement is large diameter drainfield distribution piping (see Fact Sheet 2.2, GRAVITY DISTRIBUTION FOR SOIL ABSORPTION SYSTEMS).

Three commonly used Subsurface Drainage techniques include vertical drains, curtain drains and underdrains (see diagram). Curtain and vertical drains are used to lower perched water tables and underdrains are used to lower water tables in permeable soils. Construction is similar for these types of drains but spacing and depth requirements are dictated by the soil and groundwater characteristics of the site (71).

TECHNOLOGY STATUS

Elevated sand Mounds have been undergoing modifications and improvements since the 1940's when the first system was installed; they are now considered to be fully developed. Design and construction errors made in the past have been evaluated, and appropriate remedies have been included in current construction guidelines.

Shallow Placement Systems vary from conventional soil absorption systems primarily in depth. The technology for shallow systems can therefore be considered fully developed, although installation and evaluation in a range of settings have not been reported.

"The technology of using Drain Tile for agriculture and footing drainage is well developed, and can be adapted easily to on-site systems." (33). However, installation and evaluation in a range of settings have not been reported.

APPLICATIONS

Mounds are used to overcome site limitations such as high groundwater tables, confining soil or creviced bedrock layers, very permeable soils, and soils of low permeability.

SOIL ABSORPTION SYSTEMS FOR HIGH GROUNDWATER OR SHALLOW SOILS FACT SHEET 2.4

October 1980

Shallow Placement Systems are appropriate for soils with slowly permeable subsoil or parent material horizons or where maximum separation from groundwater is sought. Using upper soil layers rather than underlying soil layers takes advantage of more permeable soils, plant and soil fauna activity, greater evapotranspiration, and quicker drying (96, 2).

Lowering the groundwater by Artificial Drainage is useful for intercepting a laterally moving perched water table caused by a shallow impermeable layer (curtain drain), or caused by a thin, shallow, impermeable layer (vertical drain). A curtain drain intercepts perched groundwater through the use of a pipe located in gravel at or above the impermeable layer, whereas a vertical drain is constructed of gravel (without a pipe) that goes through the thin impermeable layer allowing the groundwater to enter permeable soil below the impermeable layer. Underdrains are used to lower seasonally high groundwater (71). Plume recovery is appropriate primarily for existing systems near lakes to mitigate growth of aquative plants.

LIMITATIONS

Elevated Sand Mounds should not be constructed in areas where groundwater level, bedrock, or other strata having a percolation rate slower than 120 minutes per inch are located within 24 inches of the natural grade. Also, mounds should not be located in floodplains, poorly drained, very poorly drained or stoney soils (as defined by the Soil Conservation Service) (32). Maximum slopes for mounds are 6% for soils with percolation rates between 30 to 120 minutes per inch, and 12% for percolation rates of 0 to 30 minutes per inch (106).

Shallow Placement Systems should not be used if the side wall of a deeper system would have a better infiltrative surface or if more permeable soils exist deeper. Freezing of shallow systems not continuously used should also be considered (96).

Artificial Drainage of groundwater is not practical in fine textured soils of slow permeability, areas where artesian aquifers exist, or in soils that are saturated for long periods of time, especially level sites. Local codes restricting artificial drainage should also be consulted (71, 106). The irrigation system for effluent plume recovery would be subject to freezing if a constant flow of water through the nozzles was not maintained.

TYPICAL EQUIPMENT/NO. OF MFRS.

All of the techniques discussed here are available through local septic tank installers.

PERFORMANCE

Properly designed and constructed Mound Systems that are operated according to guidelines effectively treat septic tank effluent. Good performance is contingent upon siting, adapting guidelines to specific site conditions, and care in construction (38, 66, 34, 105, 6).

Shallow Placement systems have not been evaluated to the extent that mound systems have. Based on available data however, these systems perform well (105).

Little performance data exists for using Artificial Drainage in conjunction with on-site wastewater disposal. Drainage systems associated with agriculture and foundations have performed well in the past. Under proper site conditions, the adaptation of this technology to soil absorption systems should not be difficult (33). Plume recovery has not been demonstrated at a scale appropriate to on-site systems.

DESIGN CONSIDERATIONS

Mounds should be located on a flat area or on the crest of a slope, and should be relatively long and narrow with a maximum width of 10 feet. A minimum of 12 inches of sand, followed by at least 9 inches of clean rock, should be placed on the roughened ground surface. Vegetative layers should be broken up without smearing or compacting the soil to provide good contact between the original soil and the mound materials. (The bottom area of the rock bed is based on an application rate of 1.20 GPD/SF.) The distribution pipe, perforated with 3/16 to 1/4 inch holes, should then be laid on the level rock with the holes down and covered with 2 more inches of rock. The rock bed should be covered with 6 to 8 inches of straw or marsh hay (uncompacted) and red rosin paper to prevent the cap and topsoil from filtering into the rock. Surface water should be diverted from the mound and a grass cover should be established as soon as possible (48, 33).

SOIL ABSORPTION SYSTEMS FOR HIGH GROUNDWATER OR SHALLOW SOILS FACT SHEET 2.4

October 1980

Design for Shallow Placement Systems should follow established criteria for conventional subsurface soil absorption except for depth and cover requirements. The depth requirement should be established by site-specific criteria, with the main objective being the maintenance of 3 to 4 feet of unsaturated soil for treatment above a limiting layer (rock or groundwater). The system should be covered with 12 to 18 inches of loam soil backfill and a grass cover established (59, 33).

Curtain Drains are installed up slope from the soil absorption system and should be as deep as possible while maintaining a gravity outfall to allow for groundwater mounding. Deeper drains intercept more groundwater that may interfere with the treatment system. A horizontal distance of 10 feet between the artificial drains and the drainfield trenches should be maintained. Treatment systems not requiring underdrains should be considered.

RELIABILITY

Mound Systems have proven to be reliable; the predominant problems are associated with the pumps rather than the mounds themselves. Proper maintenance of pumps is therefore considered essential for keeping the system reliable and the septic tank pumping (107).

Based on limited data, Shallow Placement Systems have been operating with little maintenance and with a low failure rate (105).

Virtually no maintenance is required for Artificial Drains after installation. However, due to the various types of groundwater drainage problems, no overall statement of the reliability of all drainage systems can be made. The effectiveness of small plume recovery systems has not been evaluated.

ENVIRONMENTAL IMPACT

Detrimental impacts include aesthetics due to size and shape (if not properly landscaped) and possible drainage pattern alterations (94). Mounds allow development on some sites that might otherwise not be suitable for on-site systems. Groundwater and public health impacts are similar to conventional on-site systems.

For appropriate sites, the impacts of Shallow Placement Systems are similar to those of conventional septic tank-soil absorption systems. Backfilling causes aesthetic problems along with possible alterations of drainage patterns.

Curtain Drains result in surface water contamination if drains are too close to the soil-absorption system. Vertical and underdrains cause groundwater contamination without adequate separation distances between the drains and the soil absorption system. Plume recovery has the potential to mitigate plant stimulation effects in lakes and streams by returning nutrients to ground surface where they can be assimilated by yard vegetation. Odors and pathogens cause problems from recovered plumes if the system is not properly designed.

APPLICATION TO CLUSTER SYSTEMS

Mounds are not applicable to cluster systems because cluster systems require more suitable soils than those required for mounds. For example, groundwater mounding would create a greater problem in soils that require sand mounds than in more suitable soils.

Shallow Placement Systems can be designed for cluster systems as well as on-site treatment. The distribution of effluent would fully utilize the soil absorption system if a pump and pressurized network were used.

When designing cluster systems, the best available soils for wastewater disposal should be used. Thus the use of artificial drainage would not be required. If, however, special circumstances warrant the use of artificial drainage, it could be used in conjunction with a cluster system.

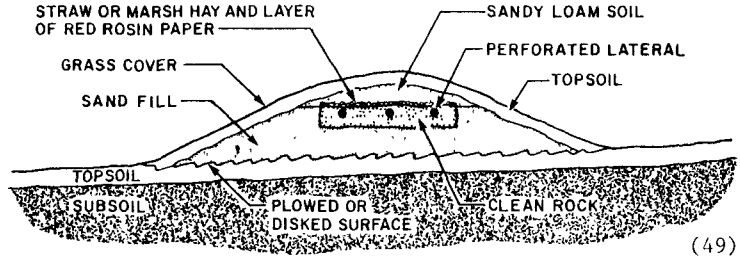
REFERENCES

104, 96, 39, 8, 32, 85, 38, 2, 66, 34, 24, 71, 49, 48, 59, 94, 105, 33, 6, 20, 106, 107.

DIAGRAM

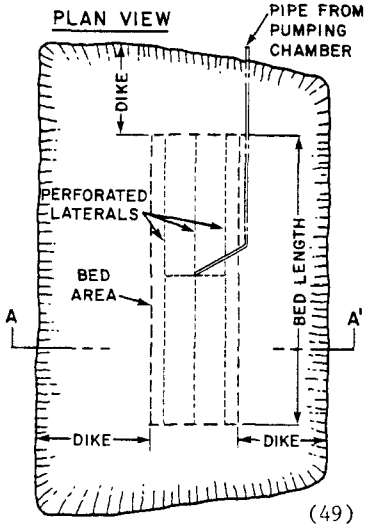
MOUND SYSTEM

CROSS SECTION A-A'



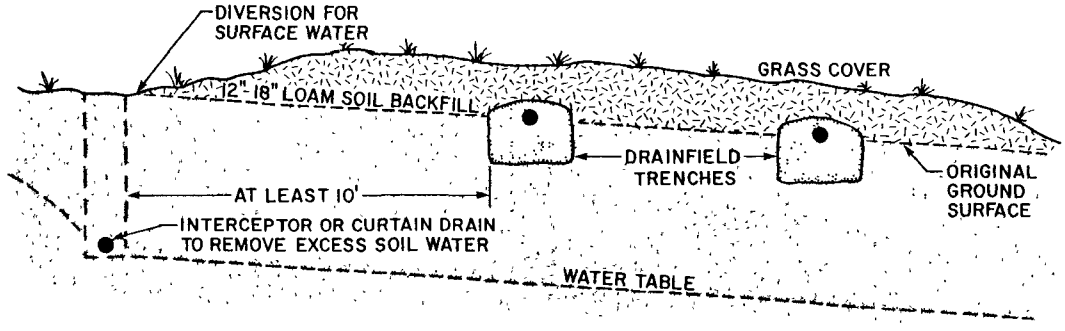
(49)

PLAN VIEW



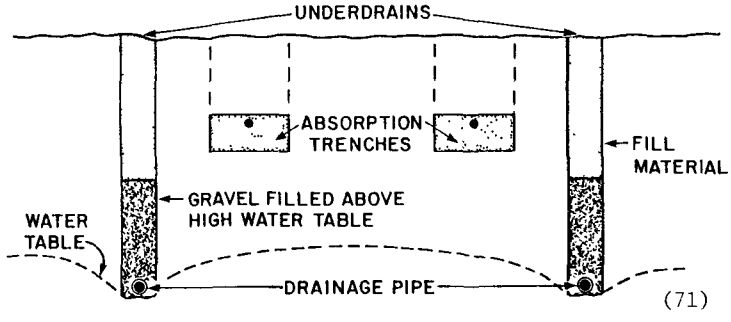
(49)

SOIL TREATMENT UNIT USING SHALLOW TRENCHES AND CURTAIN DRAIN



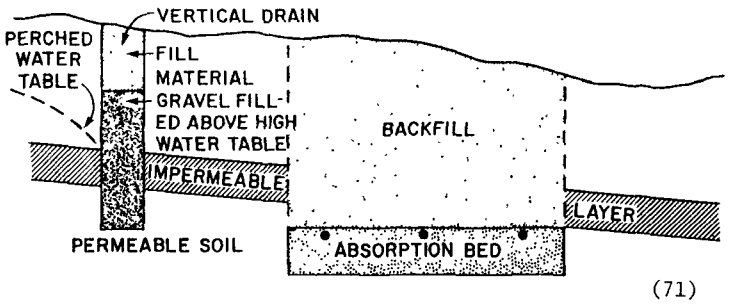
(49)

UNDERDRAINS USED TO LOWER WATER TABLE



(71)

VERTICAL DRAIN TO INTERCEPT LATERALLY MOVING PERCHED WATER CAUSED BY A SHALLOW, THIN, IMPERMEABLE LAYER



(71)

ENERGY NOTES

Mound systems require a small dosing pump that uses about 50 to 250 KWH/yr as would a shallow placement system that uses pressure distribution. Artificial drainage does not require energy if the outlet can be directed to the surface through gravity. Plume recovery would require the most energy because of the need for additional pumping.

COSTS

1980 Dollars; ENR Index = 3260

1) Mound Systems for individual home use, assuming a 3-bedroom house, range from $3,500 to $6,500 for initial construction costs. A big variable is the haul distance required for the fill material.

2) Shallow Placement Systems for a 3-bedroom house could be installed for $1,500 to $3,000 for a gravity-fed system.

3) Costs for installation of Artificial Drainage vary depending on depth required and the local cost of gravel. For effluent plume recovery, the costs of an irrigation system installed on a 1/4 acre lot with automatic controls range between $2,000 and $2,500. These figures do not include construction of a shallow well and required pump.

OPERATION AND MAINTENANCE

Mound systems require the use of a small pump in addition to the periodic pumping of the septic tank which costs between $35 and $75 annually.

Shallow placement gravity-fed systems cost the same for pumping (every 3-5 years) as do conventional systems, or $15 to $25 per year.

Artificial drainage does not require any operational costs providing a pump is not used. Estimated annual O & M costs for a plume recovery system are $100 for the irrigation system. Additional costs for operation of a small pump would be added to this.

SOIL ABSORPTION SYSTEMS FOR SLOWLY PERMEABLE SOILS FACT SHEET 2.5

October 1980

DESCRIPTION

Standard septic tank-soil absorption systems may not function adequately in slowly permeable soils. Alternatives, described below, include oversized absorption areas, seepage pits, electro-osmosis systems, mounds, and alternating drainfields.

1) Oversized Absorption Areas are constructed as are standard soil absorption systems except that the size of the drainfield is larger than that recommended for systems on more permeable soils. A larger area of slowly permeable soil is required to assimilate the effluent from a septic tank that could be treated by less area in soils with a higher permeability.

2) Seepage Pits (Dry Wells) are deep excavations used to dispose of septic tank effluent. Within the excavation, a porous walled chamber is installed and backfilled with clean rock. Septic tank effluent enters the chamber and seeps out slowly into the crushed rock, and then into the soil walls and bottom of the seepage pit.

3) Electro-Osmosis is an electric current passed through the soil to draw toward the cathode the layer of water normally bound in fine soil pores. By burying dissimilar materials on opposite sides of a drainfield, an electric potential develops in the soil water complex without the use of an outside power source. The electric potential causes the positively charged water to be repelled by the anodes and move to the cathodes located about 10 feet away from the absorption trench. The wastewater is thereby filtered and tends to dissipate through downward seepage and evapotranspiration (59, 71, 49).

4) Mounds are discussed in Fact Sheet 2.4, SOIL ABSORPTION SYSTEMS FOR HIGH GROUNDWATER OR SHALLOW SOILS.

5) Alternating Drainfields are included in Fact Sheet 2.2, GRAVITY DISTRIBUTION FOR SOIL ABSORPTION SYSTEMS.

COMMON MODIFICATIONS

1) Because of the slow permeability of soils requiring Oversized Absorption Areas, dosing and serial distribution should be considered along with alternating drainfields for use in combination with an oversized system. Additional techniques that could be combined with oversized absorption areas are pressure distribution, shallow placement, and trench liners. The use of trench liners with a regular soil absorption system is also a possibility for certain soil conditions (See Fact Sheet 2.6, SAND FILTERS).

2) Seepage Pits can be any size in diameter and depth providing they are structurally sound and can be constructed without seriously damaging the soil (71).

3) Two absorption areas can be constructed with an Electro-Osmosis System and connected to the septic tank while one field is in operation and the second is resting.

TECHNOLOGY STATUS

1) Oversized Absorption Areas and Seepage Pits are fully developed technologies.

2) Approximately 145 Electro-Osmosis Systems are currently operating in Minnesota with a zero-failure rate. This success is due in part to the extensive monitoring done in the first year of operation including eight inspections by the installers. (Personal communication with Frank Coolbroth 9/30/80).

APPLICATIONS

Oversized absorption areas and electro-osmosis systems are suited to soils with a percolation rate of 60 to 120 MPI. Electro-osmosis systems operate better in a soil mixture of clay and fine sand rather than very dense clays. Seepage pits can be used to overcome slowly permeable layers when suitable soils exist elsewhere in the soil profile. Percolation tests must be performed in all the various layers. A weighted average is computed to size the sidewall area; however, soil layers with percolation rates slower than 30 MPI are not included in the calculation.

SOIL ABSORPTION SYSTEMS FOR SLOWLY PERMEABLE SOILS FACT SHEET 2.5

October 1980

LIMITATIONS

1) Oversized Systems are acceptable if the percolation rate is between 60 and 120 minutes per inch. For a laboratory or field test of hydraulic conductivity, the acceptable range for an oversized system is up to 50% of the normal maximum for a given soil. Some regulations call for an aerobic treatment unit to precede oversized soil absorption systems (46).

2) A minimum of three feet is typically required between the bottom of a seepage pit and the water table or on impervious layer (49). Local codes should be checked before installing seepage pits.

3) Electro-Osmosis systems are not intended for sites where the seepage trenches would penetrate groundwater. Due to the complicated soil, water, and electrical interdependence, adequate tests must be performed in order to determine if the soil is suitable.

TYPICAL EQUIPMENT/NO. OF MFRS.

Oversized systems and seepage pits are locally available through septic tank installers. Electro-osmosis systems are patented and must be installed by contractors who are licensed for such work.

PERFORMANCE

1) In soils that have extremely slow permeabilities, Oversized Drainfields may not perform well. Adequate performance is limited to soils that are borderline cases between acceptable and unacceptable for conventional soil absorption systems. Performance may be improved on some sites by pressure distribution, shallow placement, alternating drainfields, etc.

2) The performance of a limited number of sewage Electro-Osmosis Systems for on-site wastewater treatment has been successful. The system has been used successfully for many years in the construction industry for draining and stabilizing slowly permeable soils during excavation.

DESIGN CONSIDERATIONS

1) Typical design criteria for Oversized Drainfields are similar to criteria for systems in suitable soils but are sized as follows: a) with permeability determined by a hydraulic conductivity test, the system is sized according to the determined liquid application rate based on a waste flow of 150 gallons/bedroom/day, or b) when percolation tests are used, sizing is based on a wastewater application rate of 0.2 gallons/square foot/day with a minimum of 750 square feet/bedroom (24).

2) Seepage Pits must maintain a minimum of three feet from the bottom of the excavation to the water table (or impervious layer) and the total excavation should not exceed ten feet. The minimum diameter for seepage pits is five feet. Design is based on a weighted average percolation rate which is used to size the infiltrative surface (or sidewall area of pit) (49, 71).

3) Because the Electro-Osmosis System is patented, the design, site testing, and construction are performed by those licensed.

RELIABILITY

Both oversized drainfields and seepage pits are reliable if properly designed and constructed on acceptable sites. Electro-osmosis however, has not been used with on-site disposal long enough to establish any reliability record.

ENVIRONMENTAL IMPACTS

Subsurface disposal systems provide sufficient filtration, adsorption, and microbial degradation of harmful constituents in household effluent. System failures, however, can occur and are related to site limitations or to improper installation, operation, or maintenance. Adverse environmental effects attributed to failures of subsurface disposal systems generally involve the contamination of ground or surface waters. This contamination then may result in public health

hazards if drinking water is affected or in accelerated eutrophication if excessive amounts of nutrients are delivered to lakes or streams.

Mitigation and avoidance of adverse environmental impacts depend on information regarding site-specific conditions that led to system failures in a community, and the use of this information to select repairs and replacement systems and to design new systems.

APPLICATION TO CLUSTER SYSTEMS

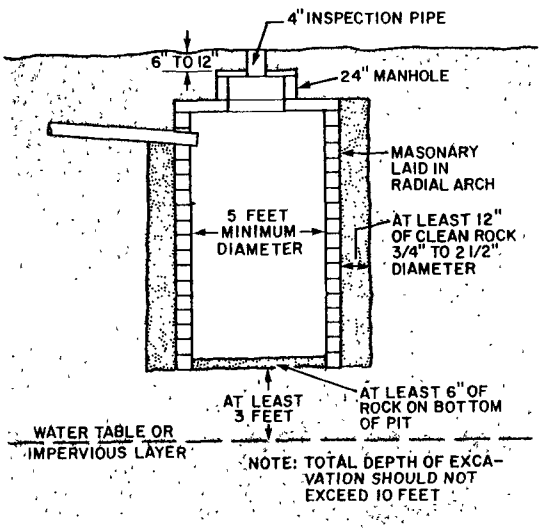
Of the three techniques discussed here, only oversized absorption fields have a possible use for cluster systems. Soils for a cluster system should not be marginal because of the amount of flow received. If the soil were basically satisfactory and an extra factor of safety were desired, then an oversized absorption system would have a place in cluster systems. Seepage pits and electro-osmosis systems are not appropriate for cluster systems.

REFERENCES

24, 49, 48, 59, 5, 76, 46, 71, 22, 72

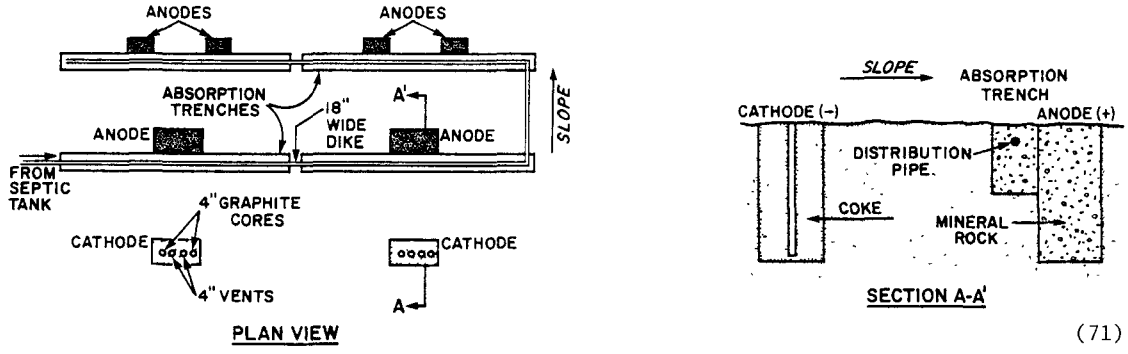
DIAGRAM

SEEPAGE PIT (DRY WELL)



(49)

TYPICAL ELECTRO-OSMOSIS SYSTEM



(71)

ENERGY NOTES

Energy sources are not required for oversized systems and seepage pits unless pressure distribution or an elevation difference between the septic tank and soil absorption systems requires the use of a pump. Electro-osmosis applied to on-site wastewater treatment requires that an electromotive potential be developed, but this potential can be established without the addition of energy by using the proper materials for constructing the anodes and cathodes.

COSTS

1980 dollars; ENR Index = 3260.

1) The following table compares the typical costs of an oversized drainfield system to a conventionally sized system for a 3-bedroom house with an assumed drainfield size.

SYSTEM	COST OF INSTALLED SEPTIC TANK ($)	COST PER FT^2 OF DRAINFIELD ($/FT^2)	SIZE OF DRAIN-FIELD (FT^2)	COST OF DRAIN-FIELD ($)	TOTAL SYSTEM COST ($)
Conventional	350-750	1.00-2.50	1000	1000-2500	1350-3250
Oversized	350-750	1.00-2.50	1250	1250-3125	1600-3875

2) Seepage Pit Systems constructed for a 3-bedroom house would cost $1,000 to $1,500.

3) Electro-Osmosis Systems have construction costs in the $5,000 to $6,500 range assuming a 3-bedroom house.

OPERATION AND MAINTENANCE

Operation costs for all three systems relate only to the required septic tank pumping. These costs range from $15 to $25 per year depending on the local pumping costs and the frequency with which the septic tanks are pumped.

SAND FILTERS FACT SHEET 2.6

October 1980

DESCRIPTION

Filtration is a process that has been used for many years to treat water and wastewater. The process involves passing the liquid to be treated over a bed of sand that is 24 to 40 inches deep and is supported by a gravel layer and is underdrained. The effluent from the sand bed is then disinfected (as required) and either discharged to surface waters or applied directly to a subsurface soil absorption system. A septic tank or some other form of solids separation is required for pretreatment prior to filtration. Filters are dosed intermittently and allowed to dry between applications. Mechanisms of purification include physical straining, chemical sorption, and biological decomposition.

COMMON MODIFICATIONS

The three basic variations of sand filters for on-site and small scale applications are 1) buried filters, 2) intermittent filters, and 3) recirculating sand filters. Buried filters are constructed underground are usually sized more conservatively than other filters, and are more easily accessible. Buried sand filters are constructed with walls to hold the units together or by using the pitwalls to contain the media. Recirculating filters are designed using a higher application rate and subsequently require less area than buried and intermittent filters. Recirculating filters have a chamber which mixes the septic tank (or other treatment process) effluent with the collected effluent from the filters, and then pumps the combination back over the filter.

A variation of the buried sand filter is lining trenches or beds with sand prior to the construction of a conventional soil absorption system. The effluent then flows through the sand directly into the soil instead of being collected by underdrains. Sand-lined beds and trenches can be used on sites with very well drained soils (with a percolation rate less than 6 MPI), or where the depth to an excessively permeable layer (e.g. rock fragments) is less than four feet from the surface. This disposal method is not allowed in floodplains or where soil mottling (or groundwater) appears at less than four feet below the bottom of the aggregate (108).

TECHNOLOGY STATUS

Filtration is a developed technology that is well suited to on-site applications. Although recirculating sand filters have been in use only since the late 1960's, they are also well developed technologically.

APPLICATIONS

Filtration is one treatment method that is advantageous for on-site disposal when the effluent required is of higher quality than that of septic tank effluent, such as when stream discharge is necessary because of site limitations. Some site conditions allow subsurface disposal only if a high degree of treatment precedes the disposal. This may result from high groundwater or from a creviced limestone layer that would not be acceptable for conventional soil absorption systems alone. Buried filters are applicable to flows from individual homes. Intermittent sand filters can be used for individual home application if shallow bedrock or high groundwater preclude the use of a buried system. Recirculating sand filters are intended to give off less odors than intermittent filters because the applied sewage is a mixture of septic tank effluent and filter effluent (37). Recirculating filters would therefore be applicable when odors could be a problem.

LIMITATIONS

The size of open sand filters is limited by land availability and, generally, the design flow is limited to 0.25 MGD. Odors may be a problem if septic tank effluent is applied directly to open sand filters. Application of effluent from an aerobic treatment process will reduce odors, extend filter runs, and produce a higher quality effluent than wastewater pretreated with septic tanks. Local availability of sand may restrict the use of filtration in some areas. Intermittent open sand filters may require an insulated cover in areas of prolonged freezing, which would also help control odors.

Buried sand filters have the problem of accessability if clogging should occur. Two buried filters could be used in an alternating system to prolong filter life, by allowing a resting cycle for each. Buried sand filters are normally used for very small flow applications, such as individual residences.

SAND FILTERS FACT SHEET 2.6

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TYPICAL EQUIPMENT/NO. OF MANUFACTURERS

Sand filters consist of distribution piping, sand, gravel, and underdrain piping, all of which are locally supplied.

PERFORMANCE

Sand filtration produces a high quality effluent with BOD_5 values usually less than 10 mg/l. Concentrations of suspended solids are usually less than 15 mg/l. Although nitrification is almost complete, the total nitrogen present is essentially unchanged. After maturation, sand filters remove only small amounts of phosphorus. Performance levels decrease with a reduction in ambient air temperature (96, 71).

DESIGN CONSIDERATIONS (71)

Pretreatment:	Minimum of Sedimentation (usually a septic tank)
Hydraulic Loading:	<1 GPD/SF (buried) 2-5 GPD/SF (intermittent) 3-5 GPD/SF (Recirculating based on forward flow)
Media:	Washed Durable Granular Material Less Than 1% Organic Matter by Weight
	Effective size: 0.50 - 1.0 mm (buried) 0.35 - 1.0 mm (intermittent) 0.30 - 1.5 mm (recirculating)
	Uniformity coefficient: <4.0 (<3.5 preferable) Depth: 24-36 inches
Underdrains:	Material: Open joint or perforated pipe Slope: 0.5 - 1 percent Bedding: Washed durable gravel or stone (1/4 to 1 1/2 inches) Venting: Upstream end
Distribution for Buried Filters:	Material: Open joint or perforated pipe Bedding: Washed durable gravel or stone (3/4 to 2 1/2 inches) Venting: Downstream end
Distribution for Intermittent & Recirculating:	Troughs on surface, splash plates at center or corners, or sprinklers
Dosing:	Flood filter to ∿ 2 inches Frequency: Greater than two times daily (buried and intermittent)
	Pump 5 - 10 min. per 30 min.; empty recirculation tank in less than 20 min. (recirculating)
Recirculation Chamber:	Volume equivalent to at least one day's raw wastewater flow.

For sand-lined trenches or beds, standard soil absorption system design criteria must be followed in addition to placing a layer of sand under the gravel aggregate.

RELIABILITY

Sand filters are quite reliable although they require a little more attention than septic tank soil absorption systems.

SAND FILTERS

FACT SHEET 2.6

October 1980

ENVIRONMENTAL IMPACT

After a sand filter treats the septic tank effluent, disposal is still a problem. Local codes should be consulted for any planned discharging system. Odors are yet another problem for intermittent sand filters receiving septic tank effluent, although covers can be installed as a mitigative measure.

APPLICATION TO CLUSTER SYSTEMS

Open sand filters are easily adaptable to multi-family units with design flows of less than 0.25 MGD. Multiple filters as opposed to single filters are suggested for larger systems. General design criteria for individual filters can be followed, but special emphasis must be given to distribution of effluent. Buried sand filters are not recommended for cluster systems.

REFERENCES

96, 71, 14, 86, 32, 8, 94, 67, 44, 108, 49, 37, 9, 36.

CHEMICALS REQUIRED

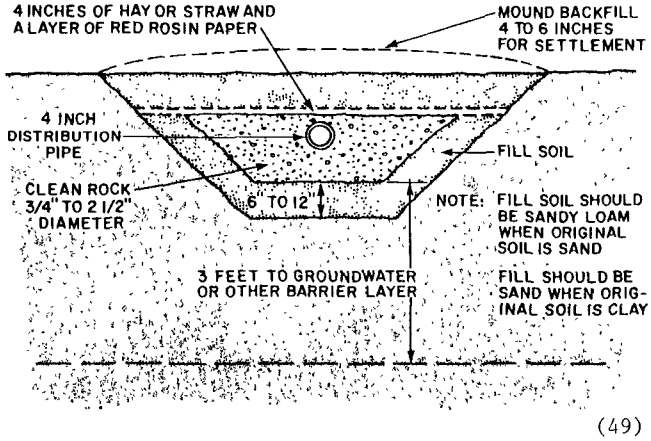
Local requirements for disinfection should be considered for discharging systems. These requirements may include the use of chlorine or iodine.

RESIDUALS GENERATED

In addition to the usual periodic pumping of septic tank solids, sand that is scraped off of the top of the filter must also be disposed of in an acceptable manner. Disposal can be by on-site burial or land filling at an approved site.

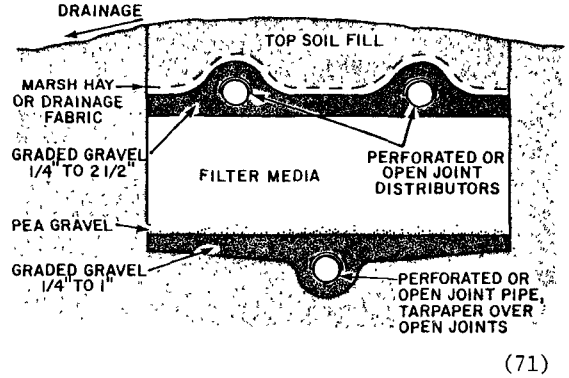
DIAGRAM

LINER SYSTEM FOR PROBLEM SOILS



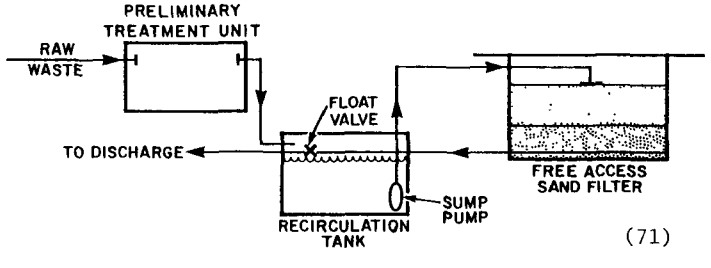
(49)

BURIED INTERMITTENT FILTER



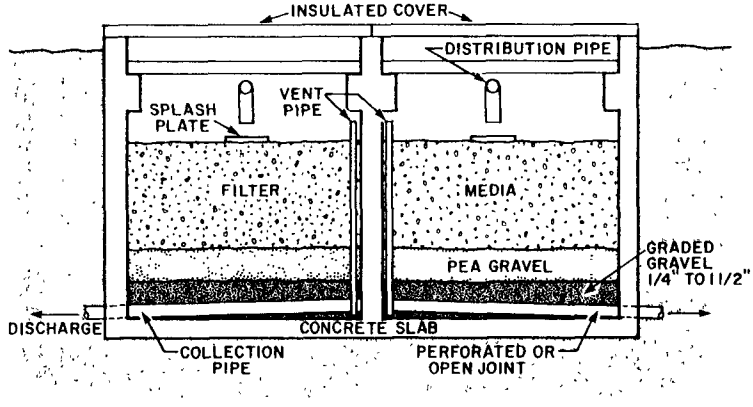
(71)

RECIRCULATING SAND FILTER SYSTEM



(71)

FREE ACCESS FILTER



(71)

ENERGY NOTES

The only power requirement is for the distribution system employed. Depending on the type of distribution system, power requirements will vary but should be less than 0.1 KWH/DAY. Recirculating may increase power by 50 to 250 KWH/yr.

COSTS

1980 dollars; ENR Index = 3260. The construction costs given below are for a complete system for a 3-bedroom house. These costs include the septic tank and associated piping, the sand filter along with underdrains, and all associated labor. Disinfection is not included.

Buried sand filters:	$2,000 to $4,000
Intermittent sand filters	$2,500 to $4,500
Recirculating sand filters	$2,000 to $4,000

The local availability of sand is a major variable in determining the system costs.

OPERATION AND MAINTENANCE

Pump items to be considered in operating a sand filter include the following:

Septic tank pumping
Pump operation and maintenance for recirculating and dosed filters
Restoration of filter capacity for recirculating and intermittent filters
Checking the filter surface periodically

The following are annual O & M costs for a typical system installed for a three-bedroom house:

Buried sand filters:	$ 15 - $ 25
Intermittent sand filters:	$150 - $250
Recirculating sand filters:	$ 70 - $100

LAGOONS FACT SHEET 2.7

October 1980

DESCRIPTION

Wastewater treatment lagoons or stabilization ponds are artificially created bodies of water that retain wastewater until natural biological processes reduce pollutant concentrations to an acceptable level. The treatment process is preferred by many small municipalities because of the low maintenance requirements. The three basic types of lagoons are aerobic, (often employing mechanical aeration), faculative, and anaerobic. For on-site use, facultative lagoons are the most widely used.

COMMON MODIFICATIONS

In addition to using lagoons as a treatment and discharge process, they are used as an evaporation and/ or infiltration system with no discharge. Controlled discharges, the releasing of wastewater to coincide with optimum stream characteristics, are another possibility. The size of the lagoon needs to be altered to accommodate the increased storage associated with a controlled discharge. Lagoons are often divided into cells for better performance.

TECHNOLOGY STATUS

The technology for lagoons is fully developed; however, the application to individual homes has not received much attention in the literature.

APPLICATIONS

Lagoons can be used for on-site treatment when subsurface disposal is not possible. Evaporative lagoons are useful in arid regions, and infiltrative lagoons are useful only where groundwater contamination is not a concern or is shown to be acceptable. Most lagoons require a substantial land area and liners may be necessary for certain applications.

LIMITATIONS

During times of high evaporation, lagoons may not maintain the depth required for proper operation without the addition of water. Low liquid levels might cause unwanted plant growth within the pond area. In cold climates an ice cover may reduce the treatment efficiency and possibly cause odors while thawing. Minimum distances to property lines and residences limit the use of lagoons to sites with sufficient area for buffers.

TYPICAL EQUIPMENT/NO. OF MFRS.

Lining systems/6 Pipe and excavation is locally supplied.

PERFORMANCE

Performance of facultative lagoons varies with climatic changes. Algae production during warm weather contributes to high suspended solids levels. Reduced biological activity due to low temperatures results in higher BOD values. Removal rates for BOD_5 usually range from 75 to 90 percent and effluent SS values of 20 to 150 mg/l are typical. Phosphorus and nitrogen removals are usually less than 50%.

DESIGN CONSIDERATIONS

Typical design standards include minimum separation distances of 50 feet from property lines as measured from the adjoining shoreline, 200 feet from the nearest residence, and 50 feet from a potable water supply, a stream, lake, or other water body. Standards call for 440 square feet of lagoon surface per bedroom with a minimum of 1,500 square feet. The hydraulic retention time should be at least 60 days. Longer times should be used when severe winters preclude discharge or for controlled discharges. The minimum freeboard should be 2 feet with an operating depth of 3 to 5 feet (32). Organic loading for winter conditions as well as for summer is typically 10 lbs/acre/day. The range is 15-65 lbs/acre/day (45).

RELIABILITY

Particularly for use with domestic wastes, lagoons are very reliable as they will continue to operate with little or no operator assistance.

ENVIRONMENTAL IMPACT

The environmental impact of lagoons is limited to possible odors during mixing caused by wind and/or seasonal temperature changes and seepage into permeable soils. Proper siting and/or lining can circumvent these impacts.

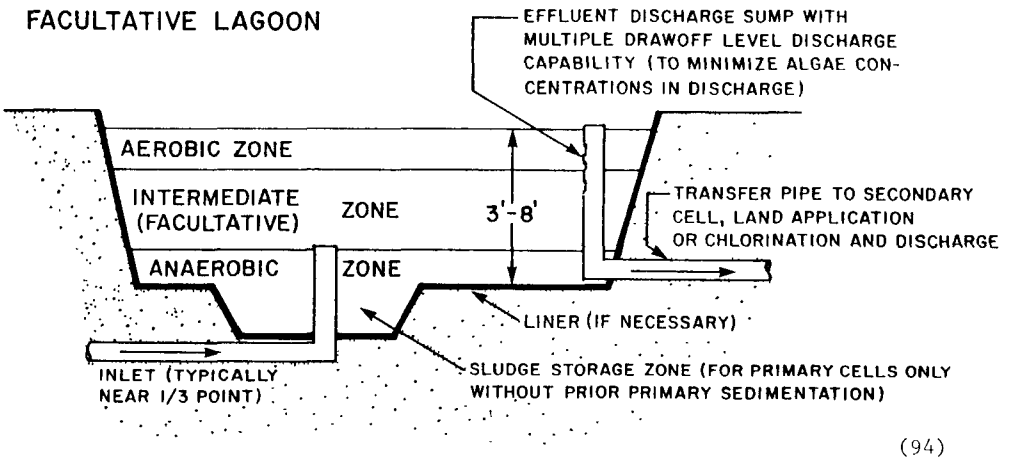
APPLICATION TO CLUSTER SYSTEMS

Lagoons are a candidate for multi-home or cluster system use where variable suspended solids and BOD are acceptable in the effluent. Low operation and maintenance along with low construction costs are major advantages for these systems.

REFERENCES

94, 51, 32, 29, 82, 92, 88, 45, 68.

DIAGRAM



(94)

ENERGY NOTES

Lagoons do not use any energy unless a) mechanical aerators are used, b) an elevation difference requires pumping to the lagoon, or c) constant feed chlorinators or ultraviolet lights are used for disinfection.

COSTS

Costs are highly dependent on local excavation costs. Other costs to be considered include the following:

1) Excavation
2) Final Grading
3) Seeding
4) Fencing
5) Piping
6) Access Road (if necessary)
7) Land
8) Inlet and Outlet Structures
9) Disinfection (if necessary)
10) Liners

For a community of 100 people, the initial cost for a stabilization lagoon is approximately $404.00 per person. For a community of 200 people, these initial costs are approximately $267 per person. Operation and maintenance costs for a 100 and 200 person community are $6.00 per person and $4.30 per person respectively (88).

OPERATION AND MAINTENANCE

Operation and maintenance consists of mowing the grass around the lagoon, adding water when necessary to maintain adequate bacterial activity and prevent mosquito breeding, and taking and analyzing samples when necessary to comply with surface discharge requirements.

FIXED FILM TREATMENT FACT SHEET 2.8

October 1980

DESCRIPTION

These biological treatment processes operate by allowing wastewater to contact a media of large surface area. Microorganisms attach to the media and metabolize the pollutants in the wastewater. The process usually works aerobically with air introduced either mechanically (e.g., diffusers) or by natural ventilation. There are many variations of this process, the two most common being: 1) rotating biological contactors, and 2) the trickling filter or packed tower.

Rotating Biological Contactors (RBCs) - are rotating discs usually made of polyethylene or styrofoam that are mounted on a horizontal shaft. The discs are partially submerged in the wastewater (about 40%) and as they rotate, aerobic microorganisms attach, subsequently treating the wastewater.

Trickling Filters - or packed towers, consist of a tank filled with rock, synthetic, or redwood media. The wastewater is distributed over the media where the fixed film of microorganisms acts in much the same way as the film on the RBCs. Aeration can be provided by natural ventilation or by diffusers.

A Submerged Media System - is a variation of the trickling filter in which the media is completely submerged in wastewater; the system can be aerobic or anaerobic (and upflow or downflow).

COMMON MODIFICATIONS

Fixed film processes such as RBC's can be used in series for additional treatment or in conjunction with other treatment processes, such as clarification and filtration to provide effluent acceptable for discharge. Recycling is another common modification as is adding air to the tanks (94).

TECHNOLOGY STATUS

Trickling filters have been in widespread use for municipal wastewater treatment plants for nearly 50 years. RBCs have been used in the U.S. since 1969. The application of fixed film reactors to on-site systems, however, is relatively new.

APPLICATIONS

Fixed film reactors are used (usually with disinfection) in applications where the effluent is discharged into surface waters. These units can be used for flows from individual houses and, by increasing the size or number of units, for larger systems such as multi-family and municipal installations.

LIMITATIONS

Trickling filters are temperature sensitive and therefore need to be insulated against cold weather. RBCs must be covered to prevent excessive algal growth from building up on the discs. For on-site applications, consideration must be given to pretreatment and ultimate disposal of wastewater. Pretreatment can usually be accomplished with a septic tank, while wastewater flowing from the fixed film reactor must undergo a solids separation process and disinfection prior to stream discharge.

TYPICAL EQUIPMENT/NO. OF MFRS.

Rotating Disc Systems/3 (only 1 is currently marketed in North America).

PERFORMANCE

Very little information exists for individual fixed film systems. Reported BOD_5 and SS removal rates are 75 to 95% depending on the particular process. The effluent is normally nitrified for aerobic processes while little phosphorus reduction takes place. Indicator organisms are normally reduced by 90-99 percent (8).

DESIGN CONSIDERATIONS

A sedimentation pretreatment process is required and can usually be accomplished by using a septic tank. Hydraulic loading varies between 25 and 100 gpd/ft^2 for upflow/downflow filters and between 0.75 to 1.0 gpd/ft^2 for RBC units. Although recycling is often thought to complicate the on-site system, hydraulic load calculations must include recycled flow as well as forward flow to the filters (71).

Organic loading can be considered from 5 to 20 lbs. BOD_5/1000 ft^3 for upflow/downflow filters and from 1 to 1.5 lbs BOD_5/1000 ft^3 for RBCs. Again, any loading from recycling is added to the loading from the forward flow to compute the entire organic loading (71). Final settling after the fixed film treatment is required. Disinfection is usually required for stream discharge and can be added after final clarification.

RELIABILITY

Biological fixed film systems are not usually as labor intensive as extended aeration systems but do require more attention than conventional ST-SAS. Because of their mechanical nature and dependence on electricity, these systems are less reliable than gravity-fed soil dependent systems.

ENVIRONMENTAL IMPACT

Odor impacts are possible if waste should turn septic or if electrical service is interrupted for an extended time period. Permits must usually be obtained for surface water discharges.

APPLICATION TO CLUSTER SYSTEMS

These systems are conducive to cluster systems because they can handle wastewater from several residences without much more operation and maintenance than that required for individual systems.

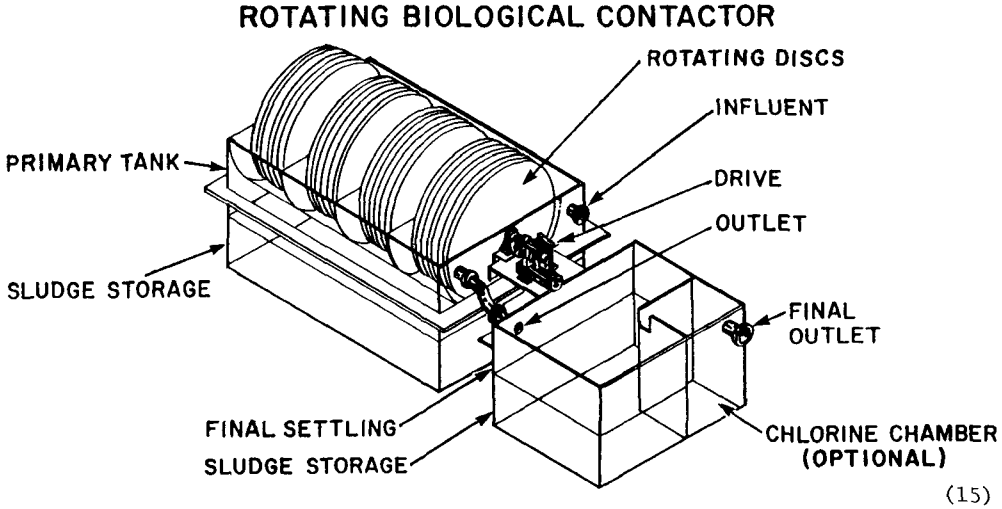
REFERENCES

59, 24, 71, 70, 92, 94, 8, 102, 103, 15, 30.

RESIDUALS GENERATED

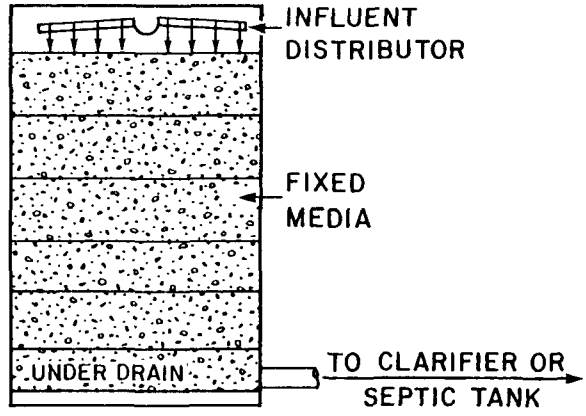
During the treatment process, the biological buildup on the media becomes too thick and the forces from the applied wastewater slough off the biomass. A new layer of growth begins; however, the solids sloughed off must be removed periodically from the reactor to prevent denitrification and rising sludge. These extra solids are normally wasted to the septic tank which may result in increasing the septage pumping interval to 1.5 to 2 year cycles (8, 71).

DIAGRAM



(15)

TRICKLING FILTER



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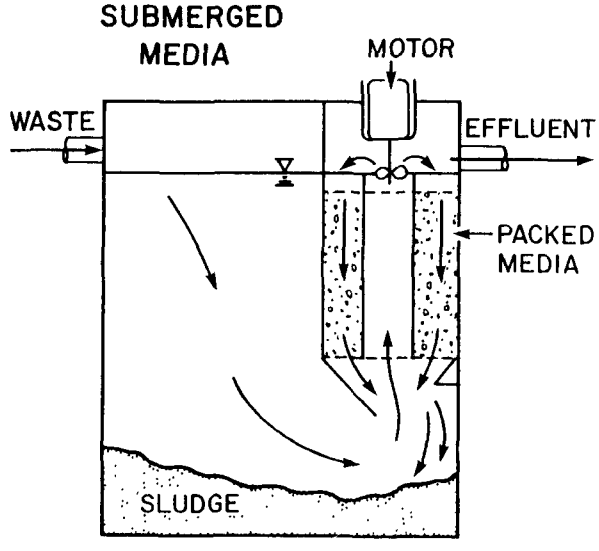


(71)

ENERGY NOTES

Depending on the process used, power requirements vary but usually range from 1 to 4 KWH/day. Additionally, an increase in septage pumping requires more energy. Although very limited data is available, operation and maintenance requirements of 8 to 12 person hours per year plus analytical services are required to ensure proper performance (71, 8).

COSTS

1980 dollar; ENR Index = 3260. Capital costs for a fixed film reactor for a typical individual 3-bedroom home application range between $1,750 and $3,000. O & M costs from $45 to $70 per year for power and septic tank pumping can be expected.

HOLDING TANKS

FACT SHEET 2.9

October 1980

DESCRIPTION

Holding tanks, which are waterproof containers that hold wastewater, provide no treatment except for some anaerobic digestion of settled solids. The wastewater must be removed from the tanks at intervals depending on size of tank.

COMMON MODIFICATIONS

Holding tanks can be used in conjunction with on-site soil absorption systems to accept wastewater when the soil absorption system cannot handle all the load due to high use or a seasonally high watertable.

TECHNOLOGY STATUS

Holding tanks of concrete, fiberglass, steel and cross-linked polyethylene are fully developed and commercially available. Tanks designed to be septic tanks are normally used since they are locally available. Tanks with hopper bottoms and built-in riser pipes with quick-connect fittings above ground avoid the disruption and spillage problems of common holding tanks. Probably because of low demand, such special purpose tanks are in common use.

APPLICATIONS

Individual buildings for which no on-site treatment options are available and for which off-site technologies are not economically competitive, holding tanks are viable options. By accepting toilet waste only, holding tanks reduce the nutrient load to on-site treatment systems in areas where groundwater or surface water pollution is possible.

LIMITATIONS

Holding tanks are considered temporary measures where on-site disposal problems exist but could also be considered permanent systems for summer or other short-term or low intensity uses. Permanent year-round applications are infrequent because pumping costs are high and management mechanisms are not available to ensure proper frequency of pumping and disposal of wastewater (32).

TYPICAL EQUIPMENT/NO. OF MFRS.

Locally supplied/Many mfrs. available.

PERFORMANCE

Concrete and cinder block holding tanks are not water-tight unless lined or epoxy coated. Fiberglass or cross-linked polyethylene tanks provide a more water-tight tank that is easier to clean when pumped out. These tanks are also durable and have a long service life.

DESIGN CONSIDERATIONS

In designing a system of holding tanks, one of the prime considerations is the disposal of the wastes after pumping. Because the tanks provide little or no treatment and the waste is in an anoxic state, special treatment provisions must be made. This type of waste could upset the biological balance of small wastewater treatment plants. Other considerations are the size of the proposed tanks based on the frequency of pumping desired, expected flows, and capacity of hauling vehicle. A meter should be installed with holding tanks to indicate the remaining capacity. High water alarms should also be a part of all holding tank installations. Holding tanks should be accessible in all types of weather for pumping trucks and risers with quick-connect fittings should allow for easier access when pumping. During construction in high groundwater areas, holding tanks should be anchored securely to avoid "floating up" of a partially full tank.

RELIABILITY

With a proper management system, including regular pumping, holding tanks are reliable. A meter, indicating capacity remaining in the holding tank, increases the reliability as does a high water alarm.

ENVIRONMENTAL IMPACT

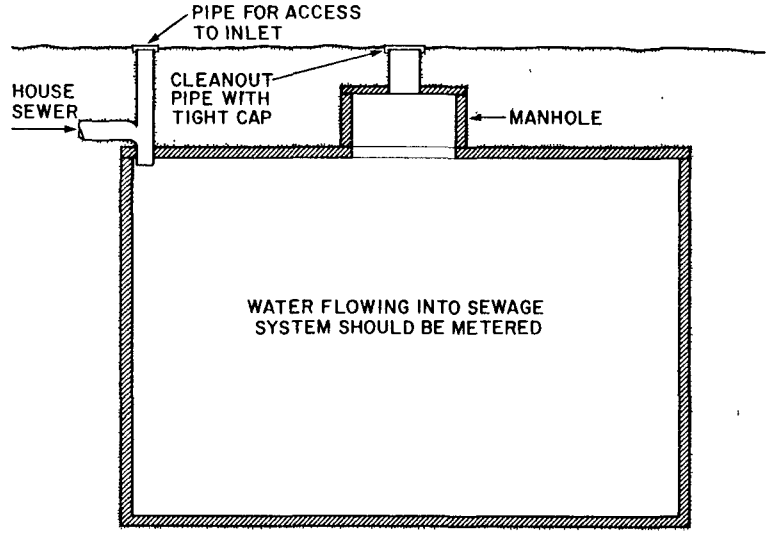
The high cost of pumping holding tanks may offer an incentive for illegal disposal. Upsets of wastewater treatment plants receiving holding tank wastes may result in decreased water quality in the receiving stream.

REFERENCES

76, 32, 85, 49.

DIAGRAM

HOLDING TANK



(49)

ENERGY NOTES

These systems are energy intensive from the standpoint of the requirement for pumping. After pumping and hauling, the waste must still undergo treatment which requires additional energy input.

COSTS

1980 dollars; ENR Index = 3260. Installing a holding tank is similar to installing a septic tank without the drainfield. Costs for a 1,000-gallon septic tank range from $350 to $750 for the total system installation including anchoring and alarm systems.

OPERATION AND MAINTENANCE

Annual pumping costs depend on the size of the tank and the daily flow rate. For example, a family of 3 producing 50 gpcd with a 1,000-gallon holding tank would require pumping every 7 days. Assuming a $40 charge per pump out, annual costs would exceed $2,000. If an air assisted toilet (see FACT SHEET 1.4) using 0.5 gallons per flush were to be discharged into a 1,000-gallon holding tank with separate greywater treatment provided, pumping would be required only 3 times per year. The annual costs for pumping would therefore be reduced to $120, which would not include any costs associated with greywater disposal.

SEPTAGE DISPOSAL BY LAND APPLICATION FACT SHEET 3.1

October 1980

DESCRIPTION

Application of septage to the land is by far the most commonly used means of septage disposal. Of the total septage generated, it is estimated that 60 to 90% of the total is disposed on land. Septage disposal on land includes surface application, sub-surface application, sanitary landfills, and disposal trenches. Septage pumped from septic tanks is directly disposed on land or treated prior to land disposal.

Surface application of septage is quite common in remote areas where human contact is minimal and land is readily available. Septage is spread over grassed areas or plowed soil by a pumping truck, by special spreading vehicles, or by a farm tank wagon usually pulled by a farm tractor. The use of separate collection and spreading vehicles depends on the size of the operation and the condition of the spreading site. Land spreading can also be achieved by ridge and furrow, and spray irrigation methods. However, the use of these methods with septage has not been reported.

Land spreading in most areas cannot be practiced year-round. In colder climates, land application should be limited to unfrozen surfaces to prevent runoff during thaws. During wet periods the saturated soils generally restrict field access by disposal equipment and increase runoff of contaminated water. Thus, to allow year-round operation of septage collection and disposal, septage storage facilities generally are included with land spreading. Storage facilities also allow monitoring of septage characteristics before its application on land.

Subsurface application of septage is usually accomplished by one of the following methods.

- Plow-Furrow-Cover (PFC) - septage is applied to the plowed furrow 6 to 8 inches below the original surface and is immediately covered by a following plow.
- Sub-Sod-Injection (SSI) - septage is injected into openings made in the ground below a sod cover crop. The septage is injected in a wide band or several narrow bands 6 to 8 inches beneath the sod surface.
- Terreator - is a patented device which injects septage below the surface of the ground into a 3-3/4 inch diameter mole-type hole made by an oscillating chisel point device.

Subsurface application offers better odor and pest control than surface spreading and reduces the likelihood of people, pets, and pests coming into contact with septage. Immediate incorporation of septage in soil, however, minimizes nitrogen volatilization, thereby increasing nitrogen loading in the soil. While this is desirable for fertilization purposes, it is not desirable for minimizing accumulation of nitrates in groundwater.

In a sanitary landfill septage disposal is mixed with solid waste and covered with soil. Landfilling municipal sewage sludge is used extensively and is limited to dewatered sludge to minimize water drainage and leaching. While septage disposal in a sanitary landfill is practiced in some states, many landfill operators are reluctant to accept septage because of potential leachate problems.

Land application of septage is sometimes accomplished in disposal trenches. A series of trenches is dug, and small amounts of septage are pumped into the trenches in lifts and allowed to dry out. When the trenches are full, they are either covered and abandoned, or dewatered solids are removed and disposed of in a landfill.

TECHNOLOGY STATUS

Surface application is fully developed technologically. Studies at the University of Connecticut (89) demonstrate the feasibility of soil injection as an acceptable method for subsurface application of septage.

A survey of sanitary landfill operators indicates that approximately one-third of the landfills surveyed accept both sewage and septage disposal (91). However, septage is rated 5 and sewage sludge 2 (median values) on a 0-10 scale of increasing potential hazard. A few states have laws completely restricting the disposal of septage in landfills. Others do not recommend landfilling because moisture control is too difficult to maintain (7).

Trench disposal is a common technique for handling septage in rural areas, however, in many cases this method is considered a temporary solution (7).

APPLICATIONS

Surface application of septage is considered a disposal and utilization method. Since septage can be considered a form of fertilizer because of its nutrient value, it can be applied at sites designated solely for disposal purposes or on agricultural land. If the land is to be put into crop production, an intermediate period of grass cover for at least one year is desirable. Septage can be applied on land directly after pumping it out of septic tanks or after stabilization. Septage that has not been stabilized should not be spread on crops in the same growing season they will be grazed on or harvested for domestic livestock. Additionally, septage should not be applied to land that is to be immediately used for growing crops for human consumption.

Subsurface application is used when odor and pest control requirements prevent the use of surface application. The proximity of the disposal site to inhabited areas may dictate the control measures.

Septage pumped out of septic tanks could be disposed of in a sanitary landfill provided that the ratio of liquid (septage) to solid waste is such that low moisture content of the mixture is achieved prior to burial. Dewatered septage could also be disposed in a landfill.

Trench disposal is used to dispose of raw septage and requires less land than other land application methods.

LIMITATIONS

Surface application rates are limited by nitrogen and trace metal contents of the septage. Nitrogen application in excess of crop requirements could leach downward through soil and contaminate groundwater. Land with slopes greater than 8% and low lying fields are not suitable for septage application.

Subsurface application rates are also limited by nitrogen and trace metals. Soil incorporation equipment does not operate well on frozen ground. Application when the ground is saturated or snow-covered also is not acceptable.

Landfills are generally not acceptable for septage disposal in areas where rainfall exceeds 35 inches/year. For septage disposal to be acceptable in such areas, leachate control facilities must be present in natural form or constructed.

Sites for trench disposal may have limited lifetime due to groundwater contamination or exhaustion of available land at the site.

TYPICAL EQUIPMENT

Surface application and sanitary landfills: The most common equipment is the septage pumper truck which combines the pumping/hauling and spreading operations. The spreading operation can be made more effective by fitting a splash plate to the truck discharge line which would increase the area of application. For periods of wet weather, a truck can be equipped with flotation tires to prevent slippage. For a large centralized disposal area with septage holding facilities, a tank wagon pulled by a tractor can be used.

Subsurface application: The PFC technique involves a 16-inch single bottom mold-board plow, a furrow wheel, and a 16-inch coulter. The SSI device consists of two plows assembled together to provide a 24-inch-wide opening, 6 to 8 inches below the surface. The Terreator is patented equipment that uses an oscillating chisel point device to make 3-3/4 inch diameter mole-type holes (99).

The incorporation of septage can be achieved by using a farm tractor and tank trailer with attached subsurface equipment, by using commercially available tank trucks with subsurface injection equipment, or by using tractor-mounted subsurface injection equipment in conjunction with a central holding facility and flexible "umbilical cord" (7). A year-round operation also will require holding facilities to store the septage during inclement weather.

Diposal trenches: Backhoe and hauling tank truck.

PERFORMANCE

Surface application of septage is considered a zero discharge, resource recovery system. Septage acts as a soil conditioner and as a fertilizer. However, improper management leads to problems such as contamination of the groundwater, heavy metal accumulation in the soil, and surface water pollution. A three-year study to evaluate the effects of land spreading of domestic septage on groundwater nitrate concentrations and microbial penetration concluded that no significant increase in well water nitrates was observed and no evidence of fecal coliform penetration was found (4). The septage application rate was from 36,000 gallons to 198,000 gallons per year per acre applied intermittently. The equivalent nitrogen loading of the septage was 50 and 200 pounds of nitrogen per acre per year. The soil on the application site was sand with a sand to loamy sand surface layer with high permeability, and thus, represented maximum potential for pollution of the groundwater.

Studies at the University of Connecticut (89) have found that the PFC method is preferable to the Terreator and SSI for subsurface application. With the SSI method, a sod cover is required and the frequency of equipment access to the field site is less because of field techniques used. The Terreator is still in developmental stages, and corrective work will be required before it can be evaluated.

DESIGN CONSIDERATIONS

The principal design factors for surface application are application rate, soil conditions, site location, and climate. Application rates determine the area needed for the volume of waste to be handled. Application rates for septage can be based on nitrogen loading or on heavy metal loading. For domestic septage, application rates based on nitrogen loading appear to be more restrictive than rates based on heavy metal loading (23). The application rates recommended by the University of Maine Life Science and Agricultural Experiment Station vary from 62,500 gal/acre/year for well drained soil to 37,000 gal/acre/year for moderately well drained soils. These application rates would result in nitrogen loading of about 500 lb N/acre/year in well drained soil and 300 lb N/acre/year in moderately well drained soil (23). These nitrogen loading rates are based on the annual harvest of a nitrogen demanding crop. On unharvested spreading sites it is possible that over a period of time, the supply of available nitrogen will exceed crop uptake of nitrogen. This would result in an increase in the amount of nitrates leaching into the groundwater. The potential contamination of groundwater on such sites could be avoided by rotating the unharvested sites so that a site with an excess of inorganic nitrogen could be used for production of a nitrogen demanding crop after an appropriate resting period.

Site location is important for minimizing odors, runoff, and groundwater contamination. The site should be in an isolated location and should be at least 300 feet away from wells, springs, intermittent streams, streams, rivers, ponds, lakes, marine waters, and floodplains. The slope of the land selected for the disposal site should be less than 8 percent. Holding facilities or alternative disposal plans may be required where climatic conditions preclude land spreading year-round.

In addition to the above design considerations, equipment selection and sizing must be included for subsurface application of septage. Additionally, immediate incorporation of septage minimizes nitrogen loss due to volatilization, and thus, net nitrogen loadings to the soil generally will be higher for subsurface disposal than surface application. To reduce buildup of nitrogen in soil and potential groundwater contamination, a nitrogen demanding crop should be grown and harvested on the site.

The location, design, and operation of sanitary landfills are governed by the criteria established by states. For landfills receiving septage, some standard operating criterias are as follows.

- New Jersey recommends an application ratio of 10 gallons of septage to each cubic yard of solid wastes. California regulations allow 25-40 gallons/cubic yard of solid wastes (7).

- The New England Interstate Water Pollution Control Commission (NEIWPCC) recommends that the landfill include facilities for the collection and treatment of leachate.
- The NEIWPCC recommends a 6 inch earth cover be applied daily to each area that was closed with septage and 2 feet final cover placed within a week after the placement of the final lift (23).
- The groundwater should be monitored on a regular basis to determine any change in groundwater quality due to septage loading.

Disposal trenches are three to six feet deep and two to three feet wide, with lengths varying with site location. Sufficient space is left between trenches to allow movement of heavy equipment. Septage should be added in lifts six to eight inches deep, depending on soil condition. After the trench is full with dried solids, the trench should be covered with at least two feet of soil.

The site selection should include consideration of soil type and characteristics, groundwater depth, acquifer size and use, proximity to dwellings, and proximity to septage sources. A minimum of four feet between seasonal high groundwater and trench bottom is recommended. A monitoring strategy to monitor groundwater contamination should also be included.

RELIABILITY

Properly managed land application of septage is a reliable means of disposal.

ENVIRONMENTAL IMPACT

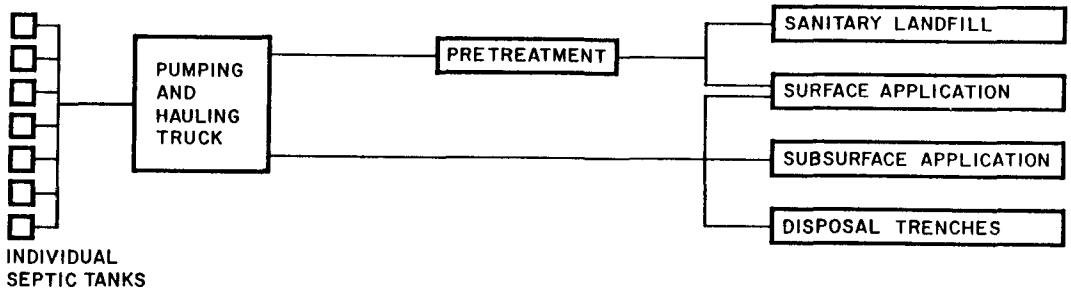
Potential detrimental environmental impacts include contamination of soil, water (surface and ground) air, vegetation, animal life and, ultimately, humans. Surface application presents more problems for surface water and air contamination than other methods but can be controlled by planting and harvesting a crop high in nitrogen uptake and by preventing application during wet periods when surface runoff would be greatest. Subsurface applications, landfills, and trenches pose a bigger threat to groundwater supplies. Proper site selection can mitigate against such impacts.

REFERENCES

91, 23, 99, 11, 85, 7, 4, 86, 89, 94.

DIAGRAM

LAND APPLICATION OPTIONS FOR SEPTAGE DISPOSAL



COSTS

Surface application: $1.50-$20.00 per 1,000 gallons of septage (not including transportation to site).

Subsurface application: $25,000-$42.50 per 100 gallons of septage (transportation not included).

Sanitary landfill: $5.00-$10.00 per 100 gallons of septage (accurate costs not available). Septage often not accepted at landfills due to low solids content.

Disposal trenches: $25.00-$36.50 per 1,000 gallons of septage (transportation not included. Site life assumed to be 10 years. Facility capacity assumed to vary from 100,000 gallons/yr to 250,000 gallons/yr).

1980 dollars; ENR = 3260 reference (94).

SEPTAGE TREATMENT AND DISPOSAL AT SEPARATE SEPTAGE FACILITIES FACT SHEET 3.2

October 1980

DESCRIPTION

Septage treatment facilities generally are regional facilities in areas where high densities of septic tank systems exist. Only a small portion (approximately 1 percent) of the total septage generated nationally is treated in separate septage treatment facilities. The process types currently in operation include lagoons, composting, and chemical treatment.

Lagoons are among the least expensive methods available for disposing of septic tank wastes, and are widely used throughout the country. The lagoon disposal system normally consists of two, three-feet deep earthen basins arranged in series. The first lagoon serves as the primary settling basin in which the majority of suspended solids accumulate, settle, and undergo further anaerobic digestion. The adjacent lagoon serves as the seepage lagoon from which supernatant from the settling lagoon is discharged. The supernatant infiltrates the soil where it undergoes further purification. An alternative lagoon system consists of a settling lagoon followed by percolation beds. These beds consist of a layer of sand over soils with good percolation characteristics. The beds have a berm running along the perimeter to retain the liquid. Once the lagoon is full, the liquid is allowed to percolate and evaporate until solids are dry enough for removal and ultimate disposal. Alternatively, the lagoon may be covered with topsoil and a new lagoon opened at another site. The lagoons will gradually lose their capacity to infiltrate water due to clogging of the soil. Returning a clogged lagoon to operation may require cleaning, scrapping, and regrading.

Septage can be converted to a compost product, which is a well stabilized, odorless material with a much wider range of uses than raw septage. The conversion process involves mixing septage with a suitable bulking agent (wood chips and/or sawdust), and circulating air through the mixture to insure proper conditions for pathogen die-off, decomposition of organics, and odor reduction. The process is carried out in three stages that include digestion, drying and screening, and curing. In the digestion stage, the compost pile is aerated and undergoes decomposition by thermophilic organisms. The digestion stage is completed in about 21 days at which time the compost is either dried or cured depending on weather conditions. The drying and screening stage involves spreading the compost pile to a depth of 12 inches for drying to 40 to 45 percent moisture content. The compost is then separated from the wood chips by screening through a rotary screen and the wood chips are recycled. Curing is the final stage in which the compost is stored in piles for about 30 days to insure that no offensive odors remain and to complete stabilization.

Chemical treatment of septage is used to improve solid-liquid phase separation, improve dewaterability, reduce odors, or kill pathogens. Chemical treatment processes include:

- Chemical coagulation
- A two-stage chemical process involving acidification followed by lime treatment
- Rapid chemical oxidation
- Lime stabilization.

Chemical coagulents such as ferric chloride, alum, lime, and polyelectrolytes are added to septage to cause a solid-liquid phase separation and to improve the dewatering characteristics of septage. Coagulant addition takes place in rapid mix tanks. The coagulant reacts with water molecules to form polymers that trap suspended solids and then precipitate out in settling tanks.

The two-stage chemical process involves acidification of septage to cause solid-liquid phase separation. The liquid (supernatant) portion from the settling tank is treated with lime prior to final disposal.

Rapid chemical oxidation of septage uses heavy doses of chlorine to oxidize the septage. The relatively inert residue is clarified, and then dewatered by vacuum filtration, lagoon or drying beds.

Lime stabilization of septage involves the addition of lime slurry to raise the pH of the material. The process consists of septage collection, mixing, and reaction with lime to pH 12. The stabilization may be followed by a dewatering step, or the stabilized liquid septage may be spread on the land. The stabilized septage is low in pathogens, its odors are eliminated, and the organics do not decompose easily.

Chemical treatment processes generally are preceded by pretreatment of the septage. The liquid and solids that separate after chemical addition may require additional treatment prior to disposal.

COMMON MODIFICATIONS

Lagoons: Many disposal sites have two sets of lagoons in series. One set may be dewatered and scrapped while the parallel set handles the incoming septage. Some lagoon operators apply septage in thin layers. Each layer is allowed to dewater before another is applied. In some cases each layer is covered with soil to help reduce odor and facilitate final covering or cleaning.

Composting: Lebo System - The patented Lebo process consists of aerating septage and then spraying it on piles of sawdust, wood shavings, or other dry organic material. A one- to two-inch application of septage is covered with additional sawdust, and alternating layers of septage and sawdust are formed into a pile. The pile is composted using natural draft aeration for about three months.

Beltsville System - The Beltsville "static pile method" is used to compost dewatered municipal sludge and could be used for septage composting. The process consists of mixing septage with a bulking agent (sawdust, wood chips, or other material). The mixture is placed in a pile on a base consisting of an aeration header covered by 12 inches of wood chips or unscreened, previously composted material. The pile is covered with a 12-inch layer of screened compost to provide insulation and prevent odors from escaping. The pile is aerated by pulling or pushing air through the header. A three-week composting period is followed by a four-week curing period. Bulking material can be recovered by screening after the composting or curing stage.

TECHNOLOGY STATUS

Lagoons for septage disposal are fully developed technologically. The Lebo system has been used for composting septage at several locations in Washington (23). Septage composting by the Beltsville method has been performed at the pilot level only (7).

Raw septage is chemically treated with lime and ferric chloride at an Islip, Long Island, N.Y. facility. The plant has experienced difficulty in producing an effluent of consistently good quality (99). Laboratory and pilot scale studies on treatment of septage (90) have demonstrated that raw screened septage can be coagulated with a number of chemicals such as ferric chloride, alum, lime, acid, or combinations of the above. However, optimizing the dosage of ferric chloride, alum and lime to achieve effective phase separation is difficult. Effective phase separation can be consistently achieved by the application of a two-stage sulfuric acid/lime coagulation process (90).

Chlorine oxidation is used in a number of areas to treat septage and septage/sewage sludge combinations. Existing chlorine oxidation installations have had operational problems including poor solids separation, odor, and mechanical problems (99).

Lime stabilization of septage has been demonstrated in two research projects by EPA (25). Obtaining acceptable septage dewatering on sand drying beds requires large amounts of lime ($\sim$ 180 pounds per ton of dry solids).

APPLICATIONS

Lagoons can be used as a method of septage disposal or as independent septage treatment facilities, with ultimate disposal of supernatant and solids by other means.

Composting is suitable for liquid septage of dewatered septage. Its low capital cost and simple process facilitates small operation in rural areas near septage generation points.

Chemical treatment processes can be used at independent septage treatment plants with complete facilities for receiving, holding, chemical treatment, and final disposal of the solids and liquid. For most small towns, a system of this type could be operated on a batch basis and would require only a part-time operator. For large installations operated on a continuous basis, a full-time operator would be required.

LIMITATIONS

Lagoon disposal systems require some form of centralized management organization to finance, construct, and operate the facility.

A market is needed to handle large volumes of compost. In a municipal facility, compost application on parks and golf courses as a soil conditioner has been acceptable. The process also requires large volumes of bulking agents which must be readily available at a reasonable cost.

A chemical handling and feed system requires a high degree of operator attention. Chemical treatment processes are capital intensive and costly to operate. Therefore, they can be located only in areas where there are high densities of septic tanks so as to achieve economies of scale.

TYPICAL EQUIPMENT

Lagoons: Front-end loader or other sludge removal equipment.

Composting: Front-end loader, blower, tractor-drawn harrow, rotary screen.

Chemical treatment: Chemical storage tanks, mixing tanks, chemical feed equipment, pumps, instrumentation.

PERFORMANCE

Operating problems with lagoons, such as clogging of the sand at the bottom, have been reported (91).

During composting, septage is converted to a relatively stable organic residue. Maximum tempera tures of between 60° and 80°C produced during the first three to five days of stabilization can destroy pathogens. The Rehoboth pilot project (47) has successfully demonstrated septage composting. Due to the high moisture content of septage (generally 90 percent), high water absorbing materials are necessary as bulking agents.

The pilot plant studies on chemical conditioning of raw screened septage with ferric chloride, lime, alum, acid or combinations of the above, has indicated effective separation in six to eight hours (90). Research at the EPA-Lebanon Pilot Plant has shown that lime stabilization of septage at pH 11.5, followed by sandbed dewatering, is a technically feasible septage treatment method. Effective fecal coliform reduction was achieved at pH 11.5 and pathogenic Salmonella species and Pseudomonas aeruginosa in raw septage were destroyed (99).

DESIGN CONSIDERATIONS

Proper siting and installation of the lagoon is essential to avoid adverse environmental impacts. Siting criteria vary from state to state. The following general guidelines established by the New England Interstate Water Pollution Control Commission provide some of the considerations associated with the planning and design of septage lagoons (100).

- Buffer zone - site must be surrounded by a 300-foot buffer zone.
- Groundwater protection - base of lagoon should be located at least 4 feet above the maximum groundwater table (in New Hampshire, the distance is 10 feet).
- Minimum lagoon depth - at least 3 feet deep.
- Access - access to the site should be by all-weather roads.
- Fencing and signs - fencing at least 6 feet high with locking gate is required. Warning signs must be posted on all sides of the facility.
- Lining - the bottom of the lagoon should be lined with at least 1 foot of good, filterable sand
- Septage receiving port - there should be concrete receiving chambers. The scavenger trucks should have accommodations for vertical discharges into the chamber. The septage should be introduced below the liquid surface through a horizontal discharge pipe.
- Hydraulic gradient - the lagoon should have approximately 1/3 of its volume located above ground level to facilitate cleanout processes.
- Monitoring - volume and operating level should be monitored with a staff gauge. A groundwater and surface water quality monitoring program should be included in the vicinity of the site.

- Grading - the site should be adequately graded to prevent surface runoff from entering the lagoons.
- Flow patterns - a minimum of two lagoons in series is recommended. Parallel lagoons are recommended so that one series of lagoons can be used while the other is being cleaned out.

Land requirements for composting depend on the quantity of septage to be treated. The facilities should include: septage receiving station and storage, bulking agent storage, mixing area, composting pads, screening system, compost curing and storage, and leachate handling.

The septage receiving station should be easily accessible by pumper trucks, and should provide coarse screening and adequate clean-up facilities. Septage storage tank capacity should be adequate for two-day peak volume. Bulking agents are needed for septage composting including sawdust and wood chips. Due to the high moisture content of liquid septage, sawdust is used as the primary bulking agent. Approximately 50% moisture content is desirable in septage/bulking agent mixtures. Pilot studies indicate a bulking agent requirement of 6 to 9 cubic yards per 1,000 gallons of septage (7). The Rehoboth pilot project used 4 cubic yards of sawdust, 3 cubic yards of wood chips, and 2 cubic yards of cow and/or horse manure per 1,000 gallons of septage (47). The mixing of a bulking agent and septage can be accomplished manually (front-end loader) or mechanically (pug mill).

The compost pile should be aerated and monitored to maintain oxygen levels of between 5 and 15 percent in the pile. The forced aeration system consists of 4 to 6 inches of perforated pipe, moisture trap, 1/3-hp blower, small compost pile for odor removal, and ancilliary piping. The successful composting operation is governed by three operational parameters: carbon to nitrogen ratio, moisture content, and temperature. Carbon to nitrogen (C:N) ratios determine the rate of composting if adequate moisture content is maintained. C:N ratios of 20 to 30:1 have been suggested as ideal (7). The Rehoboth pilot project, which used cow and/or horse manure as a nitrogen source in a septage sawdust mixture, was judged to have an extremely high C:N ratio (47). A moisture content of between 45 to 65 percent is desirable. The compost pile should reach and maintain a temperature of 60°C for effective pathogen destruction.

The dosages required for chemical treatment are as follows:

- Lime - The quantity of lime required for stabilization is variable depending on the alkalinity and solids concentration of the septage. For pH 12 stabilization, quantities of lime ranging from 0.1 to 0.3 lb lime/lb dry solids are required. For septage with 5 percent solids content, quantities range from 42 to 125 lb lime/1,000 gallons septage (40)

- Chlorine - Doses vary from 700 to 3,000 mg/l depending upon the solids content and the amount of chlorine-demanding substances present (i.e., ammonia)

- Ferric Chloride - Consistantly desirable separation of liquids and solids was achieved with doses of 400 to 600 mg/l. The supernatant from ferric chloride treatment can be improved further by adding lime in doses of 2,500 to 4,000 mg/l (90)

- Alum - Laboratory investigations have indicated that optimum alum does (as $Al_2[SO_4]_3$) range from 2,250 to 8,250 mg/l, depending on the initial septage characteristics (90)

- Acid - Average sulfuric acid requirements to adjust and maintain pH 2.0 range from 3,000 to 4,000 mg/l (90).

The chemical dosages vary with the characteristics of the septage and thus should be determined by jar tests, except for lime and acid which are determined on the basis of pH of the mixture. Chemical feed equipment must be sized for maximum chemical demand.

RELIABILITY

The Lagoon process is simple and little operator expertise is required. The major problem is that high solids carry-over into the second lagoon may result in clogging of the infiltration system. This problem is due to the poor settling properties of septage and excessive loadings (90).

Composting has a high degree of process reliability because the process is simple to operate.

Chemical processes are capable of producing consistently good results if chemical dosages are optimized and if the process is properly controlled. This requires above average operator attention.

ENVIRONMENTAL IMPACT

Odor and groundwater contamination are potential problems with septage lagoons. Odors can be a particular problem when trucks are discharging septage and when lagoons are being cleaned. Odors can be minimized by adding septage to lagoons below the liquid level, and controlled by adjusting the pH of the lagoon periodically to 7-8 by adding lime. Lagoons using leaching as a means of liquid disposal require high rates of percolation, and thus, are located in very well-drained soils. Unfortunately, most well-drained soils generally are less capable of renovating the percolating water, which increases the potential for groundwater contamination.

Potential odor problems during mixing are possible with composting. Leachate could contaminate surface and groundwater if proper collection, treatment, and disposal are not provided.

Chemical oxidation with chlorine has the potential to produce chlorinated organic compounds which could have severe environmental impacts. Disposal of chemically treated septage on land has the potential to contaminate groundwater and the soil.

REFERENCES

91, 23, 99, 11, 100, 85, 7, 47, 90, 25, 40.

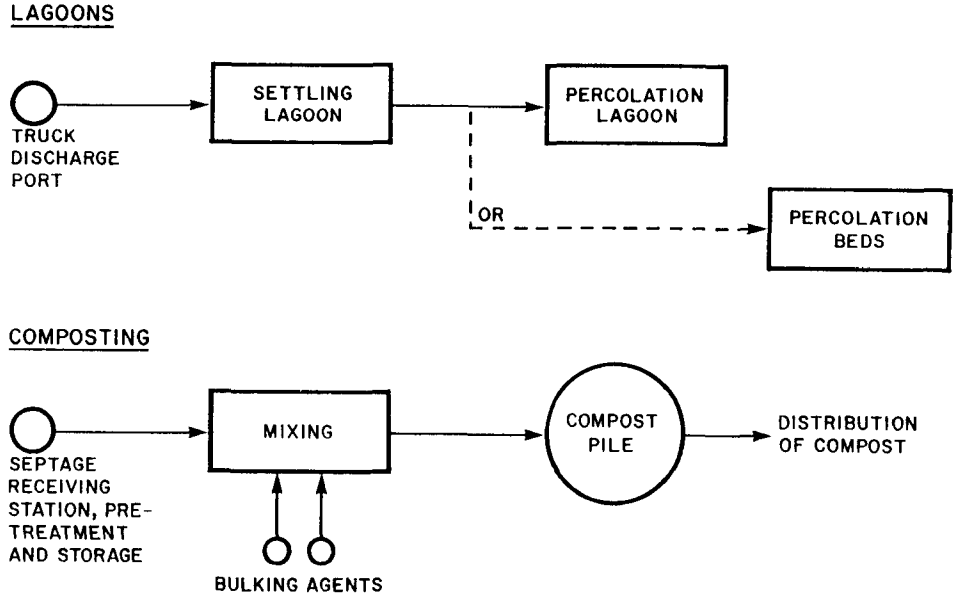
CHEMICALS REQUIRED

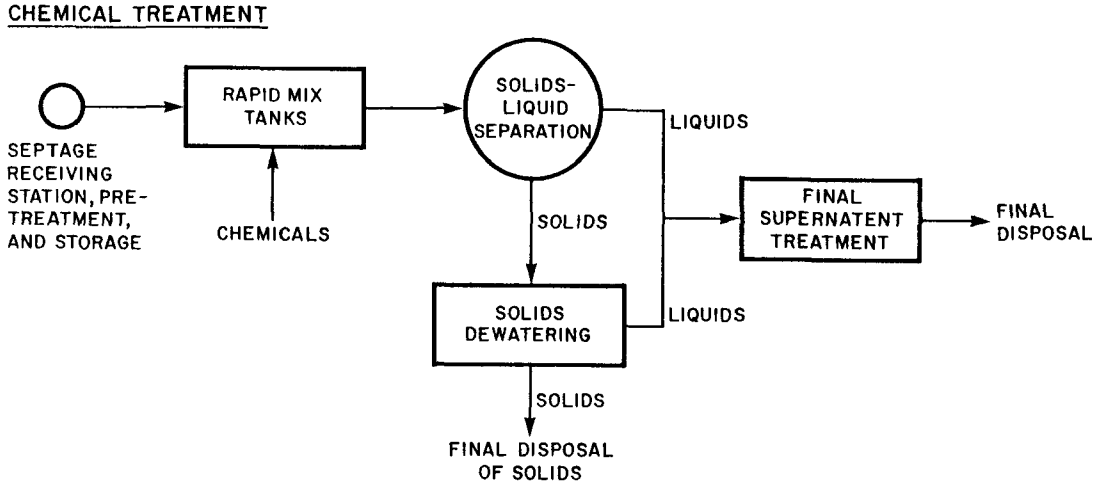
One or more of the following chemicals are required for chemical treatment of septage: lime (CaO or $CA(OH)_2$), ferric chloride ($FeCl_3$), alum (Al_2 SO_4), sulfuric acid (H_2So_4), chlorine (Cl), ferric sulfate ($FeSO_4$), polymers.

RESIDUALS GENERATED

Settled solids on the lagoon bottom are removed and buried in another location or spread on the land; alternatively the solids are left in the lagoon and covered with soil.

DIAGRAM





COSTS

The cost for lagoon disposal, based on several engineering studies, ranges from $6 to $25/1,000 gallons of septage (updated to 1980 dollars). The greatest cost determining factors are land costs and solids disposal costs. The operational cost would increase if groundwater monitoring were required (7). The estimated costs (converted from 1977 dollars to 1980 dollars) of parallel storage basins, consisting of discharge ports and flow diverting structures; three basins, each with usable volumes of 470,000 cu ft; and, a supernatant infiltration area, are as follows (11).

QUANTITY OF WASTE COLLECTED MILLION GAL/YR	ANNUAL COST FIXED $	VARIABLE $	TOTAL $	COST/1000 GAL $
1.0	7,850	4,950	12,800	12.80
1.5	7,850	6,850	14,700	9.80
2.0	7,850	8,850	16,700	8.35
2.5	7,850	10,650	18,500	7.40

The fixed costs include earth moving, grading of the infiltration channel, receiving tank and screen, dumpster, fence, controlled access equipment, water supply, concrete apron, access road, and land. The variable costs include labor, trash disposal (for screenings), sludge removal, and land spreading of dried sludge.

The total amortized cost (updated to 1980 dollars) for septage composting based on design studies varies from $30 to $50 per 1,000 gallons (7). This cost does not include income from the sale of the compost product.

U.S. Department of Agriculture research on the composting of sludge has indicated that the cost per dry ton sludge is sensitive to changes in operating costs, but insensitive to equipment, land, and site development costs. The operation is labor intensive. For 10 dry tons of sludge per day, dewatering facility labor costs are 43% of the total annual cost and 52% of the annual operating cost. Wood chips account for 24% of the annual operating cost (7). For composting of undewatered septage, sawdust is required as a liquid absorbent, which, unlike wood chips, cannot be recovered and reused. This sawdust represents a significant portion of the annual operating cost. If the septage were dewatered to 20 to 25% solids, little sawdust would be required, and woodchips could be used and recovered up to four times.

The cost (converted from 1978 to 1980 dollars) for a 2,500 GPD chemical treatment facility designed for septage treatment exclusively, has been estimated at $44 per 1,000 gallons of septage (90). The facility includes screening and equalization of raw septage, application of the acid/lime addition process, aqueous fraction treatment by intermittant sand filtration, and sludge dewatering by sand drying beds. Several cost estimates have been made for chemical treatment plants, all of which appear to be greater than $30/1,000 gallons septage. A reported cost (updated from 1975 to 1980 dollars) for lime stabilization and sand drying bed dewatering has been estimated at $44/1,000 gallons septage (25). Cost estimates (updated from 1977 to 1980 dollars) for a lime stabilization facility with land spreading ranged from $15.30 (2.5 million gallons per year collection) to $22.50 (1.0 million gallons per year collection) for 1,000 gallons of septage. The cost estimate included: septage receiving tank, bar screen, storage basin, chemical feed system, mixer tank, and land (11). Estimates of the total amortized cost (updated from 1978 to 1980 dollars) for chlorine oxidation septage ranged from $35 - $70 per 1,000 gallons (7). The cost varies with the size of the facility, type of dewatering technique selected, cost of chlorine, method of cake disposal, and need for leachate treatment/disposal.

SEPTAGE TREATMENT AND DISPOSAL AT WASTEWATER TREATMENT PLANTS FACT SHEET 3.3

October 1980

DESCRIPTION

Disposal of septage at wastewater treatment plants is estimated to account for up to 25 percent of the total septage generated; in most cases disposal is accomplished by addition to the liquid stream. However, in some instances septage is handled in the sludge stream and is processed alone or in combination with sewage treatment plant sludge.

Septage is treated in the conventional wastewater treatment plant by adding it to the liquid stream of the plant. The septage can be added to a manhole upstream in a collection system, added as a slug to the headworks of the treatment plant, or added at a controlled rate to the influent stream.

Septage addition to an upstream manhole as a slug is usually preferable to slug addition at the headworks. Upstream addition provides for better dilution of the septage with sewage, and minimizes organic shock to the biological processes and possible upset of the plant. However, addition of septage to manholes is difficult to control and regulate. Uncontrolled upstream dumping of large volumes of septages may have a deleterious impact on plant operation and effluent quality. In addition, incidences of sewer clogging due to grit and debris contained in the septage have been reported (91).

Direct discharge of septage to the headworks of the plant in quantities of 1,000 or more gallons is practiced at many plants throughout the country (91). The procedure, though simple, has been known to cause upsets in plant performance due to temporary hydraulic or organic overloads, clogging or fouling of the plant equipment, or by exceeding the solids handling capacity of the plant.

Controlled addition of septage through the use of a holding tank is the recommended method. This allows slow introduction of septage into the mainstream, thereby minimizing the impact on the plant. This method includes a receiving station for easy and safe transfer of septage from the hauler truck, pretreatment (e.g., screening) to protect the equipment downstream, a holding tank to store and equalize the septage, and pumps to add septage from the holding tank at a controlled rate.

Septage is about 50 times as concentrated as domestic sewage, and loadings of solids and oxygen-demanding substances to the treatment process can increase substantially during septage addition. The additional solids loading due to septage may result in detrimental effects on primary and secondary settling, as well as dewatering of undigested sludge (91). The additional oxygen-demanding load could depress the dissolved oxygen level to zero, and result in plant upset. The treatment plant should have sufficient reserve aeration capacity to accommodate the increased oxygen demand associated with septage.

Septage is added to the sludge stream of the wastewater treatment plant and is treated and processed as a sludge. This may involve such processes as thickening, aerobic and anaerobic digestion, grinding, dewatering, and final disposal. The addition of septage to the sludge handling system at the plant avoids possible problems with pumping, biological overloading, and generation of greater sludge volumes for final disposal.

TECHNOLOGY STATUS

Treatment by addition to the liquid stream of a sewage treatment plant is a common method of handling septage. Approximately 2.5 million gallons of septage were dumped in wastewater treatment plants in Vermont during 1976 (98). Of the 21 plants in Vermont that had received septage in 1976, half were primary treatment plants and half were secondary treatment plants. In addition, several new treatment plants in Vermont, which are presently in various stages of design and construction, are incorporating septage receiving and treating facilities.

Several researchers report good results for aerobic digestion of septage or septage-sewage sludge mixtures (23). Many treatment facilities throughout the county have treated septage-sewage sludge mixtures in aerobic digesters with generally satisfactory results, although some investigators report odor and foaming problems (99). Septage from holding/pretreatment facilities has been added to anaerobic digesters in Westport, Connecticut, and the digesters have operated with good results (99). Bench scale study of anaerobic digestion of septage mixed with treatment plant

sludge has demonstrated that the digesters function well except for a digester loaded with 100 percent septage (99).

APPLICATION

Most sewage treatment plants with biological treatment processes are able to treat septage in the liquid stream. Extended aeration plants, often found in rural areas, are able to handle septage relatively well if sufficient reserve aeration capacity is present. Long detention times in aeration tanks provide for sufficient biological assimilation resulting in acceptable reductions of the BOD and suspended solids. A conventional activated sludge system with a primary clarifier can effectively treat septage as long as organic and solids handling design capacity is not exceeded. The primary clarification facilities preceding the activated sludge process help to level out the fluctuations in loadings imposed by septage addition on the biological process.

Contact stabilization appears to be least amenable for treating septage by addition to the raw wastewater flow. This is due to the short retention time, the high soluble organic content of septage, and the high grease content of septage (99). However, addition of screened septage at a controlled rate to the contact and reaeration zone of a plant operating at 20 percent of design capacity has been demonstrated to result in minor changes in the final effluent quality (90).

Attached growth processes, such as trickling filters and rotating biological contactors, are more resistant to upsets from changes in organic or hydraulic loadings, and are suitable for septage treatment (86). The major concerns with adding septage to trickling filter plants are odor generation, filter clogging, and filter fly proliferation. Additional recirculation in trickling filters has been shown to adequately dilute septage concentrations and diminish chances of plugging the media.

Septage addition to aerobic digestion with the treatment plant sludge should follow screening, degritting and flow equalization. For addition to single-stage anaerobic digestion, pretreatment and equalization is necessary.

LIMITATIONS

The quantity of septage that a wastewater treatment facility can handle is normally limited by the plant's available aeration and/or solids handling capacity. In most rural areas, sewage treatment plants have a limited capacity and handle significant volumes of locally-generated septage. To avoid adverse impacts of septage addition on plant performance, a septage holding and controlled feeding facility should be provided. Sludge processing units should have adequate capacity to handle the additional load from septage.

PERFORMANCE

An experimental pilot plant study has evaluated septage addition to the liquid stream of an activated sludge treatment process using two approaches (87). The first approach added septage on a continuous basis to a system receiving a combination of septage and sewage in varying ratios. The second approach added septage on a shock load basis to a system treating municipal waste only. In both approaches, primary sedimentation was simulated prior to adding septage to the activated sludge. The study indicates that septage addition as either a continuous or intermittent basis is feasible if sufficient oxygenation and sludge handling capacities are available, and control of the waste mixed liquor suspended solids is possible. The particular response of the continuous flow activated sludge system receiving septage depends upon organic loading and septage characteristics. Unacclimated systems were not unduly upset by septage addition, and substantial removals of septage were obtained within a relatively short time. Septage addition monitored at three wastewater treatment plants indicated that septage is a readily treatable waste at extended aeration and conventional activated sludge plants (93).

Aerobic digestion of septage with BOD reduction of 80 percent and a volatile suspended solids (VSS) reduction of 41 percent after a 10-day aeration time; and, COD reduction of 75 percent and VSS reduction of 43 percent after 22 to 63 days aeration, have been reported for sludge stream additions (23). Dewatering and settleability have also been found to improve with aeration, but the aeration time required to affect significant improvement varied. Anaerobic digestion of septage in an unheated 20°C to 30°C digestor resulted in volatile solids reduction of 56 percent after an 82-day retention time and a loading of 0.01 pound VSS/cf/day (23).

A bench-scale anaerobic digester loaded at 0.05 pound VSS/cf/day with a 15-day hydraulic retention time resulted in 45 percent reduction in VSS.

DESIGN CONSIDERATION

The quantity of septage that a wastewater treatment facility can handle is normally limited by the plant's available aeration and/or solids handling capacity. Aeration capacity is most critical at extended aeration plants where the entire quantity of septage undergoes oxidation. Recommended levels of septage addition to different types of activated sludge plants should be followed. In general, conventional activated sludge plants are able to treat septage at about four times the rate of package plants (23). Conventional activated sludge plants with primary clarifiers function well with either constant or shock loading, whereas extended aeration plants function better with additions of septage at constant rates (93). Constant addition of septage requires a storage and septage transfer system.

A new wastewater treatment facility designed to receive septage should include provisions for the additional dissolved oxygen requirements and increased sludge production as a result of septage addition.

There is limited information in the literature regarding acceptable septage loading rates and septage/sewage ratio for aerobic and anaerobic digesters. Septage addition to anaerobic digesters between 5 and 13 percent of total sludge flow has been reported (23). It is also recommended that the initial septage addition to an aerobic digester be limited to approximately 5 percent of the existing sludge flow followed by gradual increases (23).

ENVIRONMENTAL IMPACT

Septage is highly odoriferous and is a health hazard. Personal contact with septage is highly undesirable.

REFERENCES

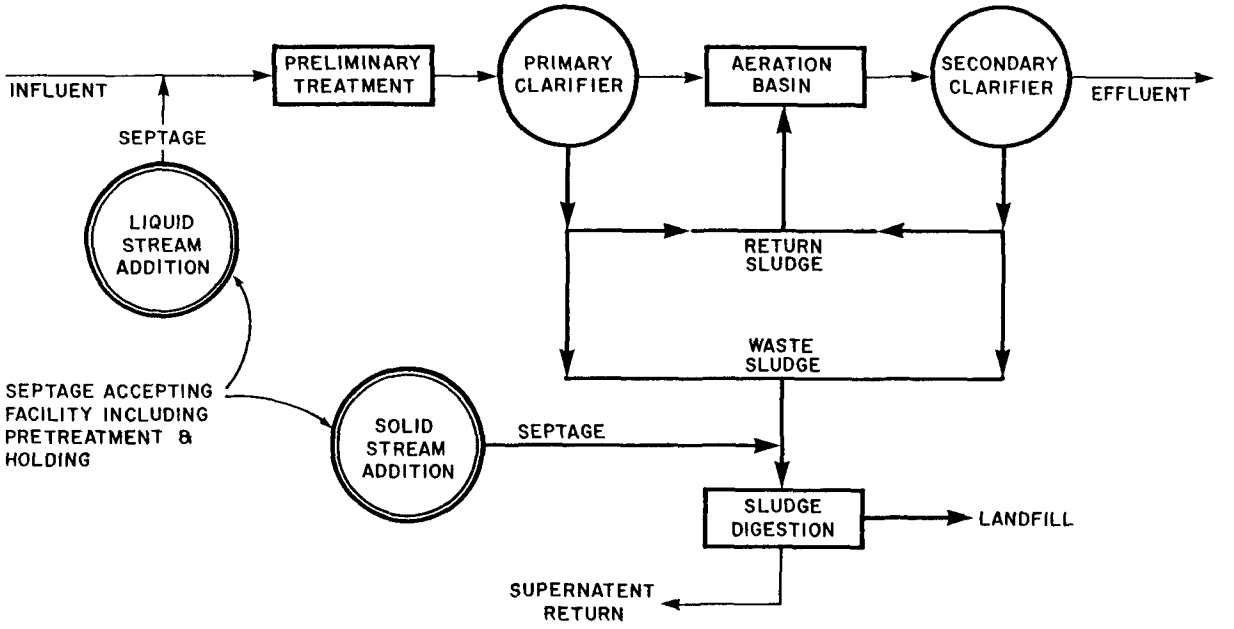
91, 23, 99, 7, 86, 93, 90, 98, 87.

RESIDUALS GENERATED

Septage addition to wastewater treatment plants results in a significant increase in sludge production. At a volumetric septage input of 1 percent of the plant flow, the theoretical sludge production rate can be expected to increase about 50 percent (7). A septage monitoring study has indicated a direct relationship between total solids in combined sewage-septage influent and sludge cake production. A 46 percent increase in total solids resulted in a 41 percent increase in cake solids production (93). Addition of septage may also increase amounts of scum and grit.

DIAGRAM

SEPTAGE TREATMENT AT WASTEWATER TREATMENT PLANTS
(SOLID AND LIQUID STREAM ADDITION)



COSTS

A nationwide survey of the charges levied for disposing of septage in 42 wastewater treatment plants has indicated that the disposal charges vary from $5 to more than $35 per 1,000 gallons, with an average charge of slightly more than $15 per 1,000 gallons (99). These data indicate that the cost of the septage hauler actually pays for dumping septage in a WWTP, but they probably do not accurately represent the actual cost of treating the septage in WWTPs. The actual cost of treating septage in WWTPs, based on engineering cost estimates, ranges from $2 to $38 (1977 dollars) per 1,000 gallons with an average of $23 per 1,000 gallons (99).

Septage treatment costs can be divided into three categories: capital equipment, operation and maintenance, and disposal site acquisition. Capital costs include the cost of solids handling equipment, aeration facilities, and septage receiving stations. Operating costs include labor, power, and possibly additional chemical requirements. Operating and maintenance costs largely depend on whether a plant has primary clarification, and on available economies of scale. The operation and maintenance cost of treating septage at a 1.5 MGD extended aeration facility is between $8 and $12 per 1,000 gallons, and the cost at a 5.5 MGD two-stage activated sludge plant with primary clarifier is about $2 per 1,000 gallons (93). Septage addition in both cases was 2 percent of the sewage flow. The cost difference was attributed to the high power cost of oxidizing septage organics in an extended aeration facility and the economic advantages inherent at the larger plant.

Engineering cost estimates (99) for treating septage with sewage sludge vary from $2 to $19 (1977 dollars). The cost depends on the size of the treatment plant, quantity of septage in relation to quantity of sewage sludge, type of stabilization process (aerobic or anaerobic) chemical requirements, and mode of final disposal.

WETLANDS APPLICATION OF WASTEWATER FACT SHEET 4.1

October 1980

DESCRIPTION

The application of wastewater to wetlands is a method of renovating wastewater through an ecosystem which involves plant and animal reactions in addition to physical and chemical interactions. Wetlands are fresh water or salt water and are naturally occurring or artificially made. Natural wetlands can vary according to source of water (surface, ground, rain, and tidal) and to the flow characteristics (inflow/outflow, no inflow/outflow, or no inflow/no outflow). Artificial wetlands are classified as marshes, marsh-ponds, ponds, or trenches (lined and unlined). Wetlands have been used predominantly for treatment of wastewater that has already been treated to secondary standards (83). Plant and animal organisms reduce BOD and utilize the nutrients in wastewater.

COMMON MODIFICATIONS

Modifications include various methods of application of the wastewater to the wetlands such as point discharge, overland flow, and ridge and furrow application. Specific site constraints dictate the selected method. Additional modifications are based on the point in the treatment process scheme at which wetlands are used. Wetlands can be used as tertiary treatment following a secondary treatment process, or as a secondary treatment process following primary treatment only. Artificial and natural wetlands can also be used in conjunction with one another to complete a treatment scheme.

TECHNOLOGY STATUS

Wetlands have been used specifically for wastewater disposal since the early 1960s. Because of the varying design requirements for specific sites, and hence the lack of consistent design criteria, the technology is not fully developed.

APPLICATIONS

Wetland discharge can be used for small communities where an advanced degree of treatment is required and high groundwater (or other constraint) precludes the use of a subsurface disposal cluster system.

LIMITATIONS

An obvious limitation to wetland discharge is the requirement for a suitable site for natural or artificial wetlands.

TYPICAL EQUIPMENT/NUMBER OF MANUFACTURERS

Construction of pretreatment processes and distribution systems are locally supplied through contractors.

PERFORMANCE

A great deal of confusion exists in the reporting of performance data for natural and artificial systems used for the treatment of wastewater. In most cases, the data are so confounded in a statistical sense that little or no usable information can be derived. Further, there is no standardization regarding the basis on which performance data are reported. For example, in some articles, performance data are reported as a function of time, while in others as a function of distance. Usually, no basis or information is given on how time and distance are interrelated. Further, the data for most of the natural systems are extremely site-specific and should not be generalized (83).

Recognizing these limitations, the removal ranges for the constituents of concern in wastewater are reported in the following table (83). From a review of the limited data presented, it can be concluded that that performance of wetlands with respect to most constituents of concern is not well defined. Further, the range of the values reported is also of concern, especially the lower removal efficiencies (83).

REPORTED REMOVAL EFFICIENCY RANGES FOR THE CONSTITUENTS IN WASTEWATER IN NATURAL AND ARTIFICIAL WETLANDS

	Removal Efficiency, %	
	Natural Wetlands	Artificial Wetlands
Constituent	Secondary	Primary
Total solids	40-75	
Dissolved solids	5-20	
Suspended solids	60-90	
BOD_5	70-96	50-90
TOC	50-90	
COD	50-80	50-90
Nitrogen (total as N)	40-90	30-98
Phosphorus (total as P)	10-50	20-90
Refractory organics		
Heavy metals[a]	20-100	

[a]Removal efficiency varies with each metal.

DESIGN CONSIDERATIONS

Few, if any, design criteria are available to accurately predict the performance of natural wetlands or to properly size artificial wetlands for wastewater renovation. Land area requirements are large and have been estimated at 30 to 60 acres per million gallons of wastewater (83). Because characteristics such as hydrology, hydrogeology, geology, and biology differ from site to site, pilot plant studies are often the best method for determining design criteria. The following table (83) lists preliminary design parameters for planning artificial wetland wastewater treatment systems.

PRELIMINARY DESIGN PARAMETERS FOR PLANNING ARTIFICIAL WETLAND WASTEWATER TREATMENT SYSTEM[a]

		Characteristic/Design Parameter					
		Detention Time, (d)		Depth of Flow, ft (m)		Loading Rate, g/ft^2d (cm/d)	
Type of System	Flow Regime[b]	Range	Typ.	Range	Typ.	Range	Typ.
Trench (with reeds or rushes)	PF	6-15	10	1.0-1.5 (0.3-0.5)	1.3 (0.4)	0.8-2.0 (3.25-8.0)	1.0 (4.0)
Marsh (reeds, rushes others)	AF	8-20	10	0.5-2.0 (0.15-0.6)	0.75 0.25	0.2-2.0 (0.8-8.0)	0.6 (2.5)
Marsh-pond							
1. Marsh	AF	4-12	6	0.5-2.0 (0.15-0.6)	0.75 (0.25)	0.3-3.8 (0.8-15.5)	1.0 (4.0)
2. Pond	AF	6-12	8	1.5-3.0 (0.5-1.0)	2.0 (0.6)	0.9-2.0 (4.2-18.0)	1.8 (7.5)
Lined trench	PF	4-20 (hr.)	6 (hr.)	-	-	5-15 (20-60)	12 (50)

[a]Based on the application of primary or secondary effluent.
[b]PF = plug flow; AF = arbitrary flow.

RELIABILITY

Reliability problems in wetland treatment systems are related to changing climatic conditions, variable wastewater characteristics, local environmental factors, and diseases that interrupt the micro-organisms, plants and animals used in treating the wastewater. In some regards, the potential for and consequences of poor system reliability is greater in wetland systems than in conventional systems because of greater environmental exposure. On the other hand, wetland systems may be less prone to upsets caused by errors in operator judgment. With so few systems operating on a long-term basis, reliability cannot be defined statistically (83).

ENVIRONMENTAL IMPACT

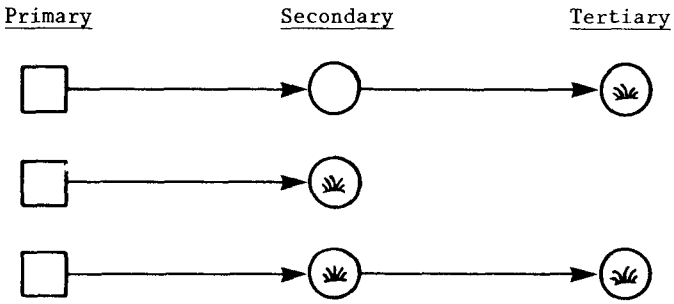
Techniques must be developed to mitigate against possible environmental impacts such as: (1) the breeding and growth of disease (vectors, mosquitos, and flies); (2) induced populations of plants and animals considered to be pests; and (3) the development of odors (83). If disinfection is to be considered, the impacts of disinfectants on the ecosystem must be fully assessed. Additionally, secondary impacts due to the increase in certain species and the decrease in others must be evaluated through pilot plant studies.

REFERENCES

83, 79, 42, 26, 84, 81.

DIAGRAM

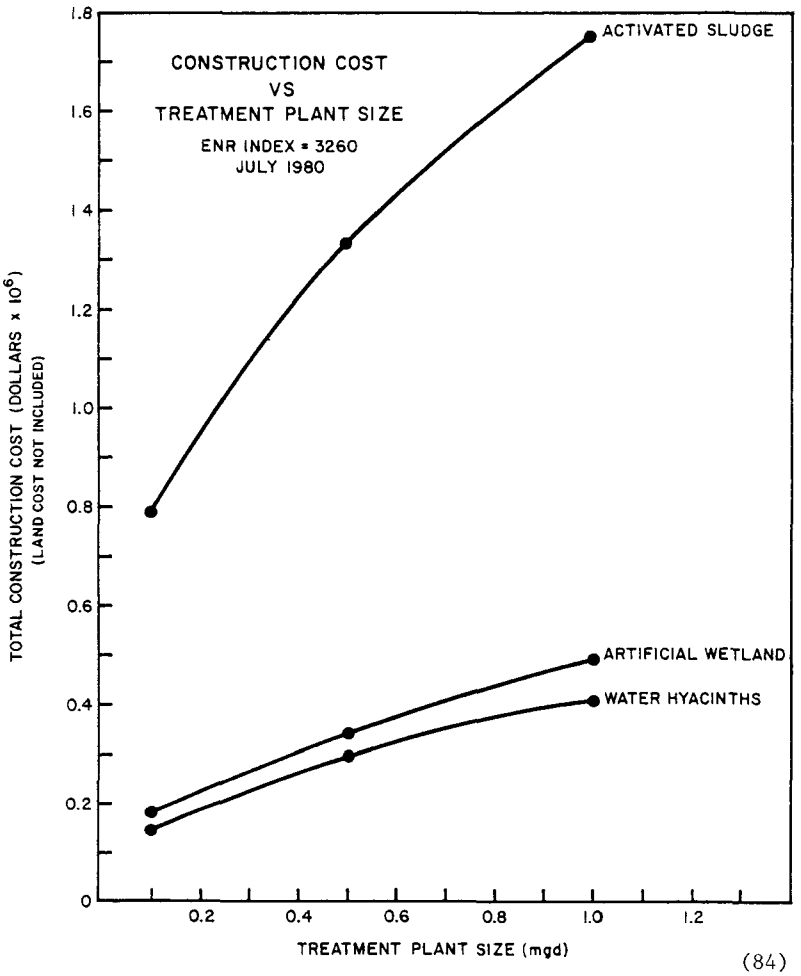
Key: Conventional Primary: Conventional Secondary: Wetlands:



Alternative process schemes using wetlands.

ENERGY NOTES

The following figure compiled from work by Tchobanoglous (84) compares total annual energy requirements to plant size for four different aquaculture systems and a conventional activated sludge plant. Total energy requirements include electricity and fuel as primary energy components. Plant construction, chemicals, and parts and supplies are considered as secondary energy components. Secondary energy requirements are based on a return period of 20 years. The purpose of the figure is to show relative differences and not absolute values.

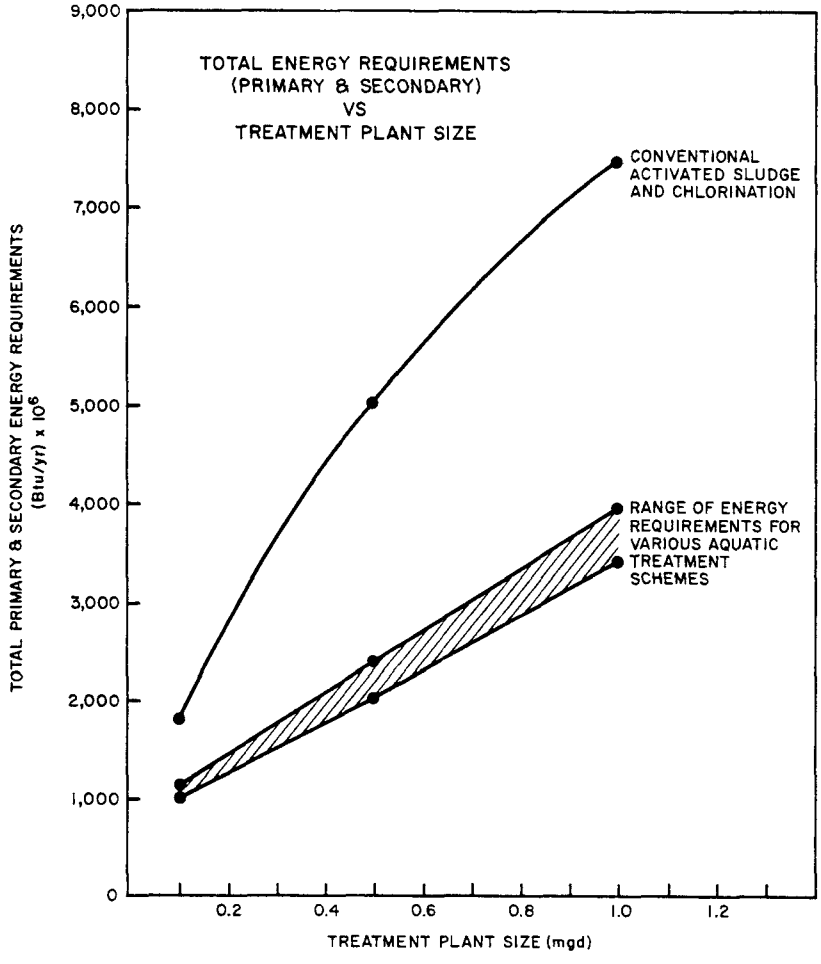


(84)

OPERATION AND MAINTENANCE

Labor requirements in person-days for artificial wetlands and water hyacinths are presented here, along with estimates for parts and supplies (84). Values for activated sludge are given for purposes of comparison.

	LABOR REQUIREMENTS (PERSON-HOURS/YEAR) PLANT SIZE (MGD)			PARTS & SUPPLIES ($/YEAR) PLANT SIZE(MGD)		
TREATMENT PLANT	0.1	0.5	1.0	0.1	0.5	1.0
Artificial Wetlands	500	1500	2000	2300	2875	3450
Water Hyacinths	1500	2000	3000	2300	4025	5750
Activated Sludge	1600	3600	5500	9200	13800	18400



(84)

COSTS

The following figure compares construction cost for complete treatment systems to treatment plant size for activated sludge, artificial wetlands, and water hyacinths. Note that land costs are not included which could be substantial considering acreage estimated for 0.1, 0.5, and 1 mgd for artificial wetlands is 4, 20, and 40 acres, respectively. Acreage estimated for water hyacinths is 1/2 of the requirements for artificial wetlands while only 1/10 of the acreage required for artificial wetlands is required for activated sludge plants. (84)

CLUSTER SYSTEMS FACT SHEET 4.2

October 1980

DESCRIPTION

Cluster systems consist of transportation, treatment, and disposal of waste from a group of homes. Transportation is accomplished through gravity, small diameter, pressure, or vacuum sewers. Treatment takes place in septic tanks at individual homes or jointly at a selected site. The treatment system can be lagoons, septic tanks, package aeration plants, fixed film reactors, or other appropriate small scale treatment process. Disposal methods can be surface land application, subsurface land disposal or stream discharge. Because information on transportation and treatment options is available through various other fact sheets, this fact sheet examines individual septic tanks followed by small diameter gravity sewers and subsurface disposal at a given site. This system is representative of the available cluster system options.

COMMON MODIFICATIONS

Small diameter gravity sewer systems cannot always provide service to all houses. An effluent pump is often required to lift the septic tank effluent to the sewer lines. Pumps are usually set in a separate pumping chamber. The common soil absorption system can be modified by dividing the field into two or three fields so that resting can be provided for the drainfield.

TECHNOLOGY STATUS

Small diameter sewers have been used extensively in South Australia with good results since 1962 (27). Subsurface disposal systems are fully developed technologically, although multi-family application is a more recent development.

APPLICATIONS

Cluster systems can be used to avoid site limitations such as poor soils or shallow bedrock at individual sites. Developed areas that are isolated from the remainder of a community are candidates for cluster systems, especially if the isolated development has small lot sizes not appropriate to on-site treatment systems.

LIMITATIONS

Unlike individual on-site systems, cluster systems require management to ensure that the treatment and disposal process is operating properly and that septic tanks are pumped regularly. Solids overflow from septic tanks contributes to clogging of small diameter gravity lines and increases the solids loading on the soil absorption system.

TYPICAL EQUIPMENT/NUMBER OF MANUFACTURERS

Small diameter pipe and materials for subsurface disposal systems are locally supplied.

PERFORMANCE

Small diameter gravity sewers have performed well, based on the limited amount of data available (27). Large subsurface disposal systems have also functioned satisfactorily in multi-family applications when properly designed, located, constructed, and maintained.

DESIGN CONSIDERATIONS

Small diameter collection sewers should be a minimum of 4" in diameter to facilitate cleaning equipment. The 4" sewers can be laid on a minimum 0.67% grade based on a minimum velocity of 1.5 FPS at half-pipe capacity. Clean-outs should be provided wherever an individual connection joins a sewer line or every 100 feet, whichever is less. Infiltration/inflow is usually less with small diameter lines than with conventional lines. Subsurface disposal systems should follow basic design criteria for individual systems with particular emphasis placed on the distribution of effluent to the soil (gravity dosing or pressure distribution is recommended). Monitoring wells are required to (1) monitor the performance of the system, and (2) protect groundwater resources by monitoring the level of contaminants reaching various groundwater levels. Hydrogeologic studies on potential sites should be conducted prior to construction of a common soil absorption system. Delegation of the management authority is necessary to ensure continued operation.

RELIABILITY

Excellent reliability can be expected for small diameter collection lines because of their dependence on gravity and their corrosion-free construction. An increase in pumps throughout the system would reduce the overall reliability. Gravity dosed soil absorption systems have a high degree of reliability. System reliability is increased by dividing the drainfield into three separate fields each designed to treat 50% of the design flow. The field that is resting acts as an emergency backup.

ENVIRONMENTAL IMPACT

Small diameter sewers limit the increased land development impact usually associated with sewers because of their smaller capacity. Construction impacts may also be less than for conventional sewers if smaller trenches, fewer manholes, and faster construction are required.

Subsurface disposal systems can provide sufficient filtration, adsorption, and micorbial degradation of harmful constituents in household effluent. System failures, however, can occur and are related to site limitations or to improper installation, operation, or maintenance. Adverse environmental effects attributed to failures of subsurface disposal systems generally involve the contamination of ground or surface waters. This contamination may then result in public health hazards if drinking water is affected, or in accelerated eutrophication if excessive amounts of nutrients are delivered to lakes or streams.

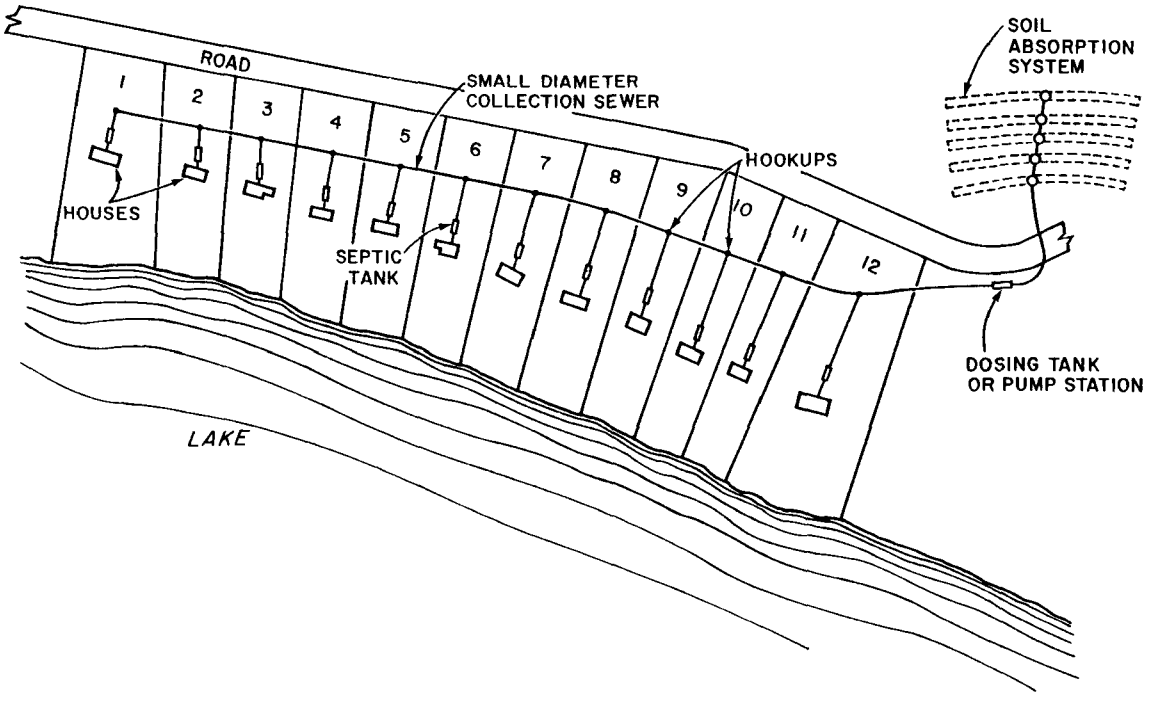
Mitigation and avoidance of adverse environmental impacts depend on information regarding site-specific conditions that led to system failures in a community, and the use of this information to select repairs and replacement systems and to design new systems.

REFERENCES

27, 94, 18, 49, 85, 77, 71, 44.

DIAGRAM

CLUSTER SYSTEM USING SMALL DIAMETER COLLECTION SEWERS AND COMMON SOIL ABSORPTION SYSTEM



ENERGY NOTES

The only energy consuming components of this type of cluster system are pumps for transporting septic tank effluent when gravity lines are not feasible and for distributing the effluent over the drainfield. Total energy required would depend on the design flow and number of pumps required in the collection system.

COSTS

1980 dollars: ENR Index = 3260.

The following construction costs for small diameter collection lines are based on a design population of 500 and a length of sewer per capita of 35.3 ft. in an area of ideal topography and new development (44).

COMPONENT	ESTIMATED COST ($/HOUSEHOLD)
Building Sewer and Septic Tank	528
Mainline Construction Cost (4 gravity sewer)	1854
Ancillary Cost	373
On-Lot (House Connection)	370
TOTAL CONSTRUCTION COST PER HOUSEHOLD	3125

The following construction costs for subsurface disposal systems assume three fields, each designed to accept 50% of the flow. A trench design using a percolation rate of 45 mpi is assumed with a three-foot-wide trench. Costs for hydrogeologic studies are not included. Monitoring wells are included in the estimates.

FLOW(gpd)	CONSTRUCTION COST($)
2,000	26,260
10,000	93,036
20,000	171,375
30,000	267,985
50,000	458,456

OPERATION AND MAINTENANCE

O & M costs for small diameter collection sewers are estimated to be $500-600/mile/year (18). O & M associated with gravity-dosed subsurface disposal system are related to checking the system periodically and taking samples from the monitoring wells. These costs are estimated to be $100 per monitoring well per year.

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CHAPTER II

EVALUATION AND DESIGN METHODOLOGIES

A. WATER QUALITY IMPACTS OF ON-SITE SYSTEMS

The purpose of this report is to review the available literature regarding on-site sytems in order to evaluate their effects on ground and surface water quality. Whenever possible, particular emphasis is placed on the literature that pertains to rural lake communities in the Region V states. Relatively few field investigations have been conducted in these areas; thus, these results are supplemented with research from other geographical areas and from laboratory studies.

Septic tank systems are used by approximately 25% of the U.S. population (Scalf et al., 1977). Wastewater from households can contain pathogenic bacteria and viruses, nutrients, and other chemical pollutants. In a successfully functioning system, the surrounding soil will absorb and purify most wastes. The goal of the system is to remove pathogens and to reduce the concentrations of the other wastewater constituents so that no adverse effects occur from consumption of the receiving water or from addition of the effluent to ground or surface waters.

In many cases, given proper design, installation, and operation, on-site systems will perform satisfactorily; however, failures do occur and can result in the contamination of local water resources. Two basic types of septic tank failures are recognized. One type of failure generally is caused by soil clogging, which severely restricts or even eliminates the wastewater flow into the absorption field. This is manifested either by surface seepage of partially treated effluent above the drainfield or by wastewater backing up into the plumbing fixtures in the house. In the first case, the seepage can create standing pools of effluent that can cause odors, attract insects, and pose health risks to playing children or that can be carried with surface runoff into nearby domestic wells, lakes, or streams. The second type of failure, which is potentially more serious and less obvious, occurs when septic tank effluent reaches the groundwater without sufficient treatment. This, too, can result in contamination of private water supplies or nearby surface waters fed by groundwater sources.

The contamination of ground or surface waters from domestic wastewater may create both public health and environmental hazards. Outbreaks of waterborne disease have been traced to microbial or viral contamination of groundwater by malfunctioning septic tanks. It would seem likely that inadequately treated wastewater that surfaces above the drainfield could cause disease from direct contact or from vector transmission, although no reports of such disease outbreaks were encountered in the literature review. In addition, nitrogen, in the form of nitrate, has been linked with cases of methemoglobinemia in infants. From the environmental standpoint, accelerated eutrophication can result if excessive nutrient concentrations reach surface waters. Lake shorelines are particularly sensitive; in some lakes, growth of aquatic plants in shallow waters has been blamed on emerging groundwater plumes containing septic tank effluent along the lakeshore. Other chemical constituents in household wastewater pose potential health and environmental problems if they are not degraded or purified in the soil before reaching receiving waters.

The available literature is discussed in this report primarily in terms of the water quality impacts that can result from the discharge of pathogens

and nutrients into soil. Other chemical compounds that possibly could be present in domestic wastewater are discussed in less detail.

The studies reviewed indicate that under some conditions the potential exists for the entry, survival, and migration of these pathogens and pollutants in aquifers and surface waters. No one has attempted to estimate the probability of the occurrence of adverse water quality impacts from the use of conventional on-site systems. This is undoubtedly a result of the many controlling factors that are involved. Generally, such things as site-specific conditions, hydrogeology factors, meteorologic considerations, system maintenance, and population or system densities work alone or in combination to bring about adverse effects.

Standard design and installation codes are implemented to protect human health and the environment. It is unreasonable, however, to expect that the application of one set of design factors throughout a wide area would prevent contamination of waters in all cases. The literature on the subject documents individual cases where problems have arisen without emphasizing that many systems operate successfully without harm to public health and the environment.

The interaction of septic tank effluent and the underlying soil is complex. Proper performance of conventional on-site wastewater disposal systems depends on the inherent ability of the soil to transmit and renovate the wastewater. This in turn is dependent on many other factors, some of which are discussed in the following sections of this report. The wastewater absorption capabilities of soils are described in more depth elsewhere (Bouma, 1975; Miller and Wolf, 1975; Tyler et al., 1978; University of Wisconsin, 1978).

Prolonged infiltration of septic tank effluent into a soil absorption field will cause the development of a crust with a low permeability at the effluent infiltrative surface. The formation of this "clogging zone" is a result of physical, biological, and chemical processes. The soil type is important. Soil is composed of many particles of different shapes and sizes that are interconnected by pores or capillaries. Depending on the soil moisture content, these spaces are filled with varying proportions of air and liquid and it is through these capillaries that the effluent flows. Most wastewater solids settle in the bottom of the septic tank but some flow out with the effluent into the seepage bed and contribute to the clogging of capillaries at the soil-effluent interface. The sewage bacteria in the wastewater merge with the soil bacteria and multiply rapidly at this surface, where they are provided with a constant source of nutrients. The resultant biomass further restricts flow through the capillaries and is effective in degrading organic wastes in the effluent.

These and other processes contribute to the clogging phenomenon. Although the rate of flow into the soil is reduced, the clogging mat also enhances the purification of the effluent by increasing its retention time. As long as there is periodic drying, the aerobic environment at the soil-effluent loading surface should be maintained, as should the infiltrative capacity. Both of these conditions are essential for the successful operation of the on-site system and contribute greatly to the renovation of typical household wastewater constituents (Harkin et al., 1975).

1. FATE OF MICROORGANISMS IN DISCHARGE FROM ON-SITE SYSTEMS

From the public health standpoint, concern exists about the potential for movement of disease-causing bacteria and viruses through the soil below on-site disposal systems into groundwater supplies and over the soil to surface waters. The potential health hazard from pathogenic organisms will depend on their presence in the wastewater and in drinking water, the numbers that are present, and the accessibility of the pathogens for subsequent human consumption. These factors are influenced further by the general status of the pathogen population, the pathogen survival rates outside the host, and the movement of the organisms within the environment (Miller and Wolf, 1975).

Bacteria normally are present in the human intestinal tract. Many of these are the same types of bacteria that are naturally present in soils and water. Together, the sewage bacteria and native soil bacteria are capable of reducing complex, organic solids into more simple and soluble compounds. In this respect, bacteria perform an essential step in the wastewater treatment process.

In addition to the normal intestinal bacteria, however, domestic wastewater also may contain pathogens, if individuals in the households are infected and are shedding pathogens with feces. Detection of all types of pathogenic bacteria and viruses is difficult, so tracer organisms are often used as indicators of fecal pollution. A low density of indicator organisms implies an absence of pathogens. Two frequently used indicators are fecal coliform bacteria and fecal streptococci because they:

- are present in high numbers in feces of humans and other warm-blooded animals;
- can survive outside the body for long periods of time;
- are detectable quantitatively with available methods; and
- are found in relatively higher numbers than the pathogens, so their detection is more likely (University of Wisconsin, 1978).

The presence of pathogens in drinking water, even if found in relatively small numbers, is considered hazardous because the minimum infectious dose has not been established accurately for all pathogens (Pipes, 1978). Available information suggests that a relatively large number of enteric bacteria, but only a few enteric viruses, are needed to bring about a clinical disease (Cooper and Golueke, 1977). There are a number of diseases that are of fecal origin. Bacteria and viruses can cause intestinal or enteric diseases such as food poisoning, typhoid fever, cholera, hepatitis, and other gastroenteric and diarrheal diseases (Craun, 1975; Pipes, 1978).

Many wastewater-borne pathogens have been implicated as causative agents in outbreaks of enteric diseases. Almost half of the 192 outbreaks of waterborne disease that affected 36,757 people in the U.S. between 1971 and 1977 were attributed to the use of untreated or inadequately treated groundwater (Craun, 1979a). Twelve percent of the outbreaks occurred in individual systems and affected an average of ten persons per outbreak. It is likely that the published figures understate the actual incidence, for several

reasons. Generally, waterborne disease outbreaks are reported only when the disease is severe and there are a number of common source cases. Also, sporadic cases go unnoticed or cannot be ascribed categorically to the waterborne route. Finally, contamination is often temporary and may be cleared up before a water sample is taken so that no agent is identified (Pipes, 1978).

Overflow from septic tanks and cesspools was responsible for 42% of the disease outbreaks that occurred in nonmunicipal systems during the period from 1946 to 1977 (Keswick and Gerba, 1980). Thus, the public health concern regarding on-site systems appears justified. On the other hand, well-functioning soil absorption fields have been shown to be very efficient in removing disease-causing microorganisms. Many system failures that result in microbial contamination of nearby waters can be attributed to improper design, installation, or operation of the systems. Numerous laboratory and field studies have been conducted to determine which factors affect the removal, survival, and migration of pathogens in soil and groundwater systems. The aim of these studies is to avoid future septic tank failures and to minimize potential health hazards. Many of these investigations are reviewed in this section.

a. Bacterial Quality of Septic Tank Effluent

While the level of treatment in a septic tank is significant, the effluent still contains extremely high numbers of bacteria. Bacteriological analyses were performed on effluent samples from five septic tank systems (Ziebell et al., 1975a). The bacterial quality data are summarized in Table II-A-1. For comparison, the current fecal coliform standard specifies that for discharge of effluents to surface waters the concentration cannot exceed 200 fecal coliform/100 ml. Although there are no comparable standards for groundwater discharge, this provides an indication of the degree to which septic tank effluent must be purified in the soil before reaching groundwater that may be used for human consumption without pretreatment. To minimize the risk of the transmission of enteric disease, therefore, the soil must be capable of removing the pathogenic bacteria that may enter the soil in septic tank effluents.

TABLE II-A-1. REPRESENTATIVE BACTERIOLOGICAL QUALITY OF SEPTIC TANK EFFLUENT (Ziebell et al., 1975a)

Bacteria/100 ml	Geometric mean	95% Confidence interval	Number of samples
Total coliforms	34×10^5	26×10^5-44×10^5	91
Fecal coliforms	42×10^4	29×10^4-62×10^4	94
Fecal streptococci	38×10^2	20×10^2-72×10^2	97
P. Aeruginosa	10×10^3	1.9×10^3-54×10^3	13
Total bacteria	34×10^7	25×10^7-48×10^7	88

b. Bacterial Removal

Soils have a tremendous capacity for the removal of bacteria from domestic wastewater. As noted, however, effluents from septic tanks contain large numbers of bacteria. These bacteria are removed primarily by straining, sedimentation, entrapment, and adsorption (McGaughey and Krone, 1967). Most bacteria are filtered out of the percolating effluent at the clogging zone, which generally occurs within the first six inches. Romero (1970) further attributes the rapid removal of wastewater bacteria to adverse conditions in the soil: abrupt temperature changes; oxygenation and nitrification; and die-off as a result of competition with native soil bacteria.

Many field and laboratory studies have been conducted to examine the efficiency of soil for bacteria removal and to determine the parameters that affect this efficiency. Factors that are considered important include soil type, temperature, pH, bacterial absorption to soil and soil clogging materials, soil moisture and nutrient content, and bacterial antagonisms (Gerba et al., 1975). The degree of saturation also is important. Unsaturated flow increases effluent and soil contact time, which enhances purification (University of Wisconsin, 1978). Most of these same factors also affect bacterial survival and movement.

As part of the Small Scale Waste Management Project at the University of Wisconsin-Madison, Ziebell et al. (1975b) conducted a series of soil column experiments to evaluate bacteria removal from septic tank effluent by different soil types under varying temperature and flow regimes. Columns of 2 feet (60 cm) in length were filled with either loamy sand or silt loam. Fewer bacteria passed through the sand column that was loaded with 5 cm/day of septic tank effluent than through the one that received a 10cm/day dose. Once a clogging zone developed at the infiltrative surface, removal rates were improved further. Soil temperature was also found in part to affect bacterial removal rates, by inducing early soil clogging at low temperatures. At 25° C, it took approximately 100 days for the clogging mat to form, whereas at 5° C, the mat developed within 10 to 20 days, but was followed by effluent ponding. This ponding was attributed to the accumulation of organic compounds that were not decomposed rapidly enough at the low temperature. For safety, the investigators recommend depth of greater than 2 feet in sands and an application rate of less than 10 cm/day. This is twice the maximum rate allowed by many state design codes. The separation distance is one-half to two-thirds of the Region V states' requirements for all soils.

The period before clogging occurs corresponds to the critical phase of the operation of a new system. During this time, high loading rates may allow the transport of pathogens deep into soil or groundwater. A newly installed septic tank system at an Illinois State Park was implicated as the source of well water contamination that resulted in an outbreak of 60 cases of gastroenteritis. The system had been installed in accordance with the local code, but during installation, groundwater was observed at 10 feet below the soil surface and the well water was reported to be turbid for a short period after installation (Center for Disease Control, 1972). It has been noted that turbidity strongly influences pathogen persistence and movement by protecting the pathogen from inactivating agents (Pipes, 1978).

Concurrent column studies of silt loam were conducted by Ziebell et al. (1975b). Silt loam soils, which are characteristic of aggregrated soils, have a slower permeability than do coarser-textured soils, but are easily saturated. At a loading rate of one cm/day and with no clogging present, effluent short-circuited through larger pores. Significant numbers of bacteria were detected in the column effluent, which presumably was due to the short retention times that did not permit adequate purification. Brown, et al. (1978) observed similar bacteria movement down root channels, or cracks in their 2 year lysimeter test of a sandy clay soil. When Ziebell et al. reduced the loading rate to 0.3 cm/day, greater bacterial removal was achieved. The authors indicated that two feet of slowly permeable soils having deep groundwater tables would provide sufficient protection against microbial contamination of groundwater. In similar soils with shallow groundwater tables, however, a greater soil depth would provide for a margin of safety.

Other investigators have singled out the importance of soil type and saturation level as key factors in bacterial removal. In a review of the literature regarding subsurface pollutant travel (Brown and Caldwell, 1977), it was concluded that in aerobic, fine-textured, unsaturated soils, most bacteria will be removed within a distance of two to three feet from the drainfield trench surface; however, if any one of these three conditions is not present, then bacteria travel might be much greater. This same review reports that a few feet of aerobic, unsaturated soil will reduce bacteria below detectable levels, while the same removal may require over 200 feet in coarse-textured, saturated soils. Fine soils are effective in removing bacteria, but they are saturated easily and then may lose some of their effectiveness. The coarser soils promote aerobic conditions, which are favorable for bacteria removal, but with the more rapid percolation rate, if the groundwater table is near, microbial contamination might result. Saturated flow regimes in any soil will force the soil water downward at a faster than normal rate, thereby reducing retention time and purification (Peavy and Groves, 1977) and increasing contamination potential.

Several investigators that have evaluated soils' ability to remove bacteria from domestic wastewater have specified the minimum soil depth that is necessary for this removal. Otis (1975) indicates that field monitoring of operating conventional septic tank systems has shown that two feet of natural soil is sufficient to reduce bacterial populations in septic tank effluent to background levels. Brown et al. (1978, 1979) evaluated bacterial removal in sandy loam, sandy clay, and clay soils. Fecal coliforms rarely were found below one foot from the bottom of the trenches in all soils and only occasionally were they detected at four feet and then only at low concentrations. Peavy and Groves (1977) demonstrated in their study that fecal coliforms were removed within four feet in sandy soils. Tyler et al. (1978) report that at a distance of one foot into soil surrounding a trench there was a three log reduction in bacterial numbers and that within the second foot, counts were at the acceptable range for fully treated wastewater. Finally, from a survey of conventional on-site systems operating in unsaturated, medium sand soils, Bouma et al. (1972) found that bacteria were removed rapidly in the soil and that bacteria beyond one foot below and to the side of the trench were similar to the natural soil microflora.

From the proceeding discussion, it should be clear that it is not possible to specify, with any certainty, the separation distance between the

bottom of septic tank soil absorption systems and water tables. Many site-specific factors are involved and must be considered in the determination. In many favorable situations, two to four feet of unsaturated soil may be sufficient for removal of bacteria and protection of groundwater.

c. Bacterial Survival

If pathogenic bacteria are not removed from the septic tank effluent within a few feet of the drainfield by filtration, adsorption, or die-off, they may survive temporarily. In general, pathogen survival in soil is dependent on the microbiological and biochemical properties of the pathogen, soil properties, and environmental conditions (Miller and Wolf, 1975). More specifically, some of these factors include soil texture, organic matter levels, pH, moisture content, temperature, and competition from native soil microflora.

In a study to determine the survival of bacteria, Rudolfs et al. (1950) found that colder temperatures and higher moisture levels in soils increased the survival rates of Salmonella typhi, the causative agent in typhoid fever. These investigators reported survival times of 2 years in frozen moist soils as compared to 24 hours in a peat soil, which has a low pH. It has been suggested that lower temperatures enhance bacterial survival by extending the lag period before exponential die-off begins (Seidler, 1979).

Zibilske and Weaver (1975) also investigated the effects of temperature and moisture content on the survival of bacteria. Moist clay and sandy loam soils were injected with Salmonella typhi and stored at different temperatures. After 12 weeks, 33% of the soil samples that were stored at 5° C and 22° C yielded viable bacteria. Whereas, at 39° C, only 25% of the soil samples contained S. typhi after 1 week. Bacteria survival rates then were compared in moist, flooded, and air-dried soil samples. In the moist and flooded soil samples, S. typhi was recovered from 25% of the samples after 12 weeks. After 1 week, 33% of the air-dried soil samples contained the bacteria.

Results of another study indicate that bacteria added to sterile soil samples will survive longer than in non-sterile soils because of the lack of antagonism from native soil bacteria. Survival rates also are increased when high levels of organic matter are added to soil. Because sewage is high in organic matter content, it would likely enhance survival rates by providing a source of nutrients (Dazzo et al., 1973).

Wide ranges for bacteria survival times are reported in the literature. Patterson et al. (1971, as cited by Viraraghavan 1978), compiled data on survival rates for various types of bacteria in soil. Survival time for Salmonella ranged from 25 to 41 days and for coliform bacteria, 4 to 90 days. Miller and Wolf (1975) presented slightly higher ranges of 1 to 85 days and 100 to 150 days for Salmonella and coliform bacteria, respectively. These ranges apparently reflect differences in soil types and environmental conditions of the experiments.

Bacterial survival in water is affected by many of the same factors that are important in soil. Generally, low temperatures, high organic matter levels, and reduced competition increase survival times in water. McFeters et

al. (1974) compared die-off rates of fecal indicator bacteria and enteric pathogens in fresh well water. They determined the time that was required for a 50% reduction in the initial population. The half-time averages for coliform bacteria was 17 hours and for fecal streptococci, 22 hours. Die-off rates for several types of pathogenic bacteria in water ranged from 2.4 hours to 26.8 hours. With these survival rates, pathogens could move considerable distances from the source of contamination while remaining infective.

d. Bacteria Migration

As has been shown, fecal bacteria and pathogens may survive relatively long periods and during this period may be transported deep into the soil or water table. Microorganisms are not capable of migration per se, but rather they are carried along by liquid flowing through the soil (Viraraghavan 1978). Generally, many of the conditions that may prevent the removal of pathogenic bacteria from septic tank effluent within the first few feet of soil are the same conditions that are responsible for the increased survival times and travel distances.

Although there are many other factors involved, subsurface bacterial movement is affected primarily by soil texture and moisture content (Harkin et al., 1979). Coarse-textured soils such as sand or loamy sand allow greater bacterial travel than finer-textured soils like silt loam or clay loam. Miller and Wolf (1975) indicate that coliform bacteria may move more than 200 feet in a coarse gravel-sand, but that distances are much shorter in finer textured soils. Brown and Caldwell (1977) reviewed the literature on bacterial subsurface travel from septic tank systems and concluded that essentially all bacteria are removed within two to three feet in unsaturated soils. However, conditions could be present that could result in much further transport. They report that microorganisms may traveL more than 325 feet (100m) if water saturates the soils below the drainfield creviced bedrock exists at shallow depths or coarse-grained soils extend from the ground surface to the water table.

Hagedorn et al. (1978) and Rahe et al. (1978) both found movements of fecal coliforms over long distances in relatively short times. Travel rates in the saturated soils were as fast as 50 feet per hour. This rapid movement was thought to be due in part to flow through the larger pores and root channels in the soil. Travel rates were greatest during periods of heavy rainfall. Other investigators have estimated that bacterial movement may be from five to ten times greater in saturated soils than in unsaturated soils (Peavy and Groves, 1978). Movement of bacteria through bedrock was shown by Allen and Morrison (1973) to be rapid. Liquid in bedrock flows through the fracture lines or joints, where the seepage velocity can be great, and leads to rapid and wide dispersal of bacteria in groundwater. In their study, tracer bacteria moved 94 feet in 24 to 30 hours. These studies highlight some of the conditions under which bacteria have been transported far and quickly.

Once pathogenic bacteria are introduced into the aquatic environment, they may persist for hours to days under the right conditions, and may be widely dispersed from the source of contamination. Evidence of bacterial travel also can be found in epidemiological studies. The pathogenic bacteria, Shigella sonnei, was implicated as the causative agent in an outbreak of 1200 cases of gastrointestinal illness in Richmond Heights, Florida, during 1974.

A community water supply well was contaminated with insufficiently treated effluent from an on-site disposal system at a church day care center located 150 feet from the well. A breakdown in the chlorination facility resulted in the distribution of contaminated water (Craun, 1977). In another disease outbreak, four members of a household in Washington state contracted typhoid fever as a result of well contamination from a septic tank system that was located 200 feet from the well (Center for Disease Control, 1972). Well water in the area was drawn from a shallow, gravel aquifer. Dye that was injected into the septic tank system reached the well in 36 hours.

Ground and surface waters in unsewered areas where no overt disease symptoms are reported have been found to contain excessive bacterial concentrations. In an area south of Tacoma, Washington, there was a 70% increase per year in median coliform bacteria concentrations in local streams, with median concentrations of 64 MPN/100 ml in 1962, 280 MPN/100 ml in 1969 and 1200 MPN/100 ml in 1973 (DeWalle and Schaff, 1980). Lakes in the area, which had significant lakeshore residential development and septic tank usage, also experienced bacterial contamination, but the actual concentrations were lower than in the stream because of dilution effects. The proportion of lakes in the area that exceeded the 240 MPN/100 ml standard was 19% in 1970, 42% in 1971, and 46% in 1972. The highest bacteria concentrations, averaging 64,000 MPN/100 ml, were found in roadside ditches, which the investigators attributed to the surfacing and runoff of undiluted septic tank effluent. Microbial contamination of groundwater also was evident. Although values were very low, coliform bacteria were present in water supply wells that drew water from 31 feet, 228 feet, and 503 feet.

Seidler (1979) states that runoff across land with malfunctioning domestic drainfields can have two undesirable impacts on the bacteriological quality of ground and surface waters. One effect is demonstrated immediately by the first flush phenomenon of heavy rains. In this case, high numbers of pathogenic bacteria can be carried to nearby waters. The second effect is manifested later and is the result of prior bacterial accumulation in sediments that are later shed into the water and are capable of regrowth during warm, low flow conditions.

These two effects were demonstrated in a study that was conducted in a rural watershed near Corvallis, Oregon (Seidler, 1979; Lamka et al., 1980). Six times during the 15-month project, private well and spring water supplies in a rural neighborhood of 78 households were sampled for total coliforms, fecal coliforms, standard plate count bacteria, and Staphylococcus aureus, a common agent of food poisoning. During the survey, 35% of the households were found to be using water contaminated with bacteria. Climatological and seasonal trends were also identified. The highest incidence of coliform contamination was noted for the sampling days that had been preceded within 24 hours by rainfall. Also, the occurrence of high total plate counts increased during the warmer spring and summer months compared to the cool months. Nine different species of enteric bacteria, including opportunistic pathogens, were identified during the study. The use of septic tank systems in soils generally unsuitable for use in drainfields was seen as a contributing factor to the poor water quality in the area (Seidler, 1979). The study area was characterized as having seasonally high water tables, shallow or slowly permeable soils, and steep slopes. This study demonstrated a seasonal effect on bacterial water quality; maximum coliform bacteria levels were observed

during the winter months when most of the rainfall and soil leaching occurred. Generally, the rainy season or even a major storm event preceded by a long dry spell provides the greatest potential both for surface water contamination by enteric organisms as a result of surface runoff from septic tank system failures and for groundwater contamination from subsurface movement under saturated flow conditions (Seidler, 1979).

Other investigators also have evaluated the bacteriological effects of septic tank failures on water quality. Reneau et al. (1975) found no effect in areas without failing systems, but reported total coliform and fecal coliform concentrations greater than 2.4×10^7 MPN/100 ml in areas where surface seepage of septic tank effluent was evident. The authors noted that 83% of the soil types in the area were marginal or unsuitable for septic tank drainfields.

In another study, private water supplies in three rural communities in South Carolina were tested for total coliforms, E. coli, and fecal streptococci; analyses indicated that 85%, 43%, and 75% of the water supplies were contaminated with the three indicator organisms, respectively (Sandhu et al., 1979). The presence of E. coli shows recent contamination because the bacteria cannot survive long outside its natural habitat. The presence of fecal streptococci in the water samples was seen as further confirmation of human sources of fecal pollution, because no farm animals were located near the water wells. The investigators attributed the bacterial pollution of the water sources in the study area to the inability of the septic tank leach fields to operate effectively in removing pollutants. The highest bacterial counts were found in an area where soils generally were waterlogged, a condition that is conducive to septic tank failures and which probably enhanced the bacterial pollution potential of groundwater. Data from the study suggest that bacterial counts decreased as well depth and it distance from the septic tank increased.

2. VIRUSES

Viruses differ fundamentally from enteric bacteria. Whereas bacteria normally are present in the human intestinal tract and in on-site disposal systems, virus presence is sporadic and only occurs when a household member is experiencing an enterovirus infection. These infections are most prevalent during July, August, and September (University of Wisconsin, 1978). Viruses cannot multiply outside a living cell as bacteria can, nor do viruses require nutrients for survival. Virus particles are extremely small. They range in size from 18 NM to 25 NM (less than one millionth of an inch), as compared to an average size of 750 NM for bacteria (Wellings, 1979). The small size plays an important role in their survival and migration in the environment. Also important is that they are electrically charged and, as such, are removed from percolating septic tank effluent primarily by adsorption rather than by filtration.

There are more than 100 different enteric viruses that can be excreted in human feces and can cause disease (U.S. EPA, 1978). These viruses include: the picornavirus group, one of which is poliovirus, the causative agent in paralytic poliomyelitis; echovirus, which is responsible for meningitis, respiratory disease, and diarrhea; and coxsackieviruses, which are capable of causing herpangina, myocarditis, and pleurodynia, as well as the three dis-

orders noted for echoviruses (Gerba et al., 1975a). Direct evidence for viral etiology of a waterborne outbreak, however, is limited to hepatitis A, adenovirus, and the Norwalk agent (Keswick and Gerba, 1980). This is due largely to limitations of methodologies for the detection of viruses in water and sensitivity of epidemiologic methods for enterovirus diseases.

Although viral contamination has been demonstrated in groundwater in a limited number of studies, groundwater often is used without treatment in rural areas. There is an indication that only one viral plaque-forming unit (PFU) is capable of producing human infection (Sproul, 1975); therefore, the presence of even small numbers of virus in groundwater that is used for direct consumption poses a potential health hazard. Furthermore, a person may contract a viral infection from contaminated water without overt symptoms developing and, as a carrier, could transmit the virus by person-to-person contact (Gerba et al., 1975b).

Viruses may enter surface waters by overland runoff containing surface seepage of septic tank effluent. This comes from individual on-site systems that are located near lakeshores or stream banks. Densities in bottom sediments may be many times greater than in overlying waters (van Donsel and Geldreick, 1971). Viruses initially adsorbed to sediment particulate matter may be resuspended in response to currents, storms, swimming, or motorboat activity and may still retain their infectivity.

a. VIRUS SURVIVAL

Enteric viruses are more resistant to environmental factors than are enteric bacteria and they tend to survive for longer periods (Marzouk et al., 1979; Romero, 1970). This is partly because soils generally are less efficient in removing viruses than they are for bacteria (Pipes, 1978). There are a number of factors that control virus survival in soil, including soil texture, presence of organic matter, ionic strength, adsorption, and pH, as well as microbial antagonism, temperature, and moisture content (Keswick and Gerba, 1980).

Several investigators have evaluated the effect of some of these factors on virus survival in soils. Yeager and O'Brien (1979) conducted laboratory studies using saturated sand and sandy loam soils and storing them at different temperatures. At 37° C, poliovirus was recovered for up to 12 days, at 22° C it was recovered up to 92 days, while at 4° C, polivirus was recovered from the soils for up to 180 days. The authors concluded that the persistence of poliovirus in saturated soils was temperature dependent, regardless of soil type; however, they also noted that adsorption to sandy loam and soil appeared to protect the viruses, whereas adsorption to sand had little effect on the rate of poliovirus inactivation. Results of other experiments indicated that soil dewatering by evaporation destroyed at least poliovirus and cocksackie in six different soil samples; these results suggest that enterovirus contamination of soil that is used for wastewater disposal may be reduced or eliminated by allowing the soil to dry between applications.

As part of the Small Scale Waste Management Program, the role of temperature on inactivation of sorbed viruses was investigated. Minicolumns filled with sandy soil were dosed with labeled virus. After 4 weeks at 6° to 8° C, 57% of the virus was still infective, whereas only 2.5% of the infectivity was

recovered after 4 weeks at room temperature (18° to 24° C). Even after 8 weeks, the sorbed viruses were not inactivated any further in the columns stored at the colder temperature. Additionally, the cold columns allowed approximately 10% of the applied virus to pass through with the effluent. The investigators concluded that sand is less retentive and virus inactivation occurs more slowly, if at all, at low temperatures (University of Wisconsin, 1978).

The results of these studies generate some concern about the operation of subsurface disposal systems. There is some indication that virus inactivation is more rapid near the surface than in subsurface layers because of the detrimental effects of aerobic soil microorganisms, evaporation, and higher temperatures (Keswick and Gerba, 1980). Therefore, the viruses that penetrate the soil more deeply are expected to survive longer than those retained near the soil surface. This, coupled with the evidence that enteroviruses survive longest during cool weather and can survive freezing (Hain and O'Brien, 1979), leads to the concern for potentially prolonged periods of contamination. Viruses that are deposited during the winter could possibly move to the surface or deeper into the soil or groundwater during the spring snow melt when flooding or saturated soil conditions are common in the northern Region V states. Even heavy rainfall or septic tank usage could permit migration of infective viruses into groundwater (Yeager and O'Brien, 1979).

Viruses have been shown to exhibit survival time of from weeks to months in aquatic environments. Water temperature especially influences survival, with high temperatures promoting inactivation and low temperatures promoting survival (Hill et al., 1971; Katzenelson, 1978). Essentially all factors that were noted as affecting virus survival in soil also effect survival in water. Winneberger (no date) has taken data that were compiled by Prickett and Cooper (1968) and has presented survival times for nine enteric viruses in different aqueous environments under various experimental conditions. Poliovirus organisms have been shown to survive from 17 days to 6 months in surface waters. It has been demonstrated that infectious hepatitis viruses have survived 1 month in groundwater and more than 10 weeks in infected spring water. Hain and O'Brien (1979) also reported that infectious hepatitis virus was shown to remain infective for 10 weeks in contaminated well water. It is reasonable to expect longer survival times in groundwater than in surface waters because the effects of sunlight are eliminated and temperatures are lower (Keswick and Gerba, 1980).

b. VIRUS REMOVAL

Viruses in septic tank effluent are removed primarily by adsorption onto soil particles and to lesser extents by bacterial enzymatic attack and natural die off (Sproul, 1975). Some viruses that are adsorbed to large solids may be removed by entrapment as well. Adsorption is not irreversible. It is dependent on soil composition; presence of cations, soluble organics, and clay; pH; and flow rate. Desorption may occur as these variables are altered, thus, the term "removal" may be inappropriate (Wellings, 1979).

The efficiency of soils for virus adsorption is closely related to soil composition. Viruses are negatively charged and adsorb readily to clays and to some silty soils, under appropriate conditions. Sandy loams and soils that contain organic matter also are favorable for virus removal. In comparison,

sands or gravel are coarser materials and have relatively smaller surface areas per unit volume or weight and are not as efficient in removing viruses (World Health Organization 1979). Good virus removal has been shown in sands, but other factors such as loading rate (University of Wisconsin, 1978), temperature, or soil conditioning (Green and Cliver, 1974) become more critical than soil texture.

Yeager and O'Brien (1979) compared the virus adsorption capacity of sand and sandy loam. The sandy loam soils, which had significantly higher clay content, cation exchange capacity, and organic matter and a lower pH, adsorbed more viruses than did the sand. These results and the importance of these soil characteristics are in agreement with other published reports (Bitton 1975; Gerba et al., 1975b).

Soil pH and cation concentration both have been shown to affect virus removal in other laboratory studies. Drewry and Eliassen (1968) conducted batch studies on five soils and found that virus adsorption increased with decreasing pH, in the range of 6.8 to 8.8, and increased with increasing cation concentrations. They indicated that decreasing the pH of the soil reduces the negative charge and thus decreases repulsion between the viruses and the soil. It also has been noted that while low pH favors adsorption, high pH can result in elution of adsorped viruses (World Health Organization, 1970).

Carleson et al. (1968) found that high concentrations of cations usually enhanced virus adsorption. Cations present in soil solution do have a profound effect on reducing the repulsive potential of soil and virus particles. This leads to a formation of a soil-cation-virus bridge and the virus is immobilized. Aluminum ions are more effective in the removal of viruses from percolating wastewater than calcium ions, which in turn are more effective than sodium ions.

Lance et al. (1976) conducted column studies using attenuated polioviruses. Most viruses were adsorbed in the top two inches (five cm) of the soil in the columns. Flooding the column surface for 27 days with virus-laden wastewater did not exhaust the capacity of this top layer to immobilize viruses. The effects of rainfall were simulated by flooding the columns with distilled water within 24 hours after virus loading had ceased. This caused the viruses to desorb and to move through the eight-foot columns. If the distilled water was applied more than 24 hours after infiltration of the effluent, viruses were not desorbed. Apparently, both the time factor and the low ionic strength of the distilled water affected desorption. The authors concluded that viruses would not penetrate to groundwater unless rains were heavy, the water table was near the surface, or the soil was very coarse or shallow.

Investigators on the Small Scale Waste Management Project at the University of Wisconsin (1978) concluded from their own laboratory studies and from a review of the literature that the most important factor in virus retention by soil is flow rate. They found that in systems operated under saturated or rapid flow conditions, a smaller percentage of the applied viruses was removed when compared to columns that received moderate loading rates, which reduced virus levels by several orders of magnitude. The more saturated

the soils, the less opportunity there is for virus contact with surfaces to which it can adsorb. Their summary of findings also includes:

- sandy soils without structure and loaded at less than or equal to five cm/day should remove viruses adequately within two feet;
- structured soils with no clogging mat present would require lower loading rates to achieve purification within two feet;
- loading rates are less important in clogged soils where the infiltration rate is limited by the clogging mat, unless surfacing of effluent occurs saving to excessive application.

It is extremely difficult to specify the minimum depth of soil necessary for virus removal. Researchers at the University of Wisconsin recommend at least two feet of sand and low application rates. Another source generalized that soil with reasonable amounts of silt and clay are effective in removing virus in less than two feet of soil (State of California, 1978). But as has been indicated, a number of variables can influence the removal efficiency of soils. The problem becomes more complex when one considers that viruses retained near the surface may be eluted and may migrate as a result of heavy rainfall or septic tank usage (Lance et al., 1976). It also has been reported that viruses can be desorbed in a low ionic environment and in the presence of organic matter without loss of infectivity (Gerba and Schaiberger, 1975).

c. VIRUS MIGRATION

Many of the specific factors that affect virus movement in groundwater have not been studied directly. These hydrogeological, biological, and meteorological factors (Keswick and Gebra, 1980) have been identified in previous sections. Although some indication of the factors that promote or are detrimental to viral persistence and movement can be determined in controlled laboratory studies, it often is not possible to generalize to field conditions where numerous site-specific variables may intervene. Most of the evidence for migration of viruses has been obtained from field investigations in which viruses have been isolated from drinking water wells in the vicinity of septic tank systems or from monitoring wells that intercept groundwater beneath wastewater land application sites. Review of these studies provides some information about the conditions under which viruses have been known to enter groundwater systems. At present, this knowledge of viral movement is incomplete and sometimes contradictory.

Mack et al. (1972) reported isolation of poliovirus from a 100-foot drinking water well that was responsible for a gastrointestinal outbreak at a restaurant in Michigan. It was postulated that some other, unidentified virus was probably the causative agent in the outbreak. The source of enteroviruses was a septic tank drainfield that was located 300 feet from the well. The viruses had to penetrate 18 feet of clay, 8 feet of shale, and 74 feet of limestone to reach the well. Very low levels of coliform bacteria were detected, however, no pathogenic bacteria were isolated in water samples.

An outbreak of infectious hepatitis at Posen, Michigan, was attributed to septic tank effluent. Only a few feet of soil separated the bottom of the drainfield and the highly permeable limestone aquifer, which was the source of drinking water for the area. Percolating water from melting snow and spring

rains carried the virus-laden effluent into groundwater, which was withdrawn by nearby drinking water wells. Other areas in the Great Lakes Region have experienced similar groundwater contamination as a result of malfunctioning septic tanks or from systems located in unsuitable soils (Weist, 1978).

An outbreak of 51 cases of gastrointestinal illness and 15 cases of hepatitis A at a migrant labor camp in Florida was reported by Wellings et al., (1975a). Echovirus was detected in 35 to 40-foot drinking water wells. One well was located 100 feet from a solid waste disposal area and five others were located in an area bordered by septic tank systems. The outbreak occurred during a dry spell and the investigators reasoned that the heavy pumping had increased the groundwater gradient and flow rate down from the septic tanks toward the wells. Viruses survived in chlorinated water and existed in the absence of bacterial indicator organisms.

Baer et al. (1977) reported an outbreak of 50 cases of hepatitis A, 26 with jaundice, that occurred in a rural Alabama school. The school obtained its drinking water from two small springs that received seepage from septic tanks. The investigators noted that periods of heavy rainfall, which probably aided in the transport of the virus, had preceded the onset of the outbreak.

These studies demonstrate that soil type and structure, proximity of well to source of sewage, rainfall, and other factors can affect viral movement from septic tank systems into groundwater and that viruses can persist in chlorinated water and in the absence of indicator organisms. These investigations, however, were prompted by disease outbreaks; reports of negative results of tests for viruses in groundwater near septic tanks are not as likely to be published. Also, private water supplies located near on-site disposal systems normally are not monitored routinely and viruses could only be detected present in domestic wastewater if someone is infected. It is likely that most operating soil absorption systems effectively remove enteroviruses from septic tank effluent.

Additional evidence for migration of enteroviruses can be obtained from reports of land application and groundwater recharge projects. It should be noted, however, that sites are generally selected with sandy soils that permit rapid percolation but may also have the least capacity for virus removal. This is especially true for recharge programs. Wellings et al. (1974) recovered virus from groundwater beneath a wastewater irrigation site. Viruses survived secondary treatment, including chlorination, and exposure to sunlight and still penetrated through 10 to 20 feet of sandy soil. Detection of virus in groundwater occurred after heavy rains.

Wellings et al. (1975b) later demonstrated lateral movement of enteroviruses in groundwater after discharge of secondary sewage effluent into cypress domes, which are ponded continuously. Viruses were shown to move from the application point through alternating sand and clay layers to a depth of at least 20 feet and over a horizontal distance of at least 125 feet to where the monitoring wells were located. Viruses survived up to 28 days in this study. Again, isolations generally followed heavy rains.

Other investigators have demonstrated even greater virus travel distances in groundwater. Vaughn et al. (1978) detected viruses in groundwater 35 feet beneath wastewater recharge basins at installations located on Long Island,

New York. They also showed horizontal transfer of up to 150 feet. Viral concentrations in observation wells at the groundwater recharge sites ranged up to 10.6 PFU/gallon. Schaub and Sorber (1977) reported viral migration distances at a rapid infiltration site in Massachusetts. The soil at the site consisted of unconsolidated silty sand and gravel of glacial origin over bedrock. Groundwater contained viruses at depths of 95 feet and at lateral distances of 600 feet from the point of application. These studies indicate that entrainment in aquifers can result in significant lateral movement of viruses.

Other rapid infiltration projects, however, have produced renovated wastewater free of viruses. For example, throughout the 10 years of operation of the rapid infiltration project at the Flushing Meadows site in Arizona, viruses were never detected in the renovated wastewater wells. The average virus concentration of the applied secondary effluent was 2,118 PFU/100 ml (Bouwer et al., 1980). At this site, renovated wastewater has completely replaced native groundwater. Viruses also were not detected in the renovated water sampled below the Whittier Narrows and the Santee, California, sites nor the San Angelo, Texas, installation. These success stories tend to indicate that, under certain conditions, the soil can be an effective medium in removing viruses from wastewater and in limiting the potential impact on groundwater quality.

3. FATE OF NUTRIENTS DISCHARGED FROM ON-SITE SYSTEMS

The nutrients that have the greatest potential for creating environmental or human health problems are phosphorus (P) and combined forms of nitrogen (N). Adverse effects may arise if excessive amounts of either of these nutrients enter nearby lakes or streams or if nitrogen reaches an aquifer that is used as a source of drinking water. High levels of nutrients, especially phosphorus, may accelerate eutrophication of surface waters because they are essential nutrients for algae and aquatic weeds. Algae growth and oxygen depletion may result, which in turn may create conditions that are aesthetically unpleasing or that are detrimental to aquatic fauna and flora.

Some forms of nitrogen present in drinking water pose potential health hazards. Nitrate (NO_3), the end product of nitrification, and nitrite (NO_2), the intermediate form, can be toxic to humans if ingested in large amounts (University of Wisconsin, 1978). Infant methemoglobinemia has been linked to these nitrogen forms. Also, there is evidence that nitrosamines, which are formed from the reaction of NO_2 with secondary amines, are carcinogenic to laboratory animals. A limit for nitrate of 10 mg/l as nitrogen in drinking water has been recommended.

Phosphorus and nitrogen normally are present in household wastes and thus are present in septic tank effluent. In most properly functioning septic tank soil absorption systems, phosphorus, will be immobilized readily in the soil; thus, phosphorus enrichment of groundwater seldom occurs beneath these systems. The same system, however, is not as effective in nitrogen removal. Most, if not all, of the nitrogen that leaves the house will eventually be converted to nitrate. Nitrate is not retained by soils, and it will be carried with the effluent into the groundwater. Dilution by uncontaminated groundwater is then the only significant mechanism of reducing NO_3 concentrations to safe levels (University of Wisconsin, 1978).

Many field and laboratory studies have been conducted to investigate under what conditions phosphorus and nitrogen from conventional on-site systems may enter ground and surface waters and degrade water quality. These investigations are reviewed in this section.

a. Phosphorus in Household Wastewater

Most of the phosphorus that is present in domestic wastewater originates from washing activities that use phosphorus-based cleaning agents. The relative amount from this source has decreased since limits on phosphorus detergents have been imposed. Phosphorus also is contributed by toilet usage. Phosphorus is present in organic and polyphosphate forms, which are converted to soluble orthophosphate.

Several investigators have conducted chemical analyses of septic tank effluents to determine the phosphorus concentrations that reach soil absorption systems and to compare these levels with the concentrations found in groundwaters adjacent to the systems. Values that are reported show a good deal of variability, which probably results from differences in the type and amount of waste products present and the wastewater flow, age of the system, and sampling and analytical procedures (Dudley and Stephenson, 1973). Otis et al. (1975) monitored effluent concentrations from six septic tank systems over a 2-year period and found average effluent concentrations of total phosphorus to range from 11.0 to 31.4 mg/l with a median value of 12 mg/l. Dudley and Stephenson (1973) have compiled literature values and report a wider range of average total phosphorus concentrations from 5.4 to 70 mg/l with a higher mean of 25 mg/l.

b. Phosphorus Removal

Phosphorus is removed from septic tank effluent and retained in the soil by adsorption and precipitation reactions. Adsorption is controlled primarily by the mineralogy of the soil and to a lesser extent by soil particle size (Jones and Lee, 1979). Clay minerals have relatively higher capacities for phosphate adsorption than do coarse sand or gravel (Hook et al., 1978). Iron, aluminum, and calcium cations are involved in phosphorus reactions. In neutral to acid (non-calcareous) soils, the phosphate ion becomes adsorbed onto surfaces of iron and aluminum compounds, while in neutral to alkaline (calcareous or limestone) soils, calcium minerals predominate in adsorption (University of Wisconsin, 1978).

As the concentration of phosphorus increases, precipitation reactions may occur if the soil contains sufficient quantities of a cation that would form an insoluble compound with the phosphate anion. At normal septic tank effluent pH levels, phosphate precipitates of calcium, in the form of hydroxyapatite, usually result (Hook et al., 1978). In hardwater areas, phosphorus is precipitated readily because of the calcium carbonate that is present in the soil and groundwater (Jones and Lee, 1977). Aluminum or iron phosphate also may occur but the pH of the effluent is not as likely to favor these reactions except during the first few months of system operation (Hook et al., 1978) or if the system is located in non-calcareous soils (University of Wisconsin, 1978).

Many field studies have demonstrated that most soils, even some sandy soils, are very efficient in reducing phosphorus concentrations before effluents reach groundwaters. Reductions of 80% to 90% of the applied phosphorus loading within a period from two to five days have been reported by Tofflemire et al. (1973 as cited in Jones and Lee, 1977). Distances required for comparable reductions range from a few inches (Miller and Wolf, 1975) to about 15 feet in sandy soils with some silt or clay (Dudley and Stephenson, 1973). Generally, phosphorus moves very slowly through the soil. If the phosphorus is not removed immediately below the soil absorption field, there is evidence that it can be removed by soil and sediment within the groundwater (Childs, 1974). Phosphorus is normally present in very low concentrations in naturally occurring groundwaters with a typical range of from 0.01 to 0.06 mg/l P (Jones and Lee, 1977).

In some cases, however, the system may not be able to immobilize permanently all the phosphorus to which it is exposed. System overloading may result from applying too much phosphorus or by applying it too rapidly. Even under normal operation, the absorptive capacity of a soil decreases with the age of the system as exchange sites for phosphorus become saturated (Dudley and Stephenson, 1973). There is also the possibility of leaching. Phosphorus retained by soils can be redissolved and transported by fluctuating water tables or by exposure to waters of low phosphorus concentrations. In these situations, the movement of phosphorus in subsurface soil layers is likely, depending on the application rate, percolation rate, and soil pH (Sawhney and Starr, 1977). Temperature and other chemical and biological variables are involved also (Hall, 1975). This potential for movement is of particular concern at lakes where distances between septic tank systems and lakeshores may be limited and phosphorus movement into the lake may be rapid.

Although there is a finite capacity for soils to absorb phosphorus, it has been demonstrated that resting the system regenerates absorption sites and that the soil is capable of removing additional phosphorus from wastewater over longer periods. In a field study by Sawhney and Starr (1977), a six-month rest period was sufficient to regenerate the absorption capacity. The septic tank system that was used in the study had two parallel absorption trenches that could accept the effluent; most conventional septic tank systems, however, do not have this unique feature.

When subsurface disposal systems are installed with proper attention given to soil characteristics and local hydrogeological factors and when they are located at adequate distances from receiving water bodies, the complete removal of phosphorus from septic tank effluent by soil generally is possible. Because many systems currently are operating without satisfactory removal efficiencies, they represent good candidates for evaluation. The results of these studies aid in the understanding of which factors adversely affect removal efficiences and allow the movement of phosphorus into groundwaters or nearby lakes and streams.

c. Phosphorus in Waters Near On-Site Systems

Because of the potential for adverse effects on surface water quality resulting from groundwater transport of phosphorus from septic tanks, a number of field investigations have been undertaken to determine the factors that affect phosphorus movement. Many of these studies have been conducted in the

Region V states where a large number of rural lakes are located and where accelerated eutrophication of these lakes is a potential problem. Phosphorus contamination is evidenced by elevated concentrations in groundwaters below septic tank systems or in nearby surface waters; it is inferred by shoreline algal growth on lakes with rural residential development. Lawn or garden fertilization on lakeshore home sites, however, is a confounding variable.

Bouma et al. (1972) measured orthophosphate (PO_4) concentrations in groundwaters below five conventional septic tank systems located in sands in central Wisconsin. Groundwater adjacent to two of the five sites had relatively high phosphorus concentrations. One system was 12 years old and was located in loamy sand soil. The other system was 8 years old, was located in medium to fine sand and peat, and occasionally was flooded by a fluctuating water table. At this site, phosphorus was transported by groundwater into a lake 90 feet away. Phosphorus concentrations equal to background levels (0.02 to 0.03 mg/l PO_4 P) were found beneath a system that was less than one year old and another one that had an intervening clay layer between the system and the water table, which was about 20 feet below. These results indicate that system age, soil type, and location of water table can affect phosphorus transport.

Eleven more existing on-site disposal systems in Wisconsin were evaluated for phosphorus removal efficiency by Dudley and Stephenson (1973). Phosphorus removal was most effective at sites that were underlain by medium to fine grained sands or in sands with some silt or clay. The authors concluded that at sites with these characteristics, total phosphorus levels should be at background concentrations within 10 to 20 feet of travel from the point of release. Low phosphorus concentrations also were found at sites with relatively new systems. Significant levels (> 1.00 mg/l total phosphorus) were found at four sites. Three of these systems were underlain by medium to coarse grained outwash sands and gravel with high permeabilities. The other site was situated in very impermeable soils derived from glacial till. The unsaturated zone beneath these systems ranged from 7 to 56 feet. Based on their study results as well as on a review of the literature, Dudley and Stephenson (1973) concluded that phosphorus enrichment of groundwaters could occur where septic tank soil absorption systems are located above high water tables or in coarse sandy subsoil, receive heavy loadings, and are relatively old. Although elevated phosphorus concentrations were found beneath some absorption fields, in at least one of these sites the groundwater flow was not toward the lake, so it is unlikely that the added phosphorus load from this system would directly degrade the lake water quality. The investigators suggested that the greatest potential for phosphorus reaching lakes from lakeshore on-site systems would occur in poorly drained soils with high water tables or in coarse sands with naturally low absorptive capacities for phosphorus.

Childs (1974) conducted a three-year waste migration study of septic tank effluents around Houghton Lake, Michigan. Results from three of the study sites indicate the importance of the chemical composition of the soil in phosphorus removal and document vertical and horizontal travel distances. One septic tank system (site 1) was located in medium to coarse soil (Rubicon sand). The water table was five to six feet below the ground surface and sloped toward a canal that eventually fed into the lake. The system had been used heavily by a family of six for 15 years. Background phosphorus concen-

trations averaged 0.02 mg/l PO_4 - P. Although present in fairly low levels (0.1 to 0.3 mg/l), orthophosphate still was detected at distances of 100 feet laterally and 4 feet vertically from the system. Groundwaters below two other systems (sites 4 and 7) showed higher phosphorus concentrations and further travel distances. These systems were situated in fine-grained soils (Newton loamy sand) and just two feet above the water table. Both systems had been in use by families of six for about 10 years. High phosphorus levels extended 6 feet into the groundwater and values as high as 8 to 14 mg/l were measured at the lake's edge 120 to 150 feet away. Directly offshore from one of these sites, aquatic plants were observed; groundwater flow direction at the site was toward the lake. A water sample from the anomalous weed bed had a concentration of 1.09 mg/l; mean lake values ranged from 0.03 to 0.08 mg/l.

These study results are in contrast with other studies that have indicated that fine-sized soils are more efficient for phosphorus removal than are coarser soils because of the greater surface areas and retention times. Childs (1972) suggests that the chemical makeup of the soil and sediment is more important than particle size alone in phosphorus retention. Soils at site 1 were rich in calcium, aluminum, and iron cations, which formed insoluble compounds with phosphorus and thus were more favorable for adsorption. There also was a greater depth of soil between the system and the water table at this site than at the others.

Childs (1974) also estimated the quantity of phosphorus from on-site systems that was transported annually to Houghton Lake by groundwater. It was estimated that residents discharge 40,000 pounds of phosphorus per year through septic tank systems but that only 1000 pounds reach the lake; this represents a removal of 98%. He also determined that the phosphorus adsorption capacities of soils above the water table in the area ranged from six months to six years. Because the phosphorus was not present in the local groundwaters and only limited amounts could be absorbed above the groundwater, Childs concluded that phosphorus must be retained by soils and sediments in the groundwater. He further concluded that the adsorptive capacity of soil may be as great under saturated as under unsaturated conditions and that phosphorus is not available for groundwater transport until after the soil and sediment filter is saturated with phosphorus.

Sawhney and Starr (1977) also investigated vertical and lateral movement of phosphorus from a septic tank drainfield. They found that movement occurred in both downward and horizontal directions and that phosphorus concentrations were similar at equal distances below and beside the trench. Background phosphorus concentrations were evident at a depth of two feet. The researchers concluded that in a soil with a deep water table, a distance of two feet should effectively remove phosphorus, but that more distance would be required for removal in shallow soils with high or perched water tables. No description of the soil texture was given.

Other investigators have not found favorable phosphorus removal efficiencies under saturated flow conditions or in the saturated zone. Peavy and Jones (1977), in their evaluation of a septic tank drainfield in a clay loam and silty clay loam soil, noted that although very little phosphate reached the groundwater, once the effluent reached the aquifer, apparently no substantial removal occurred. Reneau (1979) observed greater phosphorus movement in both fine and coarse loamy soils under saturated flow conditions

because of the lower effective soil volume that was available for phosphorus removal. In unsaturated flow conditions during low water table levels, he found that phosphorus adsorption was enhanced as a result of the increased contact and the availability of larger surface areas.

Gibbs (1977) also evaluated the effect of a fluctuating water table on phosphorus removal efficiency of lakeshore soils. The investigation was conducted at Lake Taupo, New Zealand, where increased draw-off of lake water for electricity generation simulated a fluctuating water table. During the summer high lake levels, the pumice soils removed only 24% of the effluent phosphorus, whereas at the lower lake levels, during the autumn and winter, the removal efficiency was 74%. At the lower groundwater and lake levels, more soil is exposed for purification. When this soil becomes submerged in water of low phosphorus concentration as the water table rises, the nutrients could be flushed out, leaving the soils with a higher removal efficiency for the next exposure. The cyclic flushing, however, provided a mechanism by which an estimated 80% of the discharged effluent reached the lake. High phosphorus levels were found in the lake bottom bed and decreased with distance from the shore. Macrophyte weeds were observed on the banks of some bays off the lake.

In summary, inefficient removal of phosphorus in a system generally implies the inability of the immediate soil area to reduce the concentrations to background levels or to prevent the movement of phosphorus into surface waters. Generally, finer-textured soils are better for removal than coarser soils because of increased particle surface area and contact time under unsaturated flow conditions. Good removal, however, has been obtained with sandy soils, particularly if there is some silt or clay. The chemical properties of the soil may be as significant, or more so, as particle size alone. Soils rich in aluminum, calcium, or iron oxides enhance phosphorus adsorption and precipitation reactions.

The literature is not definitive on the depth of soil that is necessary for reduction of phosphorus concentrations in septic tank effluent to background levels. Generally, the greater depth available between the bottom of the drainfield and the water table or bedrock, the greater protection against phosphorus contamination of underlying groundwaters. This is especially important in poorly drained soils or in soils with naturally low adsorptive capacities for phosphorus. In many situations, a separation distance of two to four feet of soil will be adequate.

In general, phosphorus removal from percolating wastewater is enhanced under unsaturated flow conditions more than in saturated regimes. This is evidenced by removal efficiencies in soils with fluctuating water tables, with greater efficiencies occurring when the water table is low. There is the indication that although a rising water table can regenerate absorption sites by redissolving the absorbed phosphorus, this flushing action also results in the movement of phosphorus. Depending on the nature of the soil and groundwater, additional removal may occur in the water table.

Removal efficiency is decreased while the potential for migration is increased as the phosphorus absorption sites become saturated. This can occur from sustained heavy loading or with system age. Although a soil has a finite capacity for the adsorption of phosphorus, through regeneration mechanisms

induced by wetting and drying cycles or by alternating absorption beds, the capacity of a soil can be prolonged.

Most of the studies involve individual systems that are isolated from other sources of contamination in order to describe the behavior of phosphorus from septic tank systems. Lawn fertilizers are a source of phosphorus too. Thus, any algae growth on the immediate shoreline from a household with a septic tank may be a result of the fertilizers or effluent, or both. In rural lakeshore communities, along with the hydrogeologic factors, septic tank densities, proximity of units to the lake, and groundwater flow patterns all become involved in whether the lake will be adversely affected.

d. Nitrogen in Domestic Wastewater

The major sources of nitrogen in domestic wastewaters are feces and urine, which contain urea, uric acid, ammonia, undigested foodstuffs, and bacterial cells (Sikora and Corey, 1976). In the anaerobic environment of the septic tank, nitrogen is mineralized rapidly to ammonia by microbial enzymes. The nitrogen in septic tank effluent is about 75% ammonia (NH_4) and 25% organic nitrogen (Otis et al., 1975). Average effluent concentrations from six septic tank systems had ranges of 32 to 76 mg/l for total nitrogen and from 20 to 46 mg/l for ammonia. Combined nitrate and nitrite concentrations in the analyzed effluent samples were less than 1 mg/l.

e. Nitrogen Transformations and Removal Mechanisms

The organic nitrogen in septic tank effluent is dissolved or suspended. Some is removed from solution by biological decomposition into simple forms and then adsorbed by soil particles and clays (Dudley and Stephenson, 1973). The rest of the organic nitrogen is mineralized to ammonia and these undergo the same reactions discussed below.

In a properly functioning system, aerobic conditions normally will prevail in the soil underlying the absorption field, but if the soil is submerged or oxygen is depleted, anaerobic conditions are favored. In such cases, ammonia could be leached to the groundwater before nitrification could take place. The amount that would reach the water table would depend on the number of cation exchange sites exposed to the effluent, the affinity of these sites for ammonia, the composition of the effluent, and the degree of cation exchange saturation with ammonia (Sikora and Corey, 1976). A more detailed description of the conditions that interfere with the formation of nitrates is provided in the next subsection.

The primary transformations of nitrogen that occur in the soil are nitrification and denitrification. Nitrification, or the process by which ammonia is oxidized first to nitrite and then to nitrate, is accomplished by bacteria that require aerobic conditions. As previously noted, aerobic conditions normally exist in the unsaturated zone below a drainfield and thus the formation of nitrates is very likely. This is of concern because of the public health and pollution hazards that are posed by excessive concentrations of nitrates in groundwater. Nitrification generally occurs rapidly and within a short distance of the drainfield. The point at which nitrification of the ammonia is complete will depend on the contact time and the rate of nitrification, which in turn is affected by pH, aeration and moisture conditions, number of nitrifiers, and temperature (University of Wisconsin, 1978).

The amount of nitrate that is removed by plant uptake is minimal. This mechanism is evidenced by the characteristic lush growth often present near septic tank systems. The quantity of nitrogen that is discharged from a septic tank far exceeds what can be used by plants (Sikora and Corey, 1976).

Some removal of nitrate is feasible through denitrification, the microbiological process in which nitrate and residual nitrite can be reduced to nitrous oxide (N_2O) or free nitrogen (N_2). For NO_3 and NO_2 to be present, nitrification must occur first. Denitrification of oxidized nitrogen is desirable but it can occur only in an oxygen deficient environment and in the presence of a carbon energy source (Miller and Wolf, 1975). Anaerobic conditions can exist beneath an aerobic zone if prolonged periods of saturation have occurred. For poorly drained soils or slowly permeable soils, which may enhance saturated flow and hence anaerobic conditions at depths in the soil profile, the loss of nitrates through the denitrification process may be significant. However, in most cases it is unlikely that both of these reactions will occur at a site because of the opposite aeration requirements of each (Sikora and Corey, 1976).

If nitrates are not removed either by plant uptake or through denitrification, they likely will reach groundwaters and possibly surface waters. This is because nitrate is a soluble anion and the cation exchange capacity of soil is ineffectual in the adsorption of nitrate as it moves with the percolating effluent (Miller and Wolf, 1975). Once the nitrate has reached the water table, the only mechanism by which nitrate concentrations are reduced is through dilution.

Nitrogen can be introduced into the environment by sources other than septic tanks. This further increases the potential for nitrate contamination of ground and surface waters. Lawn and garden fertilizers, animal feedlots, and barnyards are all sources of nitrogen. Decomposing vegetation, accompanied by climatic and land use patterns, can supply nitrates and can lead to their subsequent migration into groundwaters as well (McNabb et al., 1977).

f. Factors That Affect Nitrogen Transformation and Movement

Although nitrates normally are found in excessive amounts a short distance below absorption fields, conditions may be present initially that would prevent nitrification from even occurring. This would result in high ammonia concentrations in groundwaters downgradient from septic tank systems. Dudley and Stephenson (1973) found ammonia concentrations of more than 10 mg/l at two of their eleven study sites in Wisconsin that were both situated in impermeable glacial till soil, which apparently did not allow the proper aeration for nitrification to occur. Elevated concentrations also were found occasionally at other sites in sandy soils with water tables less than five feet below the absorption fields. In all their study sites, ammonia concentrations averaged 15 mg/l below the drainfields and never exceeded 1 mg/l at horizontal distances greater than 50 feet, thus, the ammonia is apparently attenuated fairly quickly by soil adsorption.

Evaluation of existing septic tank systems by researchers at the University of Wisconsin identified other conditions that may lead to high ammonia levels. In one system that was used intermittently, the effluent percolated rapidly, so nitrification did not always occur. Other findings

from these studies indicate that high ammonia concentrations will be present beneath sites where the depth of the aerobic zone between the drainfield and groundwater is less than two feet. Seasonal differences also have been noted. During the winter ammonia levels were observed to increase because the lower temperatures slow the rate of nitrification and thereby increase the distances needed for complete nitrification. Under conditions of saturated flow, very little nitrification takes place and ammonia concentrations are high. Viraraghavan and Warnock (1975) found higher ammonia values in the winter when low temperatures and high water tables existed concurrently.

Although it may appear to be desirable to prevent the conversion of ammonia to nitrate by installing septic tank drainfields in line with the previously identified factors, many of those same factors may have other equally bad or worse side effects. For example, an absorption field that is located a few inches above a water table may not provide sufficient soil volume for nitrification, but at the same time neither would it provide for pathogen removal. Also, the addition of ammonia to surface waters may exert considerable demands for oxygen. Thus, there are obviously many trade-offs involved in the renovation of septic tank effluents.

Several investigators have reported that when conditions for nitrification are present, the oxidation of ammonia to nitrate is rapid and complete. Walker et al. (1973a, 1973b) concluded that nitrification will occur readily if three to six feet of unsaturated soil separate the drainfield from the water table. In an investigation of five study sites in permeable sandy soils in central Wisconsin as part of the Small Scale Waste Management Project, nitrification was generally found to be complete within two to six inches below the clogging mat. Two other studies, which are referenced in a literature review by the University of Wisconsin (1978), indicate that nitrification is complete within one to two feet (Preul and Schroepfer, 1968; de Vries, 1972). This finding also was supported by Dudley and Stephenson (1973).

As noted previously, nitrate may move extended distances, with dilution being the primary mechanism involved in decreasing its concentration. Studies that are cited in the University of Wisconsin report (1978) have demonstrated that from 50 to 100 feet of travel in groundwater may be required to lower nitrate concentrations in septic tank effluent to below 10 mg/l (Preul, 1966; Dudley and Stephenson, 1973). Concentrations above background levels have been observed 330 feet downgradient from septic tank systems, with the highest concentrations several feet below the water table level rather than at the surface (Ellis and Childs, 1973).

Other investigators, as a result of their case studies of nitrate contamination, have attempted to specify septic tank or population densities that are necessary to accomplish acceptable dilution in an unsewered community. Walker et al. (1973) indicated that septic tank systems would not contribute significant amounts of nitrates to groundwaters in rural watersheds when the density of residences is less than one per 0.5 acre. Lower densities in other areas, however, have been associated with nitrate contamination. For example, Woodward et al. (1961), in their survey of 39 communities in Minnesota, found that when population densities were 2.7 persons per acre, 29% of the wells experienced nitrate contamination, but when densities were 0.5 person per acre, only 2% of the wells were contaminated. Insufficient dilution of

nitrates from septic tank systems were found in Nassau County, Long Island, where population densities exceeded three persons per acre (Smith and Myott, 1975, as cited in Hook et al., 1977).

These investigations have significant implications for rural lake communities where higher population or septic tank densities are common and distances to water supply wells and lakeshores are often short. Although septic tank density is one of the major factors affecting nitrate contamination of a local area, there are many other site limitations and hydrogeological conditions that also influence nitrate pollution in groundwater (Scalf et al., 1977; Schmidt, 1977). Several of these other factors have been identified in case studies of regional or areal nitrate contamination of groundwaters.

g. Cases of Nitrate Contamination

One early demonstration of widespread groundwater contamination in unsewered areas in Minnesota was given by Woodward et al. (1961, as cited in Dudley and Stephenson, 1973). They sampled 63,000 private wells in 39 communities; 48% of the wells sampled had significant nitrate concentrations and about 11% of the 48% (5.3% of the total) values greater than 10 mg/l. Several factors were identified to explain the variability among communities and the elevated concentrations. Soil type was considered important; no contamination was apparent in areas where a continuous clay horizon was present. Increases in well depth led to improvement in measured water quality. Of the older systems that were installed prior to World War II, 50% of the wells were contaminated with nutrients and coliform bacteria. Only 10 to 20% of the newer systems were contaminated. As already indicated, population density also correlated with nitrate contamination.

Delfino (1977) referred to a statewide survey of approximately 6000 private wells in rural areas throughout Wisconsin that was conducted from 1968 to 1972 by Schukrecht et al. (1972, 1975). Of those wells sampled, about 550, or 9.2%, had nitrate concentrations greater than 10 mg/l. An intensive survey also was conducted in Columbia County in south-central Wisconsin. Shallower wells tended to have higher nitrate concentrations, as did wells located close to known pollution sources. Of 33 wells tested that were located within 50 feet of sewage seepage areas, 27% had concentrations greater than 10 mg/l. Proximity to barnyards, feedlots, and manure storage piles, however, resulted in many more contaminated wells than did locations near septic tanks.

Brooks and Cech (1979) conducted a nitrate and bacteria survey of domestic water supplies in a rural east Texas community. Sands are the predominant soil type in the area. Fifty-three privately owned wells that were less than 350 feet deep were sampled. The wells with the highest observed concentration of nitrates were those located close to septic tanks. When set back distances were less than 15 feet, concentration of nitrates exceeded 10 mg/l; and when wells were located within 50 feet of the septic tank systems, nearly 50% of them had concentrations greater than background (0.9 mg/l). Elevated nitrates were found in more dug wells, rather than drilled, or in wells that were less than 50 feet deep. The authors concluded that the major cause of high nitrate levels was the improper location of septic tanks and wells with respect to each other, which apparently created a direct recycling of drained fluid from septic tanks to wells.

Two unsewered communities in the coastal plain of Delaware were investigated for the causes of nitrate contamination in private water supplies (Miller, 1975). One of these areas was characterized as having an extremely high water table, poorly drained and periodically waterlogged soils, and occurrences of overflowing septic tank systems during rainy months. Low nitrate levels were found in private water wells, and concentrations decreased with rising water tables. It was hypothesized that nitrogen compounds were never oxidized to nitrates but remained as ammonia or organic nitrogen in the groundwater. The same hydrogeologic factors, however, contributed to the high coliform bacteria levels that were measured in the wells.

In the other area, soils were deep and well-drained. More than half the samples collected from shallow wells had concentrations greater than 10 mg/l. Nitrate concentrations ranged from 5 to 30 mg/l, thus indicating that the permeable soils and low water table favored the oxidation of nitrogen compounds to nitrates and their movement into the water table. Pathogenic organisms were removed successfully by the soils. Miller (1975) concluded that when the water table is generally less than five feet, bacterial contamination from overflowing absorption fields and septic tanks is likely. When it is greater than five feet, nitrate contamination is possible.

More recently, Robertson (1979) analyzed 800 groundwater samples from wells in Sussex County, Delaware, and found that more than 20% of the wells had nitrate concentrations greater than 10 mg/l. The greatest incidence, however, was associated with animal feed lots. Septic tank effluent also was considered a significant source of nitrates, as were fertilizers, forest foliage, and precipitation.

Because of the public health concern regarding the consumption of excessive amounts of nitrates, relatively more attention has been devoted to documenting its presence in drinking water wells than in surface waters. One study, however, noted a gradual increase in nitrate concentrations in both ground and surface waters over a 30-year period that was attributed to the presence of sewage effluents from septic tank drainfields (DeWalle and Schaff, 1980). River water samples in the 104-square-mile study area, which was south of Tacoma, Washington, had a mean nitrate value of 1.3 mg/l and a maximum value of 8.6 mg/l. Mean nitrate concentrations of lakes in the area were 0.36 mg/l with maximum winter values of 2.4 mg/l. Phosphorus concentrations during the winter similarly were elevated and the investigators noted that substantial algae blooms resulted every summer. Although nitrate concentrations in excess of 10 mg/l may have a health significance, much lower concentrations may contribute to eutrophication of surface waters (Peavy and Groves, 1978).

4. OTHER CHEMICAL CONTAMINANTS

There are other chemical contaminants that occasionally could be present in domestic wastewaters. Because of the eutrophication effects of phosphorus and nitrogen and the toxic nature of nitrogenous compounds in drinking water, these contaminants were discussed separately. The intent of this section is to address briefly the type and extent of water quality impacts that could result from the inadvertent or intentional discharge of toxic or other chemical pollutants into septic tank systems.

a. Toxic Chemicals

Many synthetic organic chemicals commonly are used in household products, such as pharmaceuticals, disinfectants, deodorants, polishing agents, detergents, cosmetics, paints, and pesticides (Scalf et al., 1977). If the use of these products is not consumptive, then liquid wastes containing potentially harmful chemicals may enter the septic tank drainfield. Under some conditions, these chemicals might reach nearby surface or groundwaters.

Many of the organic substances are physically trapped, chemically changed, or biologically degraded in the aerobic soil layers below properly functioning septic tank soil absorption systems (Brown and Caldwell, 1977). In addition to the properties of the chemical itself, there are other factors that affect removal, particularly the type, structure, and chemical composition of the soil (State of California, 1978). The soil bacteria population also is a factor because these microorganisms are important in the decomposition of organic compounds (Miller and Wolf, 1977).

Although many chemicals are removed successfully, the soil is not an effective medium for removing the more stable soluble chemicals. A review of past studies (Brown and Caldwell, 1977) indicates that stable chemical compounds such as gasoline, phenols, and herbicides have been shown to travel distances from less than 1 mile to more than 20 miles from their disposal or application site. Furthermore, there is some indication that those organic substances that are removed initially may be desorbed later and may not be degraded before they enter ground or surface waters. Thus, exposure to chemical pollutants far from their source of application is possible. Mobility of chemical constituents in septic tank effluent appears to be dependent on properties of the soil, oxygen status of the soil, pH, and characteristics of the contaminant (Brown et al., 1978).

Pesticides often are singled out as especially dangerous pollutants because many of the chemicals that are used in the formulations generally are toxic to many nontarget species, including humans. Their presence in domestic wastewaters would not be expected normally and, if present, their concentrations probably would be very low. Once in the soil, pesticide residues move very slowly downward (Miller and Wolf, 1977). Pesticides are absorbed by clay particles, iron and aluminum oxides, and organic colloids and they are subject to microbial degradation (State of California, 1978). Soil studies that have been conducted under laboratory and field conditions indicate that most pesticides remain in surface soils, while a few with time may penetrate to depths of one to two feet. Even the more mobile pesticides usually are broken down rapidly by soil microorganisms. The ones that pose the greatest potential threat are the mobile pesticides that also are somewhat resistant to decomposition.

The extent to which pesticides will remain in soils after their application depends on several factors, including soil type, moisture, temperature, pH, and microorganism content. It is assumed that these same factors would exert similar influences if the pesticides were discharged to a septic tank drainfield. One of the most important factors is the degree of, and the time required for, the degradation of the pesticide itself into simpler, nontoxic forms (Murphy, 1975), which in turn is partly dependent on the water solubility, molecular, ionic, and volatile character of the pesticide (Miller and Wolf, 1977).

The biggest concern regarding the toxic chemical compounds that are persistent in the environment is their potential hazard to human health if ingested with untreated drinking water from rural water supplies either in large doses over a short time or in small doses over a long time. Incidents of acute chemical contamination generally are easy to detect. Well water may look, taste, or smell unusual and clinical symptoms of an affected individual may appear rapidly. The effects of chronic, low-level doses, however, are harder to predict, or to detect, and the victim's sub-clinical symptoms generally are difficult to diagnose and to attribute to a causative agent (U.S. EPA, 1980). No report of chemical poisoning of drinking water from on-site disposal systems was encountered in the literature that was reviewed.

Likewise, no documented case of an individual septic tank causing chemical contamination of ground or surface waters was reported in the literature reviewed. Although the possibility is remote, environmental hazards could result if groundwater carrying toxic chemicals from rural domestic sources discharge into nearby lakes or streams. The risk of water quality degradation due to toxic materials from rural on-site disposal systems seems minimal for several reasons. First, as noted previously, the presence of harmful chemicals in domestic wastewater is sporadic. Second, if the chemical is subject to degradation, setback distances from receiving waters generally provide adequate time for this removal. Finally, when the groundwater mixes with surface water, dilution reduces the concentration even further. The primary environmental hazard that could result if toxic concentrations of wastes exist for prolonged periods in nearby surface waters would be the adverse impact on the aquatic organisms. A severely toxic substance will eliminate aquatic biota until dilution, dissipation, or volatilization reduces the concentration below the toxic threshold (Mackenthum, 1969). Less toxic materials also might reduce populations of those fish and lower aquatic organisms that are unable to tolerate the concentration of the toxicant. There is no evidence that indicates that septic tank systems have caused these effects.

b. Other Septic Tank Effluent Constituents

The predominant emphasis in the literature on water quality impacts from on-site wastewater disposal systems is on the addition of pathogens and nutrients to ground and surface waters and their associated effects. This is because these constituents are the ones most likely to be present in septic tank effluent and likely to pose human health or environmental hazards. A few researchers, however, also have included other parameters in their investigations of septic tank system performance.

Brown et al. (1978) conducted a relatively comprehensive assessment of an existing system. As part of their study, they measured the concentration of selected metals in lysimeters that were located in sandy, sandy clay, and clay soils that received septic tank effluent. Concentrations were recorded for copper, zinc, nickel, lead, and cadmium. Mean soil concentrations for these metals were highest in the clay soil and were lowest in the sandy soil. In some cases, the concentrations in the soil adjacent to the septic fields increased to about two times above the background levels, but these concentrations were still much lower than what would be required to reduce plant growth. Periodic groundwater samples did not indicate the presence of these metals above the background. The investigators concluded that because the concentrations of heavy metals in septic tank effluent are low and because

they accumulate only immediately adjacent to the drainfield, even prolonged applications should not cause excessive accumulation in the soil or degradation of the receiving water.

DeWalle and Schaff (1980), rather than investigating the efficiency of one system, looked at regional groundwater quality in an area near Tacoma, Washington, that was dependent on septic tank systems. Using historical data and statistical techniques, they noted gradual increases throughout a 30-year period in the groundwater concentrations of several parameters that were associated with domestic waste from septic tanks. In addition to the increases observed for bacteria, nitrate, and orthophosphate, noticeable increases in the groundwater below the unsewered area were observed for calcium and chloride. Sodium, a major cation in sewage, showed a rather constant concentration during the period, however. The authors postulated that this was due to an ion exchange mechanism in which the sodium replaced the calcium in the cation exchange complex of the clay. Chloride concentrations tended to be higher during the summer months when maximum evaporation increased the salt content of the leachate.

Other investigators have noted high chloride concentrations in groundwaters near on-site disposal systems. Chloride is a mobile anion, much like nitrate, that is typically present in domestic wastewaters in concentrations much greater than background levels but it has more significance as a tracer of septic tank effluent than as a pollutant itself (Brown and Caldwell, 1977). At excessive concentrations in the range of 250 to 500 mg/l, chlorides at the worst may impart a salty taste to drinking water, interfere with agricultural processes (Metcalf & Eddy, Inc., 1979), or accelerate corrosion (Gehm and Bregman, 1976). Generally, levels much lower than these values are observed near septic tank systems.

Some investigators have used the presence of chlorides in groundwaters to indicate the rate and direction of effluent movement in general and nitrate movement, specifically. Chlorides, like nitrates, are not removed readily by the soil and concentrations for both are reduced primarily by dilution. Dudley and Stephenson (1973) noted consistently high correlations between chlorides and nitrates in groundwater below 11 on-site wastewater disposal systems in Wisconsin and found significant chloride contamination at all sites. Similar relationships between the two anions also have been observed by Schmidt (1977) and DeWalle and Schaff (1980).

Although nitrates and chlorides commonly are found in groundwater near septic tank systems, there are other sources that can contribute chlorides to groundwater. Thus, in a monitoring program, these sources need to be identified before septic tanks alone are wrongly accused of causing the contamination. For instance, in hard water areas, water softeners may add even higher chloride concentrations to the wastewater. Also, winter road salting or leachate and runoff from salt piles may increase the chloride content of groundwater.

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B. THE ROLE OF NEEDS DOCUMENTATION IN ALTERNATIVES DEVELOPMENT

A nearly universal obstacle to sound decisions for wastewater management in unsewered areas is lack of adequate local data on the design, use, and water quality impacts of existing conventional on-site systems. The situation is, of course, even worse for innovative systems.

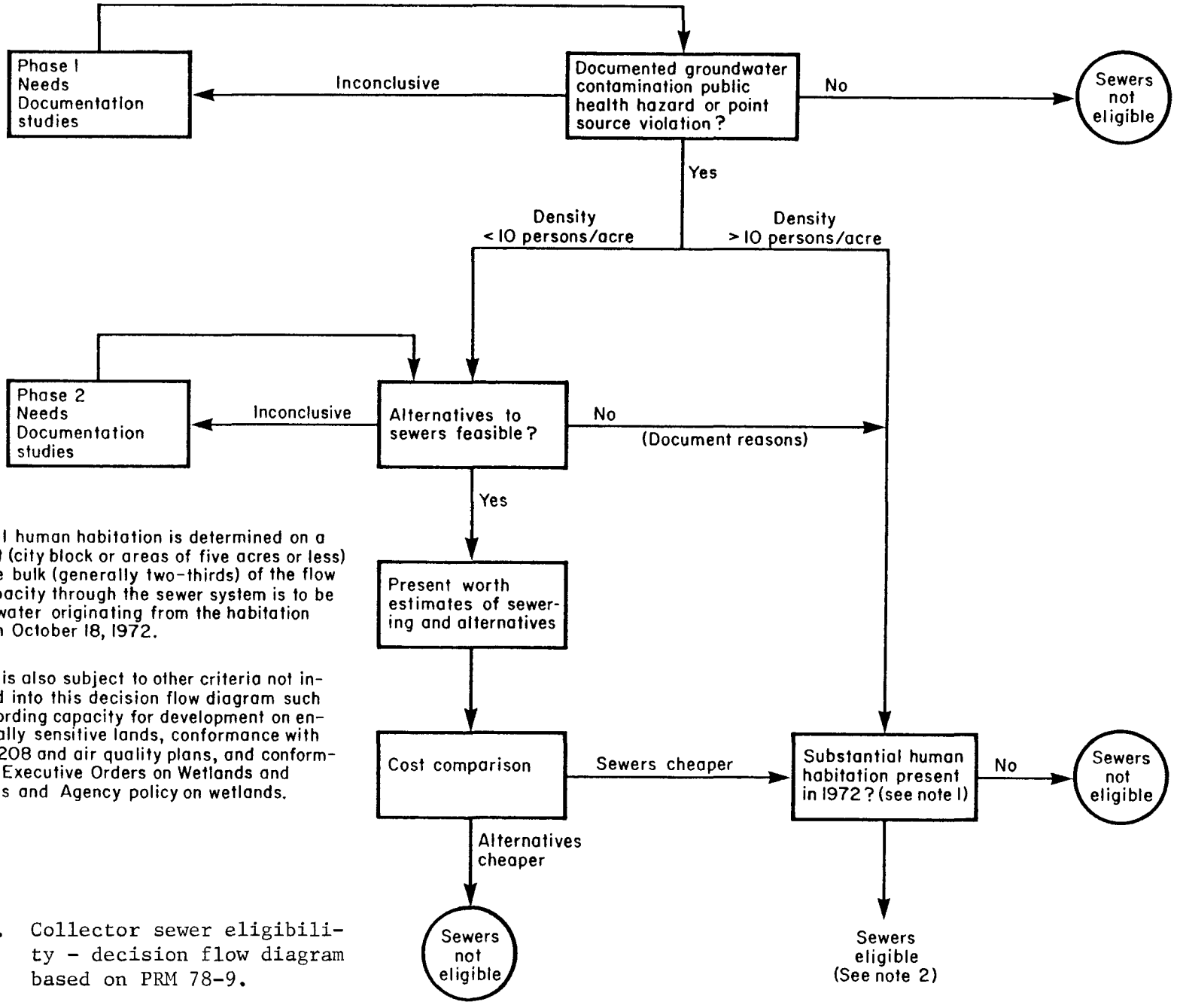
Collection of performance data is necessary to support Construction Grant applications for any unsewered areas. The utility of performance data is obvious if continued use of on-site systems is proposed. If sewers are proposed, the need for them must be documented. This requirement is stated in Program Requirements Memorandum 78-9:

> New collector sewers should be funded only when the systems in use (e.g., septic tanks or raw discharges from homes) for disposal of wastes from the existing population are creating a public health problem, contaminating groundwater, or violating the point source discharge requirements of the Act. Specific documentation of the nature and extent of health, groundwater and discharge problems must be provided in the facility plan.... A community survey of individual disposal system is recommended for this purpose.

Other eligibility criteria for collector sewers are included in PRM 78-9. These criteria and their sequence of application in eligibility decisions are presented in Figure II-B-1 as interpreted from PRM 78-9. The crux of this decision flow diagram is that a need must be documented for a gravity collector sewer to be eligible. Then, if the need is demonstrated, it must be shown that the sewer is the cost-effective means to satisfy the need.

Additional guidance on documentation of need is provided by Program Requirements Memorandum 79-8:

> Facility planning in some small communities with unusual or inconsistent geologic features or other unusual conditions may require house-to-house investigations to provide basic information vital to an accurate cost-effectiveness analysis for each particular problem area. One uniform solution to all the water pollution problems in a planning area is not likely and may not be desirable. This extensive and time-consuming engineering work will normally result in higher planning costs, which are expected to be justified by the considerable construction and operation and maintenance cost savings of small systems over conventional collection and treatment works.
>
> Though house-to-house visits are necessary in some areas, sufficient augmenting information may be available from the local sanitarian, geologist, Soil Conservation Service representative or other source to permit preparation of the cost-effective analysis. Other sources include aerial photography and boat-carried leachate-sensing equipment which can be helpful in locating failing systems. Detailed engineering investigation, including soil profile examination, percolation tests, etc., on each and every occupied lot should rarely be necessary during facility planning.



Note 1: Substantial human habitation is determined on a small tract (city block or areas of five acres or less) basis. The bulk (generally two-thirds) of the flow design capacity through the sewer system is to be for wastewater originating from the habitation existing on October 18, 1972.

Note 2: Eligibility is also subject to other criteria not incorporated into this decision flow diagram such as not affording capacity for development on environmentally sensitive lands, conformance with approved 208 and air quality plans, and conformance with Executive Orders on Wetlands and Floodplains and Agency policy on wetlands.

Figure II-B-1. Collector sewer eligibility - decision flow diagram based on PRM 78-9.

Based on application of these policies during preparation of the Seven Rural Lake EIS's, Region V in cooperation with states in the Region, developed additional guidance on the collection and use of performance data titled "Region V Guidance--Site-Specific Needs Determination and Alternative Planning for Unsewered Areas." It is attached to Technical Reference Report XVI-D as Appendix A. A key feature of this guidance is the recommended sequencing of data collection with decisions on development and selection of alternatives so that all necessary data are collected in a timely manner and unnecessary data collection is avoided.

The Region V Guidance also emphasizes performance as the relevant criterion for need. Though this seems obvious, facilities plans and state policies often rely on nonconformance with current design codes as the criterion for need. Use of nonconformance alone as a criterion would result in the abandonment of many older systems, even though they may have many more years of use remaining.

The Region V Guidance was prepared so as to apply to almost any community. It is, therefore, more complex than would be necessary in many cases. Conditions under which the recommended needs documentation/alternatives development process might be simplified are discussed in Technical Reference Report XVI-D.

C. REVIEW OF DIRECT AND REMOTE SENSING TECHNIQUES

1. INTRODUCTION

This section outlines groundwater and earth sensing techniques that can provide important data for the planning, design, and monitoring of on-site and small-scale wastewater treatment facilities.

Sensing methods can be categorized as either direct or remote. Direct sensing refers to on-site observations, well-drilling operations, and the measurement of physical and chemical parameters within water, soil, and rock samples. Remote sensing methods are indirect, non-destructive means of data collection that take advantage of the physical and chemical properties of matter.

Electromagnetic waves are constantly emitted from or reflected by water, animals, plants, and earth materials and can be detected from aircraft and satellites. When provided with sufficient ground truth reference data, an investigator can evaluate land use, plant growth trends, temperature conditions, and geological structures by the use of aerial photography and imagery.

Seismic techniques are based on the sound propagation characteristics of earth materials.

Electrical methods study the ways in which rocks and soils conduct induced or naturally occurring electrical currents.

Thermisters can be used to evaluate temperature variations in soil columns.

a. Direct Sensing

Human beings gather more information through vision than through all other senses combined. On-site inspection of proposed wastewater treatment sites is absolutely essential to obtain first-order approximations of topographic restraints, soil textures and wetness, man-made structures and excavations, and the variety and condition of vegetation. Soil erosion problems can be identified, and the apparent operating conditions of nearly all treatment systems evaluated. Visual inspection is the only way to accurately note and record the locations of data collection points such as wells, springs, lakes, and rock outcrops.

Soil parameters are of greatest priority in evaluating the potential surface or subsurface land treatment of wastewater. "Grab" samples can be obtained directly from the surface, but this is not representative of the entire soil vertical column. The use of a hand auger, mechanical auger, or a soil core tube sampler enables the investigator to document soil horizonation and classification, and to identify shallow groundwater levels. Relatively undisturbed samples of the entire soil column can be obtained after digging a trench that penetrates to the regolith. Soil samples of this type can subsequently be chemically and physically analyzed and subjected to permeability testing.

Rock samples can be collected from in situ outcroppings in and near the proposed site, then compared with published geological data for the region. It is important to identify any underlying aquifer systems that would potentially be degraded by surface or subsurface land treatment systems, especially in proximity to recharge zones.

The measurement of surface water and groundwater parameters may be necessary to establish baseline conditions and to monitor future trends resulting from wastewater system discharges. Nearby streams, rivers, and lakes should be studied* as appropriate to obtain relevant information such as:

- Temperature variations,
- Stage levels and flow velocities for streams and rivers, especially low flow conditions (measured with flow meters and weirs),
- Secchi disk turbidity measurements, and
- Chemical analyses of water samples, including
  - pH
  - Conductivity
  - Nitrates
  - Bacteria
  - Viruses
  - Phosphorus
  - Sulfates
  - Iron
  - Aluminum
  - Manganese
  - Hardness (calcium, magnesium, carbonate)

Groundwater samples should be collected from springs and existing (strategically chosen) wells in or near the proposed site and analyzed for the temperature and relevant chemical parameters such as those listed above. Data collection of surface parameters should ideally coincide with remote sensing by aerial photography or imaging in the event that the latter methods are also used.

Borehole drilling data are appropriately collected when moderate to large-scale (more than 5000 gallons per day) land application systems are proposed, when groundwater resources are particularly sensitive, or when the area being evaluated will contain numerous treatment systems or is near an ecosystem sensitive to nutrient inputs (for example, a lake or bog). Drilling enables the direct sampling of geological formations in the form of core sections, drill cuttings, or sidewall samples. Such boreholes can be outfitted with screens and surface casings to function as trend-monitoring wells. Such wells provide immediate access to groundwater for the purposes of measuring water temperatures and elevation, pH, conductivity, and chemical composition. Suction pumps cannot be used to sample water columns in wells more than 30 feet deep. Reactive gases released from well waters can be trapped by keeping wells capped, periodically sampled, and analyzed with a gas partitioner or gas chromatograph. Given access to wells, old or new, the investigator can perform

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*Samples should be collected during low-flow and storm conditions to establish variations.

aquifer pump testing (Walton, 1970; Davis and DeWiest, 1966) to ascertain the transmissivity and storativity values for the penetrated formations. Knowledge of these formation "constants" enables the prediction of future groundwater movements within a local or regional flow system.

b. Indirect Sensing

Indirect sensing techniques are used to monitor the physical phenomena associated with matter and energy, which include:

- potential fields (gravity and magnetics),
- electrical characteristics, and
- energy emittance and reflectivity.

Of the above, electrical and energy characteristics can be used in the remote sensing of environmental problems.

Indirect sensing data have significance only when they can be contrasted with parameters measured directly in the medium under investigation. In geophysics, for example, seismic and resistivity surveys can accurately determine the depths to zones of earth materials having contrasting physical properties. But without direct samplings of the strata, it will not be known what properties are being contrasted. Ideally, both ends of geophysical transects should be correlated with locations of boreholes or other sources of subsurface information like rock outcroppings and deep excavations. In this way the parameters of lithology, structure, and water table levels are established at two points and the geophysical profiles allow reliable extrapolations between these known points. Similarly, periodic remote sensing by aerial photography can show subtle changes in surface reflectivity, but environmental interpretations are useless in the absence of a well-documented ground truth data base.

2. GEOPHYSICAL METHODS

a. Seismic Techniques

Seismic refraction can be very useful in extrapolating the subsurface geological and hydrological conditions between known data locations like drillholes. Sonic impulses are generated by a variety of destructive or nondestructive means in the earth and detected at specified distances by geophones. The first signal to reach a geophone located close to the energy source will be the wave that travels directly through the soil along the surface of the earth (surface wave). At greater geophone distances, the first arrival signals will be those refracted from higher velocity subsurface layers, such as consolidated rock or water tables. Depths to these interfaces can be calculated from the distance and travel times from the energy source to a geophone, but parameters such as lithology and zones of saturation cannot be determined unless the seismic profile is tied to locations of known subsurface data.

Relatively simple, single-channel seismic refraction instruments are commercially available. A single geophone is used at varying distances to record signals generated by striking a metal plate with a hammer. A single-integrating memory recorder is a highly desirable feature which, when used, enables the detection of weaker refracted signals.

Refraction methods are very useful tools in the reconnaissance of water tables in areas for which there are only a few data locations. Refraction works best in geological columns with thick sections of unconsolidated material. This is a common characteristic of glaciated terrains in the north-central and northeastern United States.

Reflection techniques differ from refraction in that the desired data are arrival times of seismic waves reflected from successively deeper interfaces of contrasting acoustic transmission. These interfaces are caused by lateral or vertical changes in subsurface lithology. Conventional seismic reflection methods are much more costly than refraction techniques. Multi-channel recorders are used to increase the efficiency of reflection surveys, utilizing groups of geophones arranged in complex arrays over the area under investigation. Shot points are placed at intervals within the phone arrays and the results recorded. Since some geophones are located near the shot points, first arrivals of reflected waves often occur during or shortly after the passing of the surface wave ("noise"), necessitating costly digital filtering and processing of the data. An advantage of the reflection method is the ability to penetrate high-velocity zones that overlie low-velocity layers. Refraction methods cannot correctly interpret these conditions and can only provide misleading data about them. In general, reflection methods will be overly sophisticated for application to water resource studies in small waste flows management.

Microwave reflection is a relatively new technique that is a topic of ongoing research and development. Theoretically, the method is similar to conventional seismic reflection, but the acoustic source operates at a much higher frequency and the depth of penetration into the earth is reduced. Microwaves can readily penetrate dry materials, but they are greatly attenuated when passing through saturated substances. Thus, microwave reflection is useful in detecting shallow, water-saturated zones beneath the surface. Conventional seismic reflection gear is generally used to detect fairly deep targets since it is difficult to differentiate wave arrivals from shallow reflections and seismic energy sources.

Commercially available microwave systems operate in either pulsed or continuous wave modes and have been used to detect geological structures such as faults and caverns at depths greater than 200m. Although water tables can be identified in this way, resolution can be improved by contrasting simultaneously collected pulsed and continuous wave data. Depths to reflective layers are interpreted from the analysis of electromagnetic interference patterns generated by this bi-modal technique (Koerner, 1978).

b. Electrical Resistivity

Resistivity techniques can readily be used to detect water table depths. The flow of direct current is generated in the ground by two electrodes and the drop in potential between them is measured by another pair of electrodes.

Vertical resistivity profiling would be the appropriate way to detect the water table on a specific site, utilizing the Wenner electrode arrangement (Dobrin, 1976). Initially, a small electrode spacing is used. It is subsequently increased until a significant drop in apparent resistivity occurs. The resulting data are more difficult to interpret than seismic data. The method relies on matching field data curves to theoretical layer-case curves in a manner somewhat analogous to that used in evaluating aquifer test data. Although resistivity data are somewhat difficult to interpret, the equipment used is generally not as expensive as seismic equipment.

Prior to collecting resistivity data, a prospective site must be inspected for the distribution of surface and buried metallic objects like pipes and fences, which can seriously distort the data. Strong electromagnetic fields produced by low angle radar and aircraft instrument landing systems also present limitations for electrical resistivity surveys. The successful interpretation of geophysical data requires the services of personnel with a high level of theoretical expertise and field experience.

c. Geothermal Gradient

The relative proximity of groundwater to the land surface can sometimes be determined by simply measuring soil temperatures. It has been shown that a thermister in combination with a probe inserted in the soil column can be used to detect zones of groundwater discharge by means of measured temperature variations. During winter months, soils in discharge zones are generally warmer than surrounding soils, which have temperatures close to the ambient air temperature. Conversely, in summer, soils are often cooler in discharge zones. Recharge zones are not readily detected in this manner. Data obtained in this way should not be confused with the geothermal gradients measured in boreholes to determine the relative values of heat conductivity within the earth's crust.

3. REMOTE SENSING

In theory, there is no limit to the possible applications of analyzing the electromagnetic "signatures" of different kinds of matter. At present, however, the practical uses of remote sensing are limited in their application to environmental problems because of the complexity of the earth's surface. Monitoring of this surface is influenced by the many variables that regulate the emission and reflection of electromagnetic radiation. The following is a partial list of these variables:

- soil variations (specific heat, moisture, and organic content),
- rock lithology,
- ambient temperature,
- sun angle,
- barometric pressure,
- wind velocity,
- humidity,

- precipitation,
- snow and ice accumulations,
- water bodies (oceans, lakes and rivers),
- particulates suspended in air and water,
- cloud cover,
- water chemistry,
- effects of man (agriculture, structural engineering, and strip mining), and
- natural catastrophes (floods, volcanic eruptions, and meteorite impacts).

Rock lithology is a variable only in an areal sense, unless it is altered by strip mining or catastrophism. Vegetation undergoes slow seasonal change, as well as more rapid changes induced by water availability, parasitism, and disease. Snow accumulations can alter the electromagnetic characteristics of a surface within a fraction of an hour. Although most surface variations on the earth are seasonal or diurnal in nature, catastrophic events like volcanic eruptions can produce radical changes which endure for hundreds or thousands of years. Man's efforts in agriculture and engineering have completely altered many natural landscapes.

Because of the extreme variability of the earth's surface, it is apparent that aerial sensing data have little value unless the information can be contrasted with data collected simultaneously on the ground. In this way, a time-correlated data base is accumulated with which to compare future variations detected by remote methods. Apparent associations between remote sensing information and ground truth data can be tested with regression analyses.

From the standpoint of earth monitoring, the most useful portions of the electromagnetic spectrum are the infrared and visible light regions. The intensity of most of the ultraviolet band is strongly scattered and attenuated by dust particles in the atmosphere. Two-thirds of the solar energy intercepted by this planet reaches the surface, and can be subdivided as follows:

- 10% Ultraviolet (UV),
- 50% Visible Light, and
- 40% Infrared (IR).

a. Aerial Photography

The first serious use of aerial photography as a sensing technique occurred during World War I, in which it was used militarily to monitor troop and equipment concentrations in France. The current state-of-the-art in this branch of remote sensing owes much of its progress to government financing for the research of military applications.

Remote sensing via aerial photography can be of practical value in the trend monitoring of on-site wastewater treatment facilities. Color and infrared photography can be used in combination to study changes in the types and concentrations of foliage that occur in response to the availability of water and nutrients. Remote black and white photography is not of practical value in the study of foliage differences. Vegetation is in general highly

reflective in the infrared region, and different kinds of foliage reflect varying amounts of IR. For example, grasses generally reflect more than 70% of intercepted IR, while fir trees reflect less than 20% (Fritz 1967).

In the event of the failure of a treatment system, a high nutrient load would be released in the subsurface. During the growing season, such an occurrence would promote an increased vigor in plant growth, resulting in greater total IR reflectivity. Surface outbreaks of effluent are even more apparent, since water is a strong absorber of IR. Such areas generally appear black on color infrared film.

A serious obstacle to the photographic monitoring of septic systems is the density of trees and bushes in an area. These plants can sometimes effectively shield a drainage system from remote observation. This obstacle can be partially overcome by obtaining substantial overlap between photographic frames. Stereo observation can sometimes reveal malfunction "signatures" that would otherwise be obscured.

Comparative true color and infrared aerial photography was acquired, interpreted for surface malfunctions, and field-verified for each of the Seven Rural Lake EIS's. This effort, made by U.S. EPA's Environmental Photographic Interpretation Center for Region V, was the first full-scale use of the technique as a community-wide survey of surface malfunctions from septic tank systems. Materials and techniques used were published in the appendices to the Draft EIS's. Another example of the use of comparative color and infrared aerial photography in the evaluation of septic system performance is reported by Slonecker (1980).

There is a common misconception that IR photography can be used to detect subtle temperature differences within an environment. That kind of sensitivity can only be achieved using an infrared imaging scanner.

b. Optical Scanning Systems

Unlike conventional photography, optical scanning systems are able to detect very subtle variations in the electromagnetic radiations of matter. Systems have been developed and used for sensing in the infrared, visible light, and ultraviolet regions. Infrared scanners have the greatest potential for applicability to the sensing of environmental phenomena. Although the IR spectrum lies between the wavelengths of 0.76u and 1000 u, most imagery is obtained in either the 3 to 5u or 8 to 14u atmospheric "windows."

The basic elements of a scanner are optical mirrors, a signal amplifier, and an IR detector. In order to increase its sensitivity, the detector is housed in an insulated jacket filled with liquid nitrogen. An image beam is focused on the detector by a rotating mirror, generating a signal that is amplified and modulated for strip-wise projection onto photographic film (Taylor and Stingelin, 1969). It has been discovered that the greatest equipment sensitivity is achieved by scanning during the early evening hours, thus avoiding the recording of reflected solar IR energy. Temperature variations of less than 1°C can be detected in aerial imagery, but only at altitudes of 5000 feet or less.

IR scanning can be directly applied to the planning and monitoring of on-site waste treatment facilities. The presence of surface or groundwater is usually associated with a significant thermal gradient in the environment. Thus, groundwater discharge zones can be readily detected, and surface drainage patterns within an area can be much more reliably distinguished than with conventional photographic interpretations. Structural zones with high transmissivities, such as faults and fractures, can often be identified even when they do not appear in aerial photography. Also, seepage patterns along lakes, rivers, and canals often produce detectable temperature gradients where the groundwater first encounters surface waters. Periodic monitoring of soils can potentially show changes in water saturation due to seepage and precipitation.

Although IR imaging is obviously a useful technique in monitoring groundwater and surface water movements, it has the disadvantage of being very costly and requiring special expertise for the proper operation of the complex equipment. On a regional basis, the technique might best be used to periodically monitor the apparent performance levels of septic systems, thus identifying areas, localities, and individual systems that may be in need of on-site inspections. At the same time, patterns of groundwater influx to delicate ecosystems like lakes, bogs, and marshes can be monitored. Used in this way, aerial IR imaging may prove to be a cost-effective technique for reducing the frequency and extent of required on-site inspections, but only in the event that adequate ground-truth data have been historically collected.

4. SATELLITE IMAGERY (LANDSAT)

In relation to the scope of this paper, the most important contribution of LANDSAT imagery is its use in the regional classification and monitoring of trophic conditions in lakes. A classic study of this type was done in Wisconsin (Holmquist and Scarpace, 1977) making use of LANDSAT multispectral scanner data.

The objective of the study was to ultimately establish a broad ground-truth data base to enable the future independent monitoring of lake trophic conditions via satellite imagery. Four wavelength bands were used, including portions of the visual and infrared regions. The study involved a statistical comparison of spectral reflectivity with ground-truth investigations. Surface data were collected in conjunction with satellite imaging passes. Measurements included the gathering of Secchi disk data and the collection of water samples for laboratory analyses of transmittance and reflectance based on suspended solids. Although rather complex data processing methods were required, a high percent reliability was achieved in the prediction of trophic levels.

5. CONCLUSIONS

The results of this section are listed in Table II-C-1. The sensing techniques described are summarized in terms of relative cost, parameters measured, and types of technical expertise required under ideal staffing conditions. It is especially important to compare the relative cost of a technique with its reliability and level of managerial applicability.

TABLE II-C-1. DIRECT AND REMOTE SENSING TECHNIQUES

Technique	Relative Cost and Parameter(s) Measured		Level(s) of Applicability to Small Waste Flows Systems	Technical Expertise Required	Reliability of Method
Visual Inspection	Low Cost	Slope Soil Texture Vegetation Types and Variety Man-Made Structures	Individual Systems	Soil Agronomist Environmental Engineer	Excellent
On-Site Sampling of Soil, Rock, Springs and Wells	Low Cost	Soil Horizonation Groundwater Chemistry Geologic Formations Water Table	Individual Systems Local	Soil Agronomist Hydrogeologist Chemist	Good
Borehole Drilling and Aquifer Testing	High Cost	Soil Horizonation Groundwater Chemistry Geologic Formation Formation Constants Water Table Depth	Local Area Regional	Driller Hydrogeologist Chemist	Excellent
Seismic Refraction	Medium Cost	Water Table Depth Lithology Variation	Local Area Regional	Geophysicist	Good
Seismic Refraction	High Cost	Water Table Depth Lithology Variation	Local Area Regional	Geophysicist	Good
Microwave	High Cost	Water Table Depth Lithology Variation	Local Area Regional	Geophysicist	Good
Electrical Resistivity	Medium Cost	Water Table Depth Salt/Fresh Water Interface	Local Area Regional	Hydrogeologist	Good
Temperature	Low Cost	Relative Proximity to Groundwater Discharge	Individual Systems Local	Hydrogeologist	Fair
Aerial Photography	Low Cost	Lake Trophic Status Geologic Structure Variations in Plant Growth Drainage Patterns	Individual Systems Local Area Regional	Photographic Processor and Interpreter	Fair
Optical Scanning	High Cost	Drainage Patterns Geologic Structure Groundwater Seepage Discharge	Individual Systems Local Area Regional	Physicist Photographic Processor and Interpreter	Excellent
Satellite Imaging	High Cost	Lake Trophic Status	Area Regional		Fair

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D. SEPTIC LEACHATE DETECTOR RESEARCH

1. INTRODUCTION

The Septic Leachate Detector (SLD) is an instrument that is capable of detecting effluents of septic tanks systems by responding to a combination of conductivity changes and fluorescence. An SLD was used on 35 mid-western lakes during preparation of the Seven Rural Lakes Area EIS's. The results of shoreline scans with the SLD and concurrent water sampling for nutrient and bacteriological analysis constituted one of several data inputs to alternative development and decision making for those EIS's.

The original shoreline scans and sample preparation were designed to locate sources of actual or potential nutrient inputs and bacteriological contamination. While these surveys accomplished their purpose quite adequately for the level of decision making they supported, review of the survey methods, instrument operation and the data raised questions about use of the information generated. Generally stated the questions were:

- Does the instrument detect all effluent sources?
- Does it give positive readings for non-wastewater sources of fluorescent materials or dissolved solids?
- Can the SLD be used alone to quantify nutrient inputs to lakes from septic tanks systems?
- If the instrument cannot be used alone to quantify nutrient inputs, can it be used with other techniques to do so economically?

Two field trips and several laboratory studies were sponsored by U.S. EPA Region V to address these questions. This report describes the SLD used in these efforts and describes the work conducted to address them.

2. DESCRIPTION OF THE INSTRUMENT AND THEORY OF OPERATION

The SLD used is an ENDECO (Environmental Devices Corporation, Marion, Massachusetts) Type 2100 Septic Leachate Detector System, also referred to as the Septic Snooper™. It is protected by U.S. Patent Number 4,112,741.

The instrument and its theory of operation are described by excerpts from the manufacturer's operation manual (ENDECO, 1980):

> The ENDECO Type 2100 Septic Leachate Detector System is a portable field instructment that monitors two parameters, fluorescence (organic channel) and conductivity (inorganic channel). The system is based on a stable relationship between fluorescence and conductivity in typical leachate outfalls. Readings for each channel appear visually on panel meters while the information is recorded on a self-contained strip chart recorder. Recording modes are selectable between individual channel outputs or a combined output. The combined output is the arithmetic result of an analog computer circuit that sums the two channels and compares the resultant signal against the background to which the instrument was calibrated. The resultant output is expressed as a percentage

of the background. Also, the combined recorded output is automatically adjusted for slow background changes. The system can be operated from a small boat enabling an operator to continuously scan an expansive shoreline at walking pace and, through real time feedback, effectively limit the need for discrete grab samples to areas showing high probability of effluent leaching. Expensive laboratory time for detailed nutrient analysis is greatly reduced while survey accuracy is increased substantially.

...The unit [is] powered by a standard 12-Volt automotive battery. The plug-in, flow-through conductivity cell is in series with the fluorometer. The probe/wand houses a marine-type centrifugal pump. Discrete samples may be drawn directly from the instrument discharge for subsequent laboratory analysis....

...Wastewater effluent contains a mixture of near UV florescent organics derived from whiteners, surfactants and natural degradation products that are persistent under the combined conditions of low oxygen and darkness...

Aged effluent percolating through sandy loan soil under anaerobic conditions reaches a stable ratio between the organic content and chlorides which are highly mobile anions. The stable ratio (cojoint signal) between fluorescence and conductivity allows ready detection of leachate plumes by their conservative tracers as an early warning of potential nutrient break-throughs or public health problems.

The Septic Leachate Detector System consists of the subsurface probe, the water intake system, the logic analyzer control unit, panel meters and the strip chart recorder....

The probe/wand is submerged along the shoreline. Background water plus groundwater seeping through the shore bottom is drawn into the subsurface intake of the probe and is lifted upwards to the analyzer unit by a battery operated, submersible pump.

Upon entering the analyzer unit the solution first passes through the fluorometer's optical chamber where a continuous measurement is made of the solution's narrow band response to UV excitation. The solution then flows through a conductivity measurement cell. An electrode-type conductivity/thermistor probe continuously determines the solution's conductivity. The solution exits the conductivity cell directly to the discharge where discrete samples may be collected if indicated by the response of the leachate detector. Both parameters are continuously displayed on separate panel meters. Zero controls are provided for both parameters (organic and inorganic) to enable "dialing out" the background characteristics to provide maximum sensitivity, as well as enhancing the response caused by a suspected abnormality. Span Controls are also provided to control the sensitivity of each parameter separately during instrument calibration. This is helpful in determining relative concentrations of leachate outfalls.

The signals generated and displayed on the panel meters are also sent to an arithmetic/comparator analog computer circuit designed to detect changes in the ratio of organics and inorganics typical of septic

leachate. The output of this circuitry is recorded continuously on a strip chart and is the key indicator of a suspected leachate outfall. However, isolated increased in either parameter may be cause for concern and should be sampled for analysis for other potential forms of nutrient pollution.

The operation manual does not reveal the type of light filters used in the instruments' fluorometer. Since the spectral characteristics of filters are crucial to answering some of this report's questions, the filter types and key characteristics are presented in Table II-D-1. The filter's responses to various wavelengths are shown in Figure II.D.1.

TABLE II-D-1. SPECTRAL CHARACTERISTICS OF FLUOROMETER LIGHT FILTERS IN ENDECO MODEL 2100 SEPTIC LEACHATE DETECTOR

Excitation filter	Minimum Wavelength	Peak Wavelength	Maximum Wavelength	% Transmittance at Peak Wavelength
Filter A	320 nm	360 nm	390 nm	35%
Emission Filters				
Filter B	400 nm	500 nm	-	90% at 550
Filter C	385 nm	430 nm	490 nm	46%
Filters B + C	410 nm	435 nm	490 nm	38%

3. CHEMISTRY AND FLORESCENCE OF BRIGHTENERS, DETERGENTS, AND NATURAL ORGANIC COMPOUNDS

a. Brighteners

Brighters and whiteners are chemicals added to cleaning products to improve the apparent "whiteness" of fabrics. The energy of incandescent light bulbs is typically deficient in the shorter (blue) wavelengths. This causes fabrics to seem yellower than they would be in sunlight. The blue wavelengths reflected by fabrics are artificially augmented by brighteners and whiteners which absorb invisible ultraviolet light, then release this energy by fluorescing at blue wavelengths.

During clothes washing these fluorescent chemicals absorb to the surface of fabric fibers. Fabrics are made of both natural and synthetic fibers which have a wide range of surface characteristics. There are a variety of fluorescent chemicals designed to absorb onto different fabrics. The American Society for Testing and Materials groups the commercial whiteners into five classes by molecular structure. Figure II.D.2 reproduces the chemical structure of the classes. (ASTM, 1976).

Figure II.D.1. Septic Leachate Detector's Filter Responses

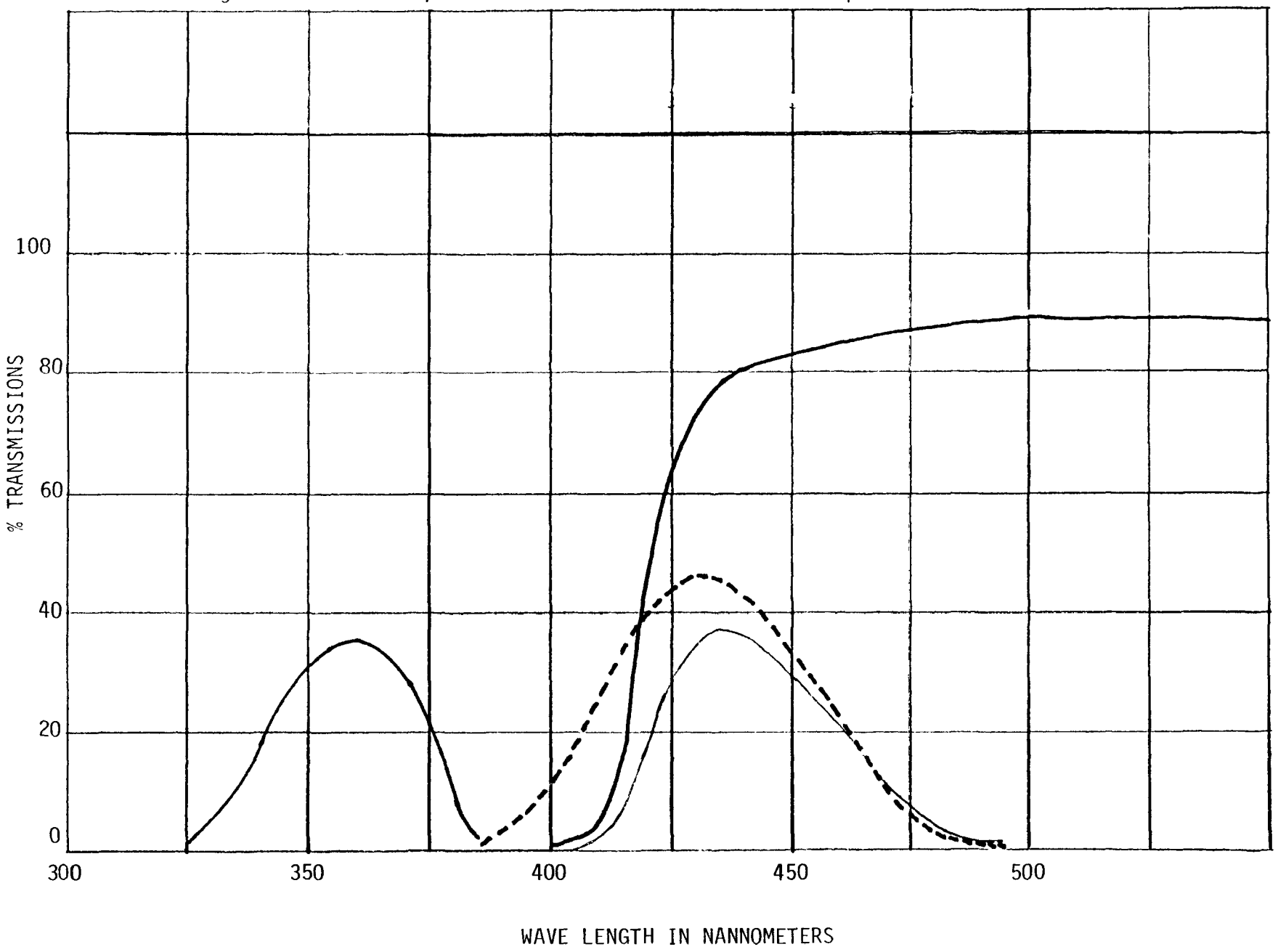
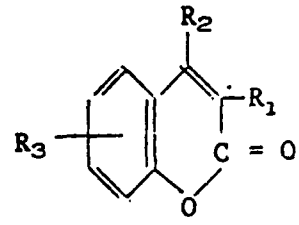
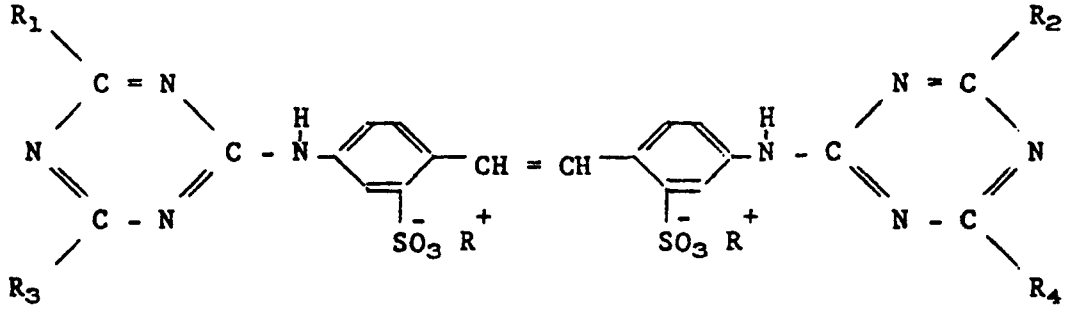


Figure II.D.2. Chemical Structures of Five Classes of Fluorescent Whitening Agents (from ASTM, 1976)

C COUMARIN

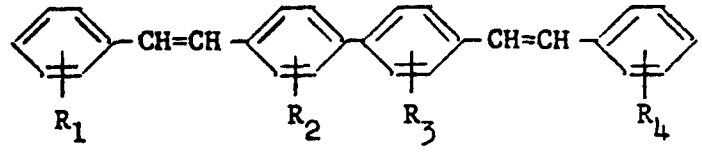


DASC DIAMINOSTILBENEDISULFONIC ACID–CYANURIC CHLORIDE

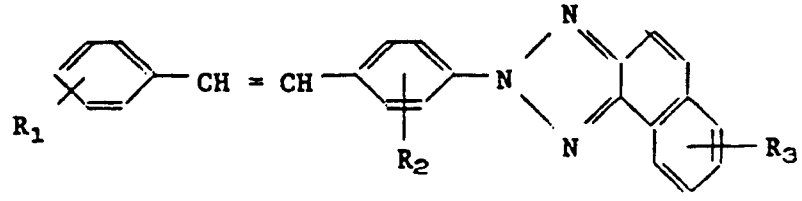


$R^+ = H^+, Na^+, K^+$, etc.

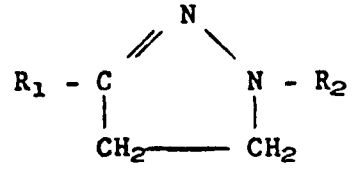
DSBP DISTYRYLBIPHENYL



NTS NAPHTHOTRIAZOLYLSTILBENE



P PYRAZOLINE



Samples of pure whiteners were requested from manufacturers. Nine samples were received. Each of the five classes was represented. Absorption and fluorescence spectra of the samples in dilute aqueons solution were recorded using an Aminco-Bowman spectrophotofluorometer. Table II-D-2 presents relevant data. The samples are listed in order of their fluorescence peak wavelengths.

Table II-D-2 includes the transmission characteristics of the excitation and emission filters in the SLD. It can be seen that all of the whiteners except DSBP-1 and the second unlisted one (suspected of being Type C) will strongly absorb the light passed by the excitation filter. The excitation spectra of DSBP-1 and the second unlisted sample do overlap the wavelengths passed by the filter but not as completely as the spectra of the other whiteners. All of the samples fluoresce strongly in the wavelengths passed by the excitation filter.

b. Detergents

A number of commercially available cleaning products were acquired. Dilute solutions of the products were tested with the SLD for fluorescence. Samples were also analyzed for their absorption and fluorescence spectra with an Aminco-Bowman Spectrophoto fluorometer.

Spectral characteristics of cleaning products analyzed by spectrophotofluorometer are listed in Table II-D-3. Recorder tracing of the absorbance and fluorescence responses are provided in Appendix A.

Comparing the absorbance of these products with the spectral characteristics of the SLD's excitation filter (Table II-D-1), it can be seen that the peak absorbance of all these products are at lower wavelengths than the transmission peak of the filter, 360 nm. However, in all but four of the products, the overlap between filter transmission and absorbance is probably sufficient to mitigate fluorescence. The excepts include two fabric softeners (Sta-Puff and Downy) which absorb at low wavelengths and a cleanser (Ajax) and a liquid dish detergent (Ivory) which absorb weakly at any wavelength.

Comparing fluorescence of these products with the spectral characteristics of the SLD's emission fliters, the peak fluorescence of the products fall below the transmission peak of the filters, 435 nm, but overlap is good except for the cleanser which fluoresces very weakly.

The fluorescence of these cleaning products is presumed to be caused chiefly, if not totally, by added brighteners. The detergent ingredients may fluoresce weakly but at wavelengths below the lower transmission threshold of the SLD's emission filters, 410 nm. Figure II-D.3 illustrates the fluorescence of a common detergent without brighteners.

While detergents do not themselves fluoresce, they can substantially improve the fluorescence of some materials which have a low quantum yield (proportion of absorbed light energy that is given off as fluorescence). The detergents do this by forming molecular aggregates, called micelles, around the fluorescing material. Micelles act liks a shield, preventing contacts between the fluorescent material and other dissolved chemicals. This reduces the amount of absorbed light that may be transferred by non-fluorescing

TABLE II-D-2. ABSORPTION AND FLUORESCENCE CHARACTERISTICS OF WHITENER SAMPLES (wavelengths in nannometers)

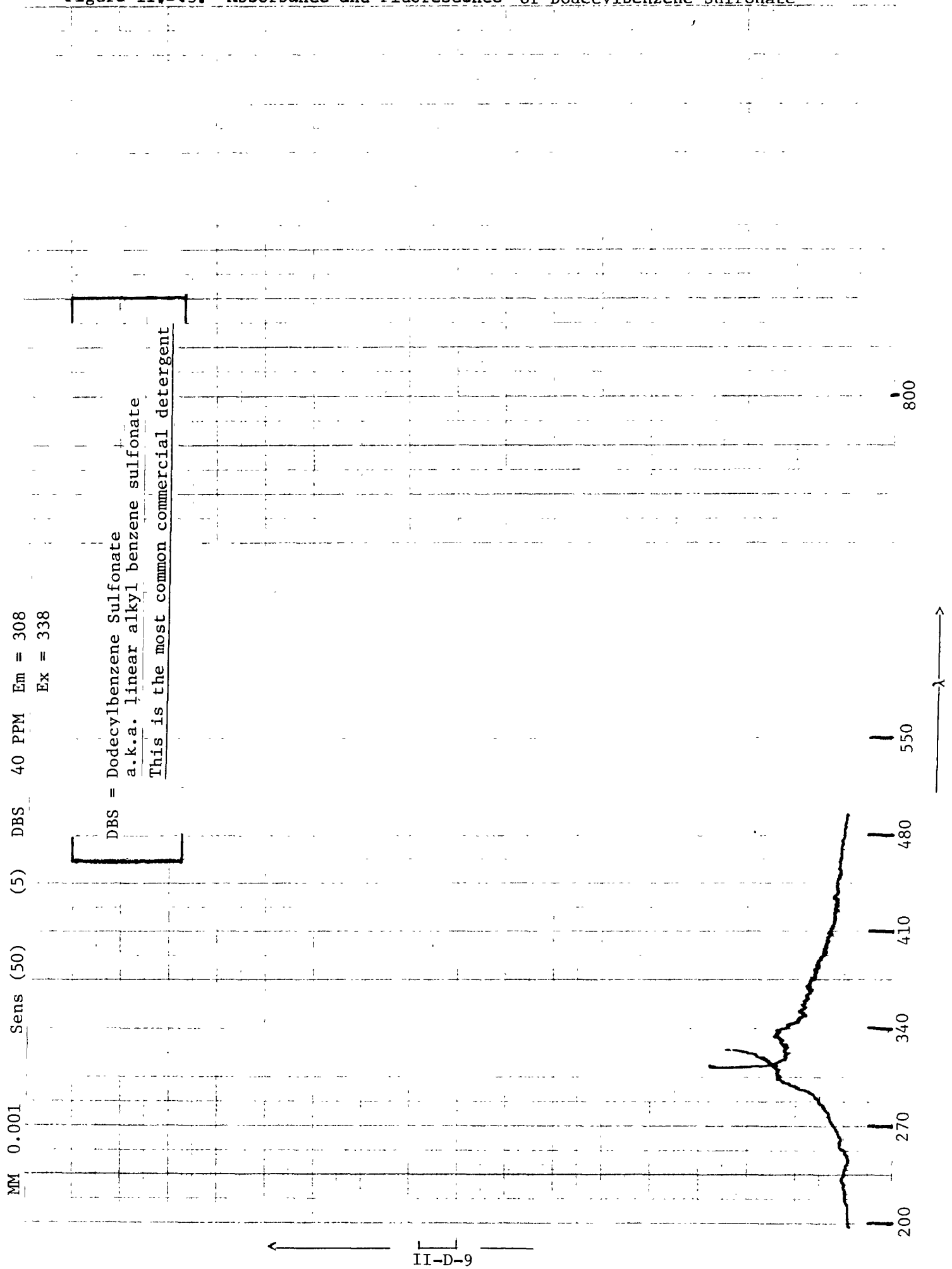
ASTM Type Designation of Sample[a]	Absorbance			Fluorescence		
	min.	peak(s)	max.	min.	peak(s)	max.
DSBP-1	368	387	416	[b]	408	571
Unlisted	280 <	320 384	< 421	368[b] <	391 410(D)	< 553
DASC-3	288 <	311 370(D)	< 421	370[b] <	391 410(D)	< 536
DASC-4	284 <	315 370(D)	< 412	368 <	393 410(D)	< 525
NTS-1	271	357	424	#63	425	571
P-1	271	360	448[b]	368 <	434(D) 460	< 563
C-1	267 <	331 395	< 439	395	441	579
C-2	280 <	336 407	< 448	395	444	563
Unlisted	280 391	322 420	360 448[b]	395	448	554
SLD Filters' Transmittance	320	360	390	410	435	490

[a] - As identified by ASTM, 1976. See Figure II.D.2.
[b] - Recording not complete. Values presented are approximations.
< - Spectra associated with peaks substantially overlap.
D - Dominant peak.

TABLE II-D-3. ABSORBANCE AND FLUORESCENCE CHARACTERISTICS OF DETERGENT SAMPLES (wavelengths in nannometers)

Cleaning Product	Absorbance			Fluorescence		
	min.	peak(s)	max.	min.	peak(s)	max.
Final Touch	275	325	380*	380*	420	525*
Wisk	290	350	395	375*	425	560
Arm & Hammer	275	332	380*	38-*	410	525
Borateem	275	345	380*	380*	418	530
Joy	275	350	390	390	430	530
Ajax	No distinct peaks. Weak response.					
Downy	260	305	330	350	380	500
Ivory	Several small peaks from 230 to 380			370 weak response 430 weak response		
Fab	275	338	380*	380*	410	530
Tide	275	338	380*	380*	416	530
Clorox 2	275	340 355	 370	380	395 415	 530
Fresh Start	290	341	395*	365	412	530
All	260	332	380*	380*	405	530
Palmolive	275	345	380*	380*	390 417 475	 560
Yes	290	355	400	360	385 405	 530
Trend	275	335	380*	380*	418	530
ERA	280	335	396	380	418	530
Woolite	280	356	395	360	428	560
Sta-Puff	200	240 305	 335	350	380	475
Ivory Snow	260	350	380&	390	400 425	 530*
Bold 3	260	330 350	 360*	380*	410	560
Cheer	275	335	380*	380*	410	530

Figure II.D.3. Absorbance and Fluorescence of Dodecylbenzene Sulfonate



mechanisms. Thus, in sufficient concentrations, detergents can increase the fluorescence of some materials by an order or magnitude (Armstrong et al., 1981). This "micelle effect" may not be significant with chemicals like brighteners designed to have high quantum yields. However, it may be significant with natural products of organic decompositon as discussed in the next subsection.

Cleaning product solutions were made up with both distilled water and a 0.03 M solution of a pure detergent, sodium dodecylsulfate (SDS), to be analyzed by the SLD's fluorometer. The purposes of these measurements were to verity the SLD's predicted responses and to see if the detergent solution would demonstrate a micelle effect. Cleaning products were chosen to represent 1) strong and weak fluorescence responses as indicated by the spectrophotofluorometer and 2) different types of cleaning compounts. Solutions were made at approximately 2,500 and 25 parts per million by volume. (By comparison, one cupful of cleaning product in an average house's wastewater at 200 gallons/day would be about 300 ppm.) The polarizing filter on the SLD's fluorometer was set as for the septic tank samples discussed in Section II.D.4.a.

The names of the cleaning products and the readings are presented in Table II-D-4. For those solutions that fluoresced, both the meter reading and the zero adjustment required to bring the meter on scale are reported. Those solutions which fluoresced so strongly that both zero and span adjustments together could not bring them on scale, the words "off-scale" arre entered.

As expected, the Ivory liquid dish detergent and the Ajax cleanser showed negligible responses. At the lower concentration, 25 ppm, only the laundry detergent and a fabric softener had readings appreciably higher than the water or soap solution in which they were made. No micelle effect is apparent at this concentration suggesting either that the quantum yields of their fluorescing chemicals are high or that the concentrations were just too low.

At 2,500 ppm all products except Ivory and Ajax fluoresced. Most of the readings were in the range obtained for septic tank samples reported in Section II.D.4.a. Differences between SDS solutions and H_2O solutions were not consistent. The micelle effect may be responsible for the difference in Sta-Puf readings. However, the bleach's fluorescence in detergent solution was actually smaller than in water. The other tree fluorescing products were about the same in both solutions or were off-scale for both.

These tests indicate that fluorescing chemicals are not present in all household cleaning products. However, the products with substantial amounts of fluorescing chemicals, including all of the laundry detergents, can be detected by the SLD at concentrations equivalent to about one teaspoonful of product in a residence's daily wastewater.

c. Natural Organic Materials

The possibility that organic materials present naturally in surface water bodies may produce positive SLD readings is indicated in the manufacturer's operation manual. Vivid indications of this were witnessed during field studies on Crooked Lake, Indiana. (See Section II.D.6.) Shoreline scans detected especially strong fluorescence where streams flow into this lake from

TABLE II-D-4. SLD FLUOROMETER RESPONSES TO SELECTED CLEANING PRODUCTS

Cleaning Product	25 ppm				2500 ppm			
	H_2O solution		SDS solution		H_2O solution		SDS solution	
	Zero	Meter	Zero	Meter	Zero	Meter	Zero	Meter
Distilled H_2O	0	11			0	11		
0.03 M SDS			0	46	0		0	46
Ivory Dish Detergent	0	18	0	43	0	22	0	54
Ajax Cleanser	0	12	0	36	0	14	0	37
Clorox 2 Fabric Bleach	0	17	0	40	Off-scale		296	50
Downy Fabric Softener	0	12	0	40	224	50	216	50
Sta-Puf Fabric Softener	0	8	0	41	53	50	266	50
Tide Laundry Detergent	0	24	0	38	814	50	687	50
Final Touch Fabric Softener	0	30	0	82	Off-scale		Off-scale	

wetlands and in the lake's hypolimnion. The streams were colored light yellow by soluble decay products from vegetation in the wetlands. Fluorescence of the hypolimnetic water may also have been from this source or from decaying vegetation within the lake itself. The lake water was light brown in color.

A sample of the hypolimnetic water was analyzed by an Aminco-Bowman spectrophotofluorometer to determine how its spectral characteristics compared with the brighteners and cleaning products. Curve I in Figure II.D.4 shows the absorbance and fluorescence spectra of the lake water. Its absorbance and fluorescence characteristics are:

Absorbance			Fluorescence		
min.	peak	max.	min.	peak	max.
280	340	375*	375*	416	550

These values are characteristic of the cleaning products discussed above. In fact, had it not been for the color of the hypolimnion water and the field observation that the entire hypolimnion produced very strong SLD readings, the spectral scan alone would suggest that this was a detergent sample.

The result of the micelle effect on the fluorescence of this sample is dramatic as illustrated by Curve II of Figure II.D.4. At a photomultiplier tube voltage of approximately one-third that of Curve I (M.M. = .003 vs. M.M. = .001 for Curve I), the intensity of fluorescence is 2.7 times greater. The overall enhancement is an eight-fold increase. This finding presents the possibility that the SLD may respond positively, not only to actual increases in concentrations of fluorescent materials, but also to constant concentrations, the fluorescence of which is enhanced by detergents present in sufficient amounts to form micelles.

To further explore the fluorescence characteristics of organic materials that might be present in surface waters, laboratory grade tannic acid, humic acid and a lignosulfonate were analyzed with the spectrophotofluorometer. Their spectral characteristics are presented in Table II-D-5.

It can be seen by comparison with Table II-D-1 that humic acid and tannic acid absorb and fluoresce strongly in the wavelengths at which the SLD operates. Overlap with the lignosalfonate is not as complete but is probably sufficient for the SLD to detect this and similar materials.

To determine whether the micelle effect intensified the fluorescence of these laboratory grade organics, an experiment similiar to that performed with Crooked Lake water was conducted with tannic acid. Figure II.D.5 shows the result. The detergent caused a nine-fold gain in fluorescence.

d. Separation of Brighteners and Natural Organics

To investigate the feasibility of identifying which substances in septic tank effluents fluoresce, the mobility of various brighteners and organic materials was determined by thin layer chromatography. Nine brighteners tannic acid, humic acid, and lignosulfonate were separately spotted on both alumina (polar) and Whatman KC18F (non-polor) plates. The mobile phase used

Figure II.D.4. Absorbance and Fluorescence of Crooked Lake Hypolimnion Waste With and Without Added Detergent

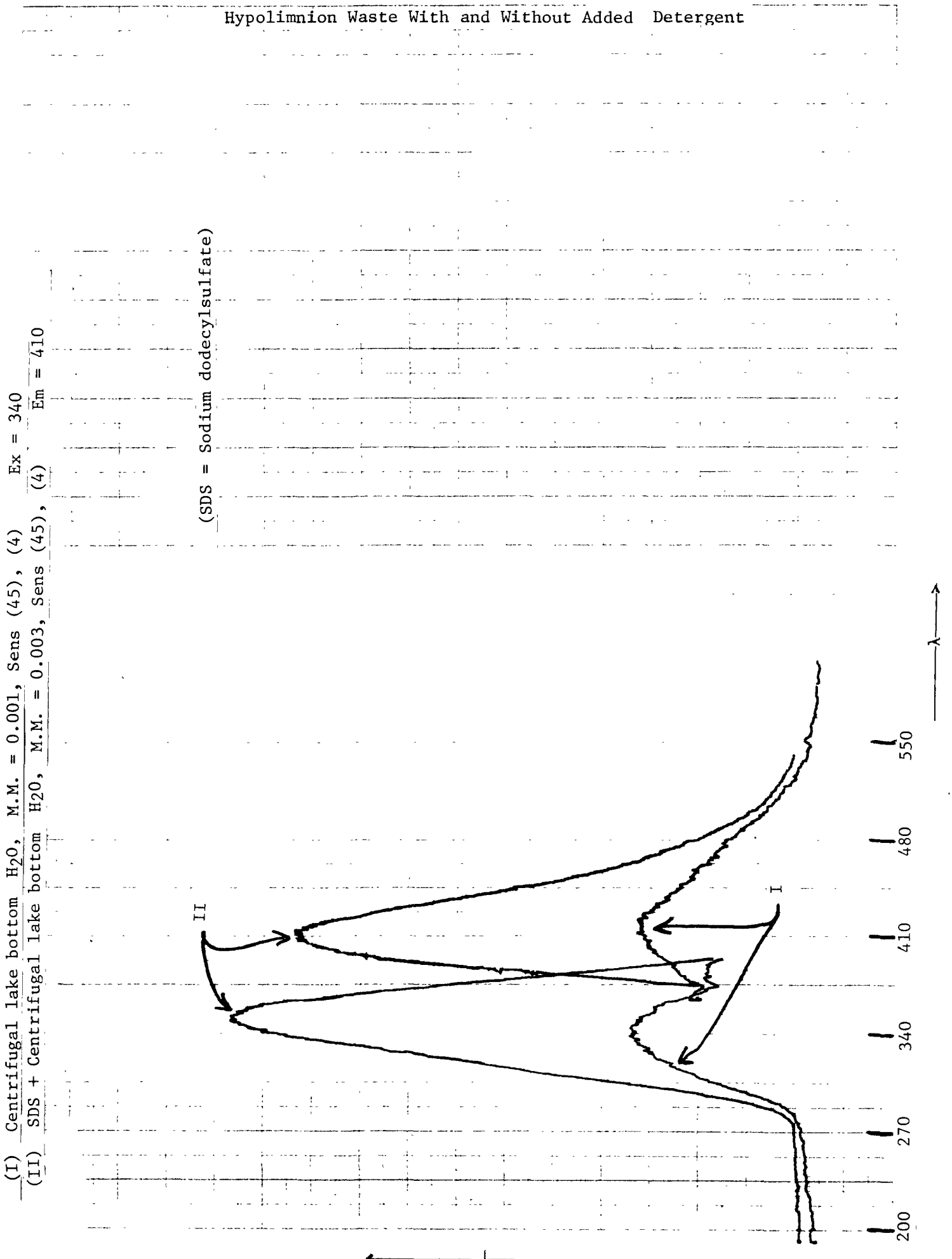
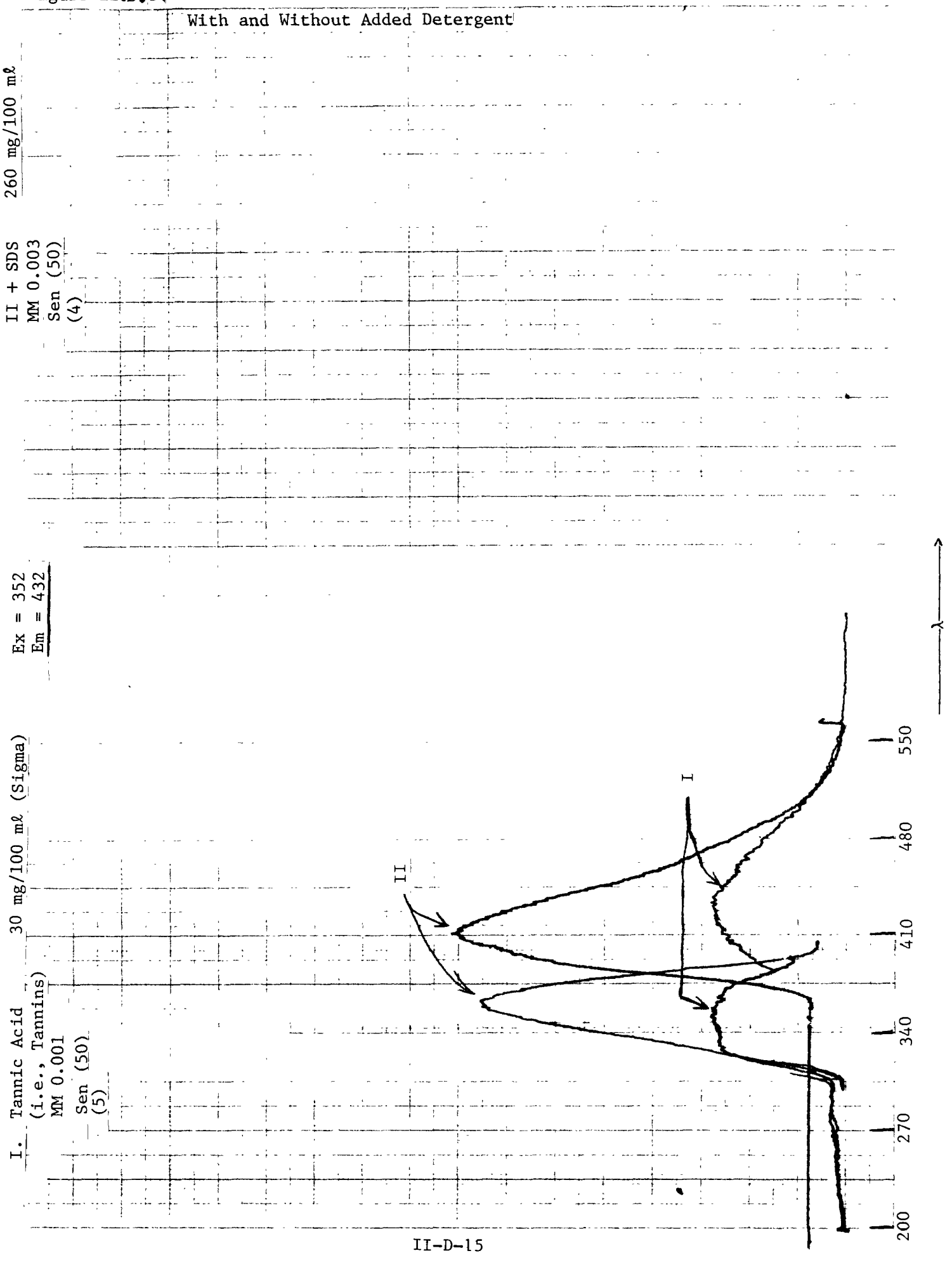


TABLE II-D-5. ABSORPTION AND FLUORESCENCE CHARACTERISTICS OF SELECTED ORGANIC MATERIALS (wavelengths in nannometers)

Organic Material	Absorbance			Fluorescence		
	min.	peak	max.	min.	peak	max.
Humic Acid	280	360	430	410*	465	580
Tannic Acid	310	352	400*	380*	432	550
Poly H (lignosulfonate)	270	310	325*	325*	345	550

* Spectra not complete due to overlap.

Figure II.D.5. Absorbance and Fluorescence of Tannic Acid With and Without Added Detergent



on both plates was a 60:40 mixture of methanol and water. The resulting spots were located by a black light UV source and traced on a transparent overlay. Figures II.D.6 and II.D.7 reproduce these tracings.

Two interesting conclusions can be drawn from these results. First, the mobility of the brighteners differs significantly from that of the organic materials and thereby these substances can be distinguished. The organic materials are non-polar and thus are immobile on the polar solid phase (alumina) but are highly mobile on the non-polar solid phase. In contrast, the brighteners are mobile on both solid phases.

Second, seven of the nine brighteners separated into two or more spots on one or both solid phases. The two brighteners that did not separate, NTS and DSBP groups, were also the only samples to have single absorption peaks and single fluorescence peaks (see Table II-D-2). The samples that separated evidently were mixtures, all of the components of which may not belong to the identified brightener group.

The success of this experiment in distinguishing between laboratory grade brighteners and the other organic materials suggested thin layer chromatography as an appropriate tool for studying fluorescent materials in septic tank effluent as reported in the next section.

e. Discussion

These studies suggest at least four causes of the localized increased in fluorescence to which the SLD might respond:

1) brighteners from cleaning products which flow or leach from on-site systems into surface waters,

2) naturally occuring organic materials released from decomposing vegetation,

3) organic materials released from decomposing human feces and kitchen wastes, both of which have substantial contents of plant material, and

4) increases in fluorescence of any of these materials by means of the micelle effect when sufficient concentrations of detergent are present.

For the practical purpose of locating wastewater or leachate discharges into surface waters, these several potentially interactive sources of localized fluorescence increase do not prohibit meaningful interpretation. Three of the sources are related to wastewater. The naturally occuring organic materials may usually be distingued by their broader spatial distribution and their coincidence with natural features such as inflowing streams, wetlands, and thermal layers in lakes.

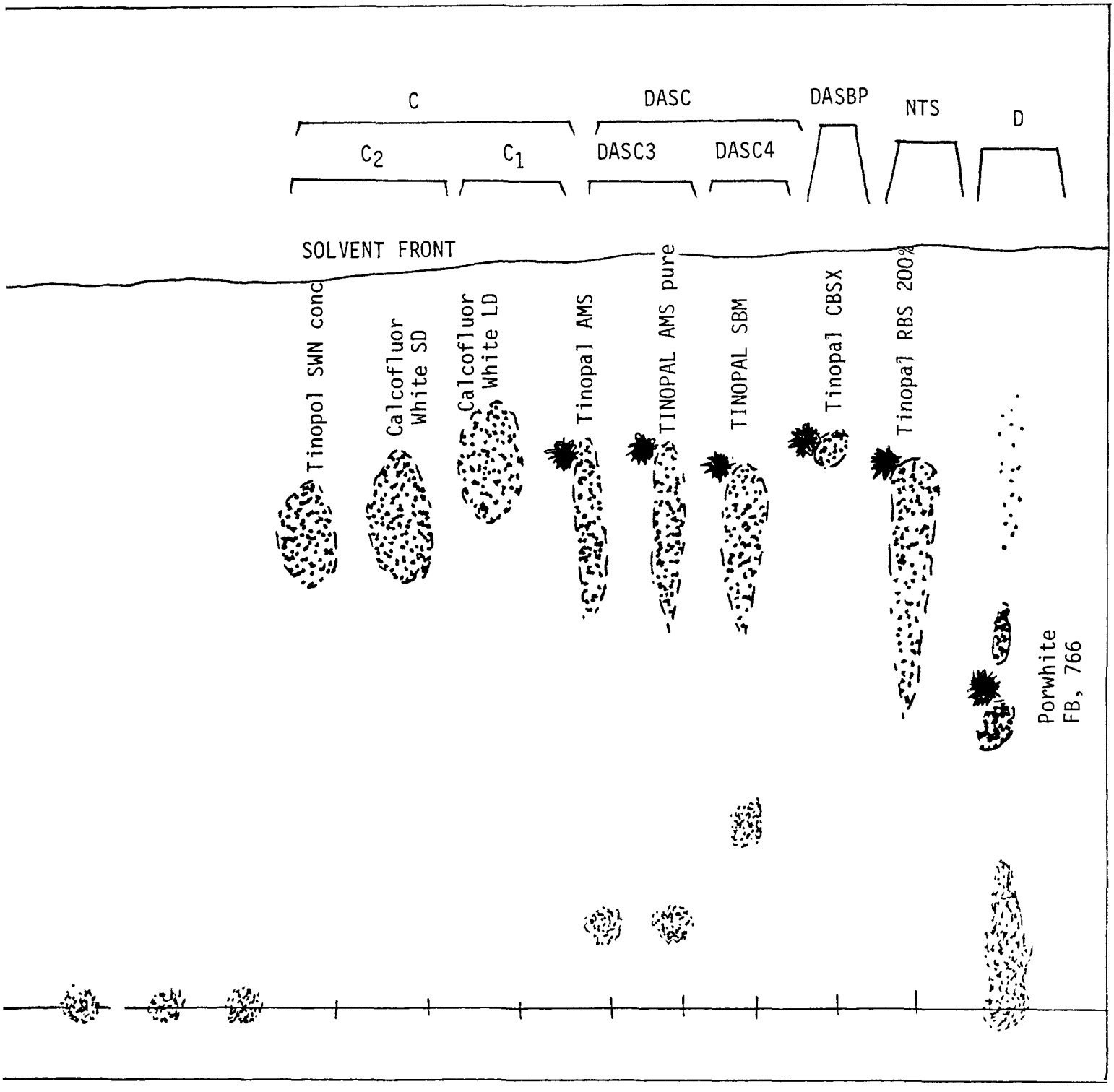
Supposedly the SLD's second recording channel, conductivity, along with its integrated circuit would distinguish between the natural and wastewater sources.

Figure II-D-6.

Figure II.D.6. Thin Layer Chromatography of Laboratory-grade Organic Chemicals on Polar Solid Pha

S. P. = Alumina

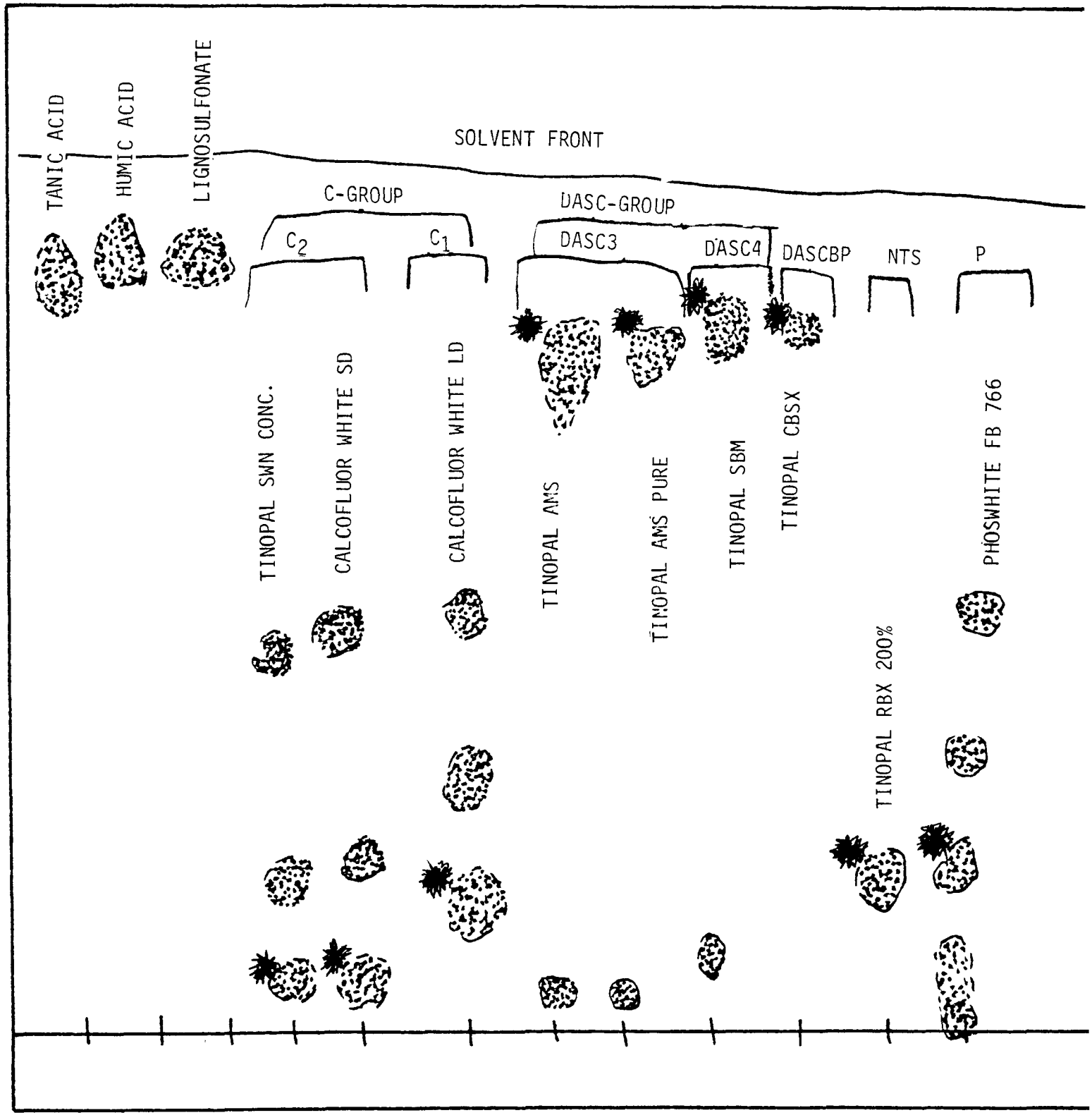
M. P. = 60:40 (v:v) methanol: H_2O



= main "brightener" spot

Figure II-D-7.
S.P. = Whatman KC18F
M.P. = 60:40 (v:v) methanol: H_2O

Figure II.D.7. Thin Layer Chromatography of Laborator grade Organic Chemicals on Non-polar Solid Phase



✱ = main "brightener" spot

4. CHARACTERISTICS OF SEPTIC TANK EFFLUENTS

Early and still the most popular explanations of the SLD's theory of operation emphasized the role of brighteners as the detected material. The possibility of having harmful levels of nutrients or bacteria in effluents or leachates but not fluorescent materials was stirred by this emphasis. To address this possibility as well as to assess the correlations of fluorescence with other chemical parameters of interest, 35 samples were collected from septic tanks in the vicinities of Cincinnati, Ohio, and Alanson, Michigan. The tests performed on these samples and results are discussed below.

a. Chemistry and SLD Fluorescence

Septic tank samples were collected by commercial septage haulers during pumping operations requested by owners of the septic systems. Haulers were requested to collect samples from the liquid layer in tanks so as to approximate septic tank effluent as much as possible. Tha appearance of the collected samples suggested that the haulers were unsuccessful at minimizing collection of settled or floating solids. The samples are considered to be septage, therefore, not effluent.

Samples were tested without filtering for nitrate, nitrite, and total Kjeldahl nitrogen, total phosphorus and methylene blue active substances by the following methods:

Parameter	EPA Reference Method Number*
NO_3-N	352.1
NO_2-N	354.1
Total Kjeldahl Nitrogen	351.4
Total Phosephorus	365.3
Methylene Blue Active Substances	425.1

* "Methods for Chemical Analysis of Water and Wastes." 1979. U.S. EPA, Cincinnati, Ohio. EPA 600/479-020.

Subsamples were filtered through glass fiber filters for analysis by the SLD. Four milliliters of each sample were composited to make a reference solution for instrument calibration. Conductivity of the reference solution and 24 individual septic tank samples was too high for measurement by the SLD. Conductivity measurements for the remaining 11 individual samples ranged widely requiring the full range of both span and zero meter adjustments. Units reported for the inorganic channel readings are combined from these two meter adjustments by the following formula:

$$I = 2{,}000 - I_{50}S + I_{50}Z$$

where:

I = inorganic channel reading in relative units

$I_{50}S$ = span reading when the meter is adjusted to read 50

$I_{50}Z$ = zero reading when the meter is adjusted to read 50.

It has not been determined whether I increases linearly with conductivity.

With two exceptions, samples 16 and 24, organic channel readings could be taken from the zero adjustment necessary to bring the meter to a fixed point (50). All other samples fell into the scale of 26 to 1,000 units on the zero adjustment. Fluorescence is reported in these relative units. The reference solution was used to set the fluorometer's polarizing filter such that organic channel span and zero adjustments of 500 would produce a meter reading of 50.

Results of these tests are presented in Table II-D-6. Concentrations of nitrate and nitrite nitrogen were very low, reflecting the anaerobic state of the samples. Concentrations of total Kjeldahl nitrogen and total phosphorus in these unfiltered samples were substantially higher than average values for septic tank effluent of 10-16 mg/L.-P reported in the literature. This is due to excessive solids in some samples related to the manner of collection.

To assess the correlation between these parameters, linear regression analysis was performed between pairs of parameters. Nitrate and nitrate nitrogen were not included. Table II-D-7 lists the correlation coefficients, slopes, and y-intercepts calculated for the pairs.

Correlations between parameters in the first four pairs are very high with degrees of certainty above 99.9%. However, three of the four pairs that include fluorescence (organic channel on the SLD) have a degree of certainty below 90%. The weaker correlations may have resulted from the fact that SLD tests used filtered samples whereas chemical analyses were run on unfiltered samples.

Of particular note is the degree of certainty of the correlation between the organic and inorganic channel readings. At approximately 85% degree of certainty, this weak correlation throws into question the manufacturer's suggestion that there is a stable ratio ("cojoint signal") between fluorescence and conductivity, and that the SLD can be calibrated to this ratio as a means of measuring strength of wastewater present in surface waters. The data presented here, however, is not sufficient to disprove this part of the manufacturer's theory of operation.

b. Thin Layer Chromatography

Eighteen subsamples of filtered septic tank samples were analyzed by thin layer chromotography using polar and non-polar solid phases with a methanol/water mobile phases as described above in Section II.D.3.d for laboratory chemicals. Subsamples were concentrated approximately 20 times by vacuum evaporation before being spotted on the plates.

Results of the tests on the alumina solid phase are shown in Figure II.D.8 and II.D.9. Figures II.D.10 and II.D.11 show the results on the non-polar solid phase.

TABLE II-D-6. CHEMICAL AND SLD ANALYSIS OF SEPTIC TANK SAMPLES

Sample Number	NO_3-N (mg-N/l)	NO_2-N (mg-N/l)	TKN (mg-N/l)	TP (mg-P/l)	MBAS (mg/l)	SLD Organic channel (relatvie units)	SLD Inorganic channel (relative units)
6	0.026	<0.05	85.0	6.66	4.24	180	968
7	<0.05	<0.05	75.0	7.06	3.97	152	790
8	0.18	<0.05	127.	26.2	5.77	633	-
9	<0.05	0.76	103.	18.7	7.22	502	-
10	0.07	<0.05	163.	29.9	5.14	560	-
11	<0.05	0.06	163.	29.1	1.80	574	-
12	<0.05	<0.05	185.	27.0	3.05	720	-
13	<0.05	<0.05	200.	25.1	4.93	1000	-
14	0.18	<0.05	270.	47.1	2.01	772	-
15	<0.05	<0.05	127.	25.7	8.69	630	-
16	<0.05	0.22	228.	87.0	5.34	-	-
17	<0.05	<0.05	140.	16.3	2.84	638	-
24	<0.05	<0.05	190.	8.30	1.18	-	-
25	<0.05	<0.05	110.	36.3	1.14	456	-
26	<0.05	<0.05	85.0	35.0	1.81	305	1000
27	<0.05	<0.05	152.	34.4	1.74	608	-
28	<0.05	<0.05	152.	35.4	2.13	668	-
29	<0.05	<0.05	163.	30.0	3.38	666	-
30	<0.05	<0.05	128.	34.7	2.80	660	-
31	<0.05	<0.05	153.	35.5	0.676	431	-
32	<0.05	<0.05	190.	34.4	0.578	337	1950
33	<0.05	<0.05	125.	31.3	3.00	808	-
34	<0.05	<0.05	130.	31.0	3.08	372	1621
35	<0.05	<0.05	136.	28.4	2.83	498	1705
36	<0.05	<0.05	500.	540.	20.4	808	-
37	<0.05	<0.05	62.0	11.5	0.40	130	-
38	<0.05	<0.05	123.	58.2	5.56	238	-
39	<0.05	<0.05	100.	28.0	8.48	619	-
41	<0.05	<0.05	33.8	61.1	2.43	460	-
42	<0.05	<0.05	55.0	12.8	0.29	154	-
43	<0.05	<0.05	47.5	12.6	0.90	98	952
44	<0.05	<0.05	49.0	13.9	3.11	188	1786
45	<0.05	<0.05	107.	14.2	1.35	276	1468
46	<0.05	<0.05	16.0	13.8	3.41	98	1820
47	<0.05	<0.05	48.0	8.40	3.13	26	836

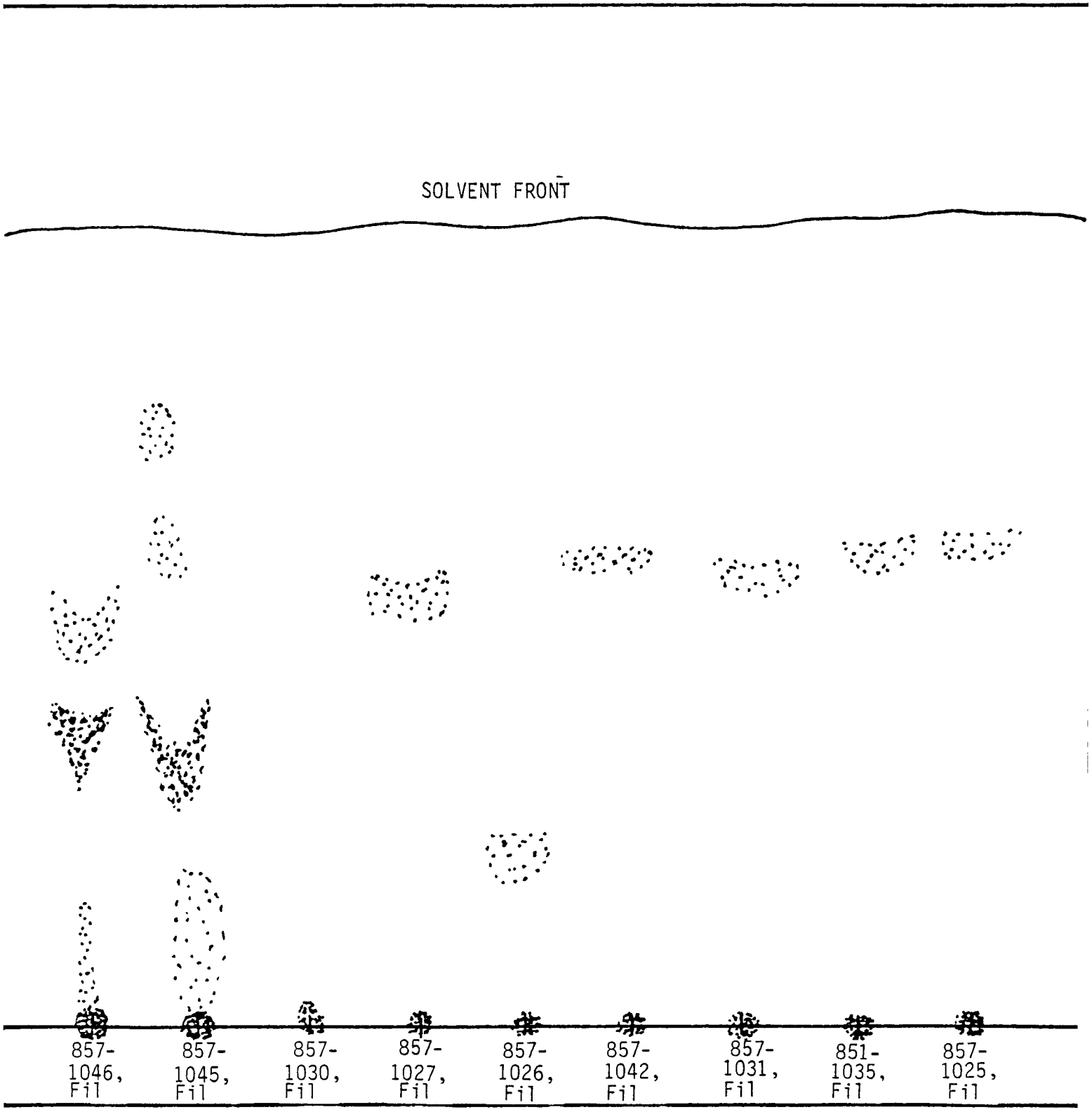
- = Off-scale.

TABLE II-D-7. LINEAR REGRESSION ANALYSIS OF CHEMICAL AND SLD DATA FOR SEPTIC TANK SAMPLES

Parameters X	Parameters Y	Correlation Coefficient	Slope	Y-Intercept
TP	MBAS	.806	.033	2.269
TKN	TP	.788	.814	-67.0
Organic	TKN	.677	.207	30.88
TKN	MBAS	.618	.026	.152
Organic	Inorganic	.505	1.60	986
Organic	MBAS	.293	.0038	1.8067
Organic	TP	.267	.084	1.73

Figure II-D-8.
S.P. = Alumina M.P. = 60:40 (v:v)
methanol:H_2O

Figure II.D.8. Thin Layer Chromatography of Septic Tank Samples on Polar Solid Phase



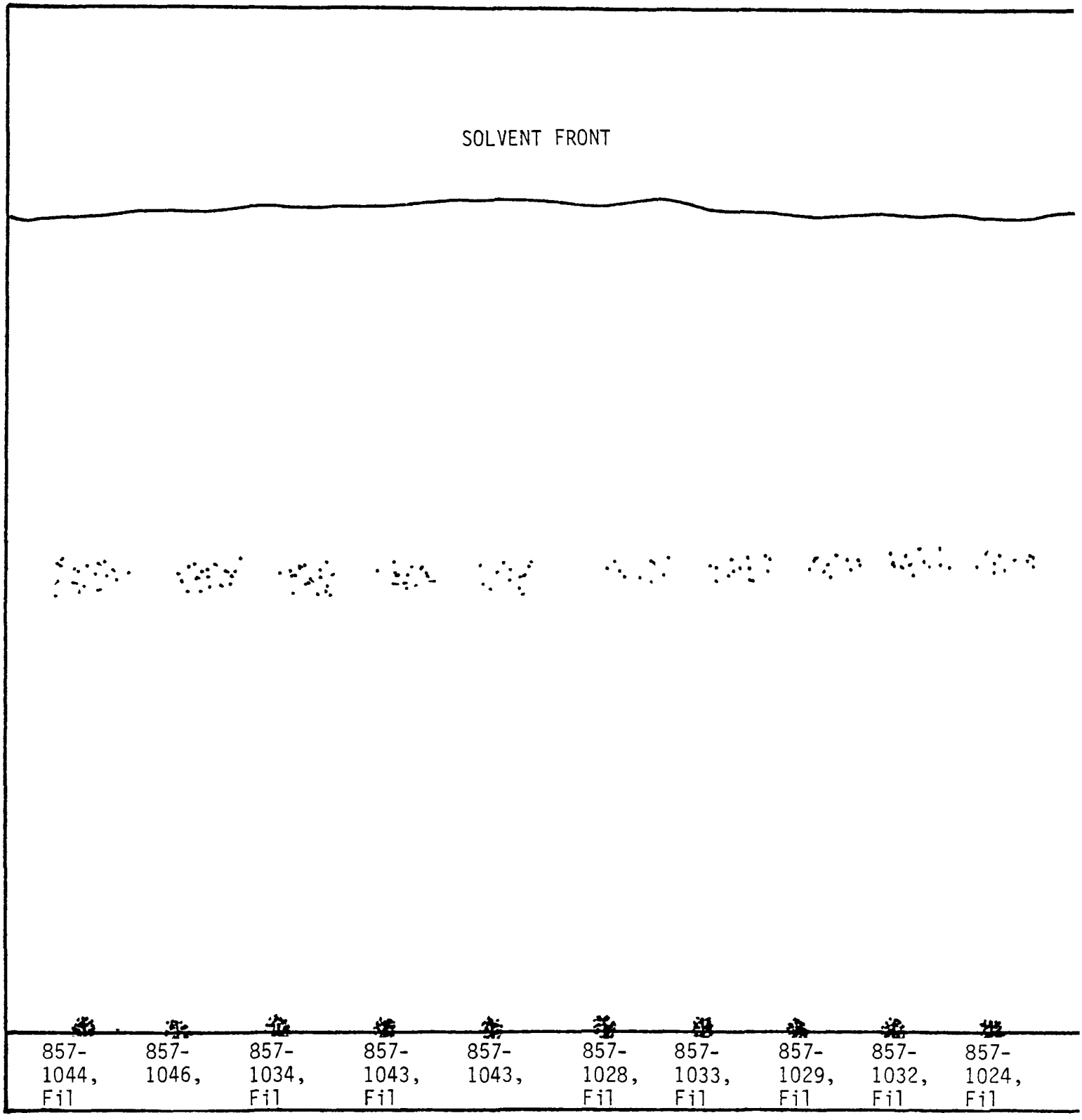
Almost all material remained at origin. Other spots were very very light!

Figure II-D-9.

S.P. = Alumina

M.P. = 60:40 (v:v) methanol: H_2O

Figure II.D.9. Thin Layer Chromatography
Septic Tank Samples on Non-Polar Solid Pha



Almost all material remained at origin.

Figure II-D-10.
S.P. = Whatman KC18F, reverse phase
M.P. = 60:40 (v:v) methanal: H_2O

Figure II.D.10. Thin Layer Chromatography of Septic Ta Samples on Non-Polar Solid Phase

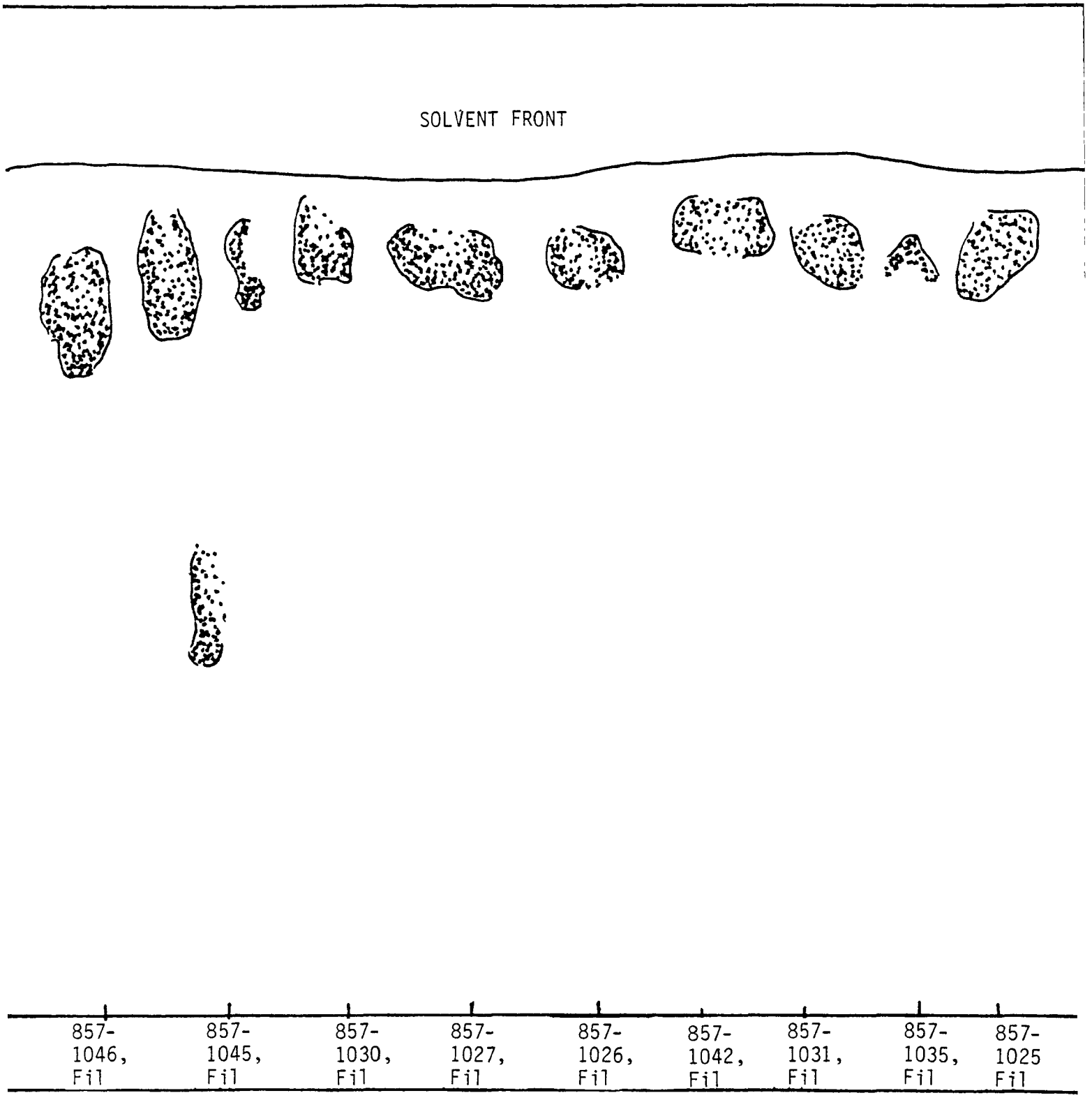
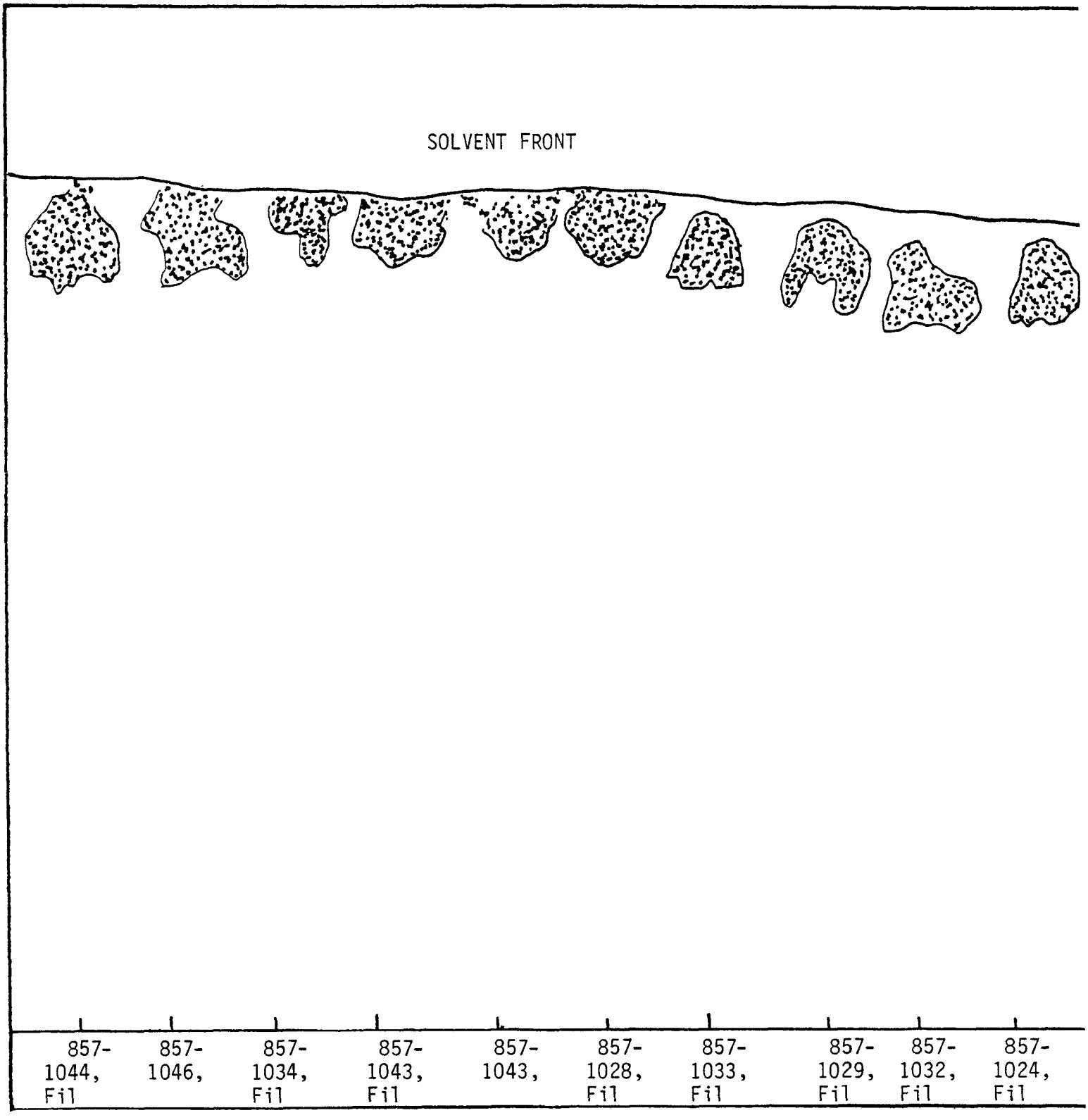


Figure II-D-11.
S.P. = Whatman KC18F, reverse phase
M.P. = 60:40 (v:v) methanol:H_2O

Figure II.D.11. Thin Layer Chromatography of Septic Tank Samples on Non-Polar Solid Phas



Almost all material goes with the solvent front.

A large proportion of each sample behaved like non-polar organic materials and are suspected of being soluble decomposition products of feces and food wastes.

With two exceptions, the amount of fluorescent material that migrated at all like the brighteners was very small and close to being undetectable by visual inspection under black light. By contrast the spots that moved with the solvent front (non-polar solid phase) and that stayed at the origin (polar solid phase) were visible with the unaided eye and were dark brown under black light.

Brighteners are suspected of being the source of faint spots seen in intermediate positions on the polar solid phase. Their intensity and their similarity to the mobility of pure brighteners is great in only two samples. One of the nineteen samples produced no brightener-like spots.

This information strongly suggests that brighteners play only a minor role in the fluorescence of anaerobically treated domestic wastewater.

c. Absorption and Fluorescence

Eighteen of the subsamples analyzed by thin layer chromatography were also analyzed by spectrophotofluorometer. The absorption and fluorescence characteristics of all samples were surprisingly similiar. Absorption peak wavelengths fell within the range of 320 to 340 nannometers. Fluorescence peak wavelengths fell within the range of 385 to 400 nannometers. Minimum absorbance was at approximately 200 nannometers for all samples and maximum fluorescence at 575 nannometers. Figure II.D.12 illustrates a typical recording.

The fluorescence scans of these samples were continued through the entire range of visible light. The purpose for doing this was to seek fluorescence at higher wavelengths which might be used to identify wastewater without the interference of naturally occurring organic materials. The small peak between 600 and 700 nm in Figure II.D.13 is typical of the long wave length response from the samples. This peak was initially though to be fluorescence. However, additional testing showed 1) that the peak disappeared when the sample was filtered (see Figure II.D.13) and 2) that excitation at increasingly higher wavelengths shifted this peak to higher wavelengths (see Figure II.D.14). It was concluded that the 600-700 nm peak was due not to fluorescence but to harmonic effects of light scattered by colloidal material left in the samples after the preparatory filtering through glass fiber filters. Indeed, the peaks of these harmonic responses were all twice the excitation wavelengths.

d. Stability of Fluorescence

Knowledge of the effect of time on the fluorescence of materials in wastewater could be helpful in interpreting fluorescence data from wells or other water resources where time of travel is long. Five filtered septic tank samples that had been stored in opaque containers at room temperatures for three years were measured using the SLD as calibrated for the samples discussed above. Three of the samples had readings of 526,990 and 1,000 and two were off-scale (high). These readings were high compared to the 500 reading for the composite sample used to calibrate the fluorometer.

Figure II.D.12 Absorbance and Fluorescence of a Typical Septic Tank Sample.

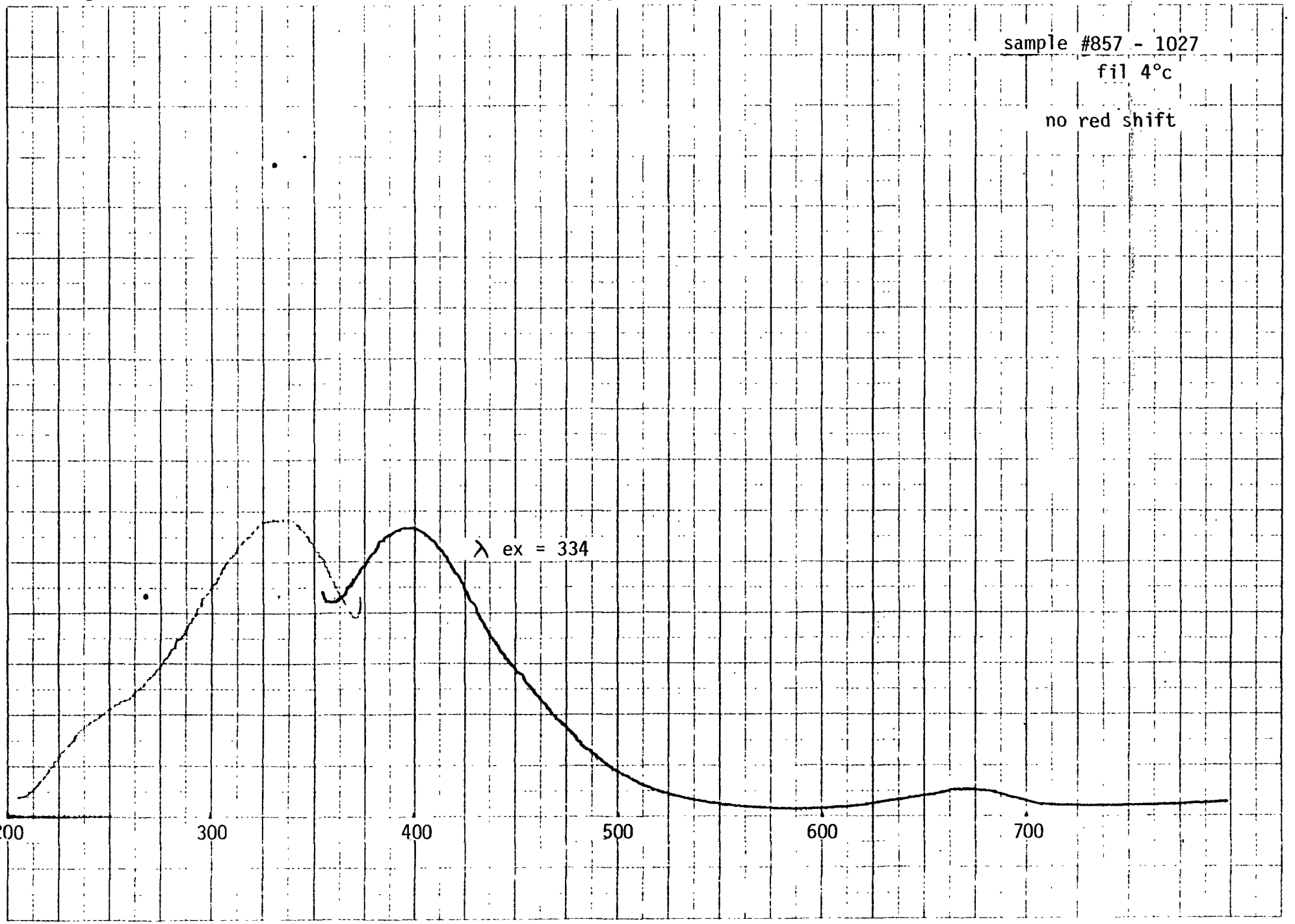


Figure II.D.13 Long Wavelength Response in Non-Filtered and Filtered Septic Tank Sample

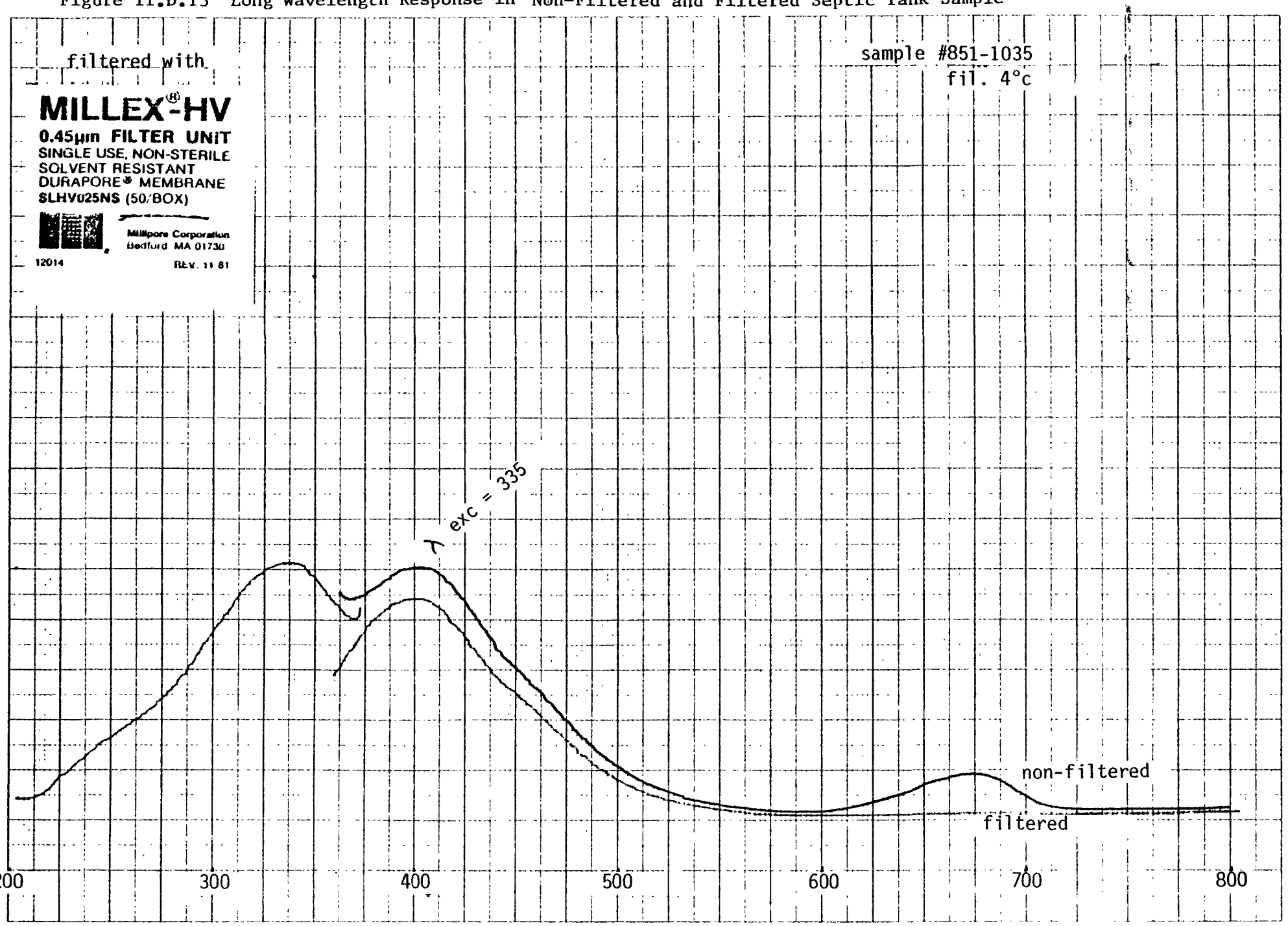
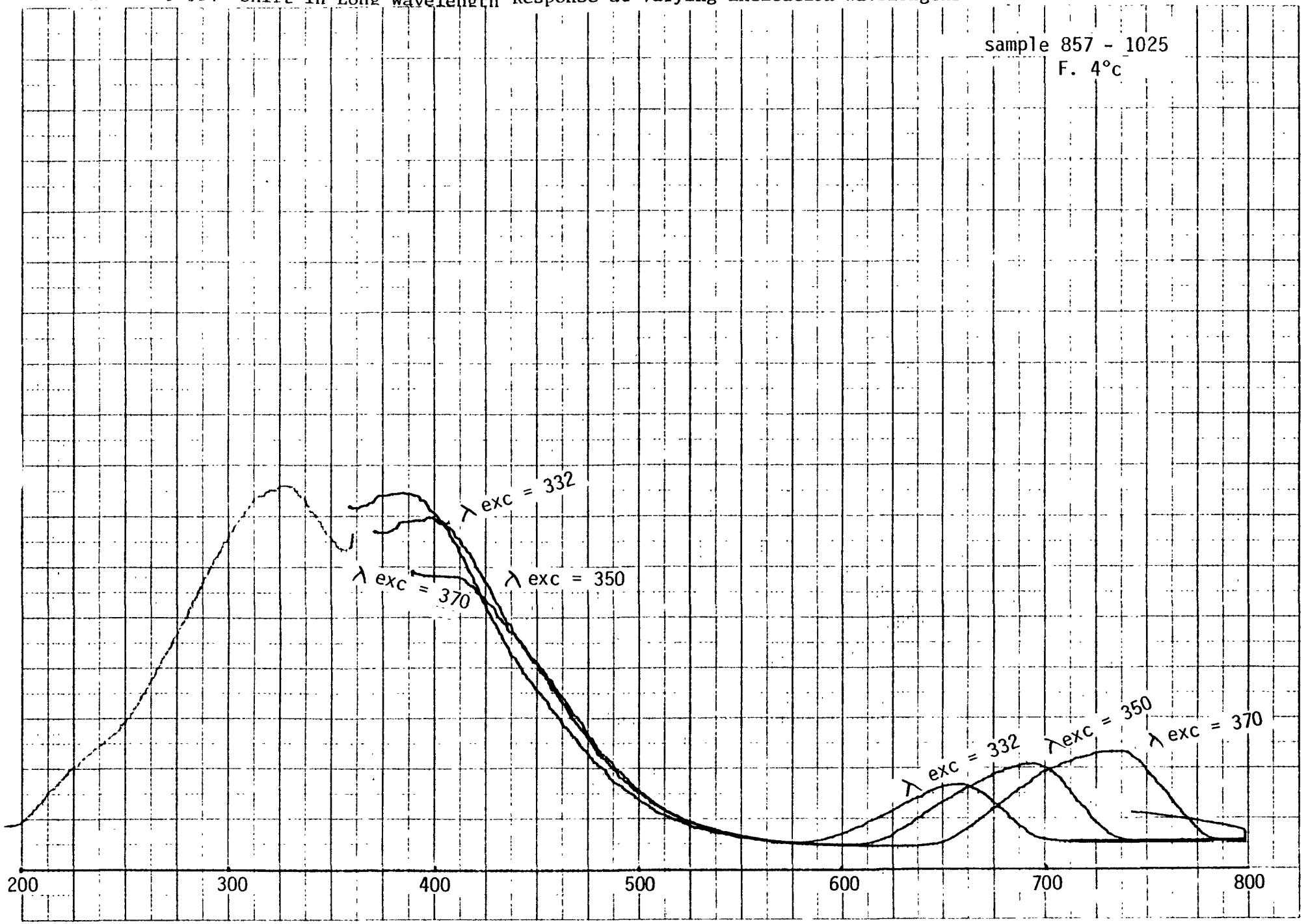


Figure II.D.14 Shift in Long Wavelength Response at Varying Excitation Wavelengths



These higher readings could be due to any of several factors such as source of wastewater or concentration overtime by evaporation. However, the fact that any fluorescence was detected of all indicates that the material which fluoresce are quite stable. Since no attempts were made to sterilize the old samples, it appears that these materials are not subject to bacteria degradation.

5. SOIL COLUMN STUDIES

An attempt was made to measure the passage or orthophosphate, ammonia nitrogen, nitrate nitrogen and detergent whiteners simultaneously through soil columns under saturated conditions. The purpose of this work was to compare the mobility of major nutrients through soils relative to detergent whiteners.

Sandy, sandy clay loan and peat soils were collected from sites near glacial lakes in the midwest. Soils were packed in Plexiglass tubes 5 cm inside diameter by 65 cm length. The sandy clay loam and peat soil turned out to be insufficiently permeable to acquire sufficient volumes of permeate for complete analysis. Therefore, two sand columns were set up and one column was set up with 75% sand soil mixed with 25% of the sandy clay loam.

Columns were fed artificial effluent and dilution water. Based on analysis of major ions in groundwater collected at the soil sampling sites, the dilution water was made of distilled water plus calcium, sodium and magnesium salts to approximate the ionic strength of the groundwater. The effluents were made up to have total dissolved solids concentrations approximately 500 mg/L in excess of the groundwater. The extra ions were added as salts of sodium plus ammonium nitrate. A commercial detergent was hydrolized in weak acid to provide orthophosphate and fluorescent material. Sufficient detergent was used to produce an orthophosphate concentration similiar to what is typical of domestic wastewaters, 12 mg/L-P.

Effluent solution alone was passed through Sand Column #1 for three and on-half hours until breakthrough of orthophosphate, nitrate and ammonia were apparent. Then dilution water alone was passed through the column for seven more days to trace the wash-out of nutrients and fluorescent materials. The data for Sand Column #1 are presented in Table II-D-8. Ammonia and nitrate concentrations and conductivity in the permeate fell rapidly to low, relatively stable levels within about 12 hours. Phosphorus concentrations fell to about 1 mg/L after 24 hours and remained at that level for the remainder of the test. Apparent fluorescence did not reach a peak until 20 hours after effluent application was ceased, then fell to a low level within 12 hours. Absorbed fluorescent material may have started leaching out after the third day as suggested by the increasing fluorescence levels.

Sand Column #2 received mixed effluent and dilution water from start to finish over the same eight day period. The ratio of dilution water to effluent was not carefully measured but is estimated to be 10:1 to 5:1. Nutrients and fluorescence in the permeate from this column reached a somewhat erratic equilibrium within 24 hours as shown in Table II.D.9. Fluctuations in all parameters was probably due to varying ratios of feed solutions resulting from differentially falling lead losses in feed bottles.

TABLE II-D-8. SAND COLUMN #1 - PERMEATE DATA

Date	Time	PO_4 (mg/L-P)	NO_3 (mg/L-N)	NH_4 (mg/L-N)	Conductivity	Fluorescence (Meter reading)
15 July	2:00 p.m.	0	4	4	167	
	2:20	0	19	8	247	
	2:25	0	26	7	264	
	2:40	.74	17	7	205	
	3:05	.77	6	3	170	
	3:27	.85	6	2	175	
	3:50	.15	42	10	197	
	4:10	1.1	91	24	355	
	4:40	1.5	115	81	484	
	5:30	4.7	26.52	19.6	301	
	8:00		31.74			14
	11:00 p.m.		>61.26			20
16 July	4:50 a.m.		2.88			22
	10:00	3.8	1.86	<	197	-
	8:00		1.98			25
	9:00		.96			26
	10:30		1.38			32
	11:45 a.m.		.90			28
	12:50 p.m.	1.8	1.08	<	195	46
	1:55		.78			31
	2:45					31
	3:55					29
	5:00					26
	5:55					27
	6:45 p.m.	.6	.66	<	192	39
	7:35					18
	8:45					16
	9:50					19
	11:00					19
	11:45 p.m.	1.0	.36	<	214	14
17 July	8:00 a.m.					13
	9:50					15
	12:15 p.m.					19
	2:15					16
	3:10	.7	4.32	<	236	23
19 July	4:00 p.m.					20
20 July	4:30 p.m.					22
22 July	8:45 a.m.	1.0	.12	<	200	38
	4:00 p.m.					26

TABLE II-D-9. SAND COLUMN #2 - PERMEATE DATA

Date	Time	PO_4 (mg/L-P)	NO_3 (mg/L-N)	NH_4 (mg/L-N)	Conductivity	Fluorescence (Meter reading)
15 July	5:30 p.m.	.4	2.16		168	-
	8:00 p.m.					50
16 July	5:30 a.m.					26
	9:20	1.1	60.48	29.68	354	
	10:30					31
	11:45					18
	12:50 p.m.					32
	1:55					20
	2:45	6.68	13.75	24.05	237	34
	3:55					47
	5:00					19
	5:55					15
	6:45					19
	7:35	6.70	9.60	6.72	227	15
	8:40					13
	9:50					13
	11:15					14
17 July	8:00 a.m.					13
	12:15 p.m.	.4	3.18	<	225	41
	3:10 p.m.					19
	-					16
18 July	2:00 a.m.					20
	5:55 a.m.					20
	5:00 p.m.					20
	11:30	5.8	54.30	8.68	252	21
19 July	4:00 p.m.					29
20 July	4:30 p.m.					32
21 July	4:00 p.m.					42
22 July	8:45 a.m.	8.0	1.08	30.66	241	46
	4:00 p.m.					35

Sand and Clay Column #3 was fed solutions like Column #2 but over a longer period of time since it took much longer to produce ammonia or phosphorus breakthrough in the permeate as shown in Table II-D-10. Unfortunately only a few fluorescence measurements were taken but these showed a higher level than either of the two sand columns. This could be due to a lower dilution water:effluent ratio, to natural fluorescent materials leaching from the soil to lesser retention on the soil. Unfortunately, insufficient notes were kept regarding feed rates to confirm or reject that hypothesis. No control columns were leached with the dilution feed to determine if the solis themselves produced fluorescent materials. The reason for higher fluorescence in this column cannot, therefore, be assessed.

6. FIELD INVESTIGATIONS

Surveys of septic tank effluents along shorelines were conducted previously in seven study areas examined by Region V. Taken together those surveys located few effluent plumes relative to the number of lakeshore dwellings. One major lake, Otter Tail Lake, Minnesota, was surveyed twice with radical differences in the number of plumes identified (K-V Associates, 1979a and b). Information provided by the surveyor, Dr. William Kerfoot, also suggested that the emergence of effluent plumes into lake waters is a dynamic process. Variability in groundwater flow, changing wind and water currents, and seasonal use of dwellings were all recognized as influencing how detectable plumes might be and whether they might even appear and disappear intermittently.

The low number of plumes detected in some lakes raised questions about the location of plume emergence. Surveys were conducted as close to shorelines as possible assuming that effluents flow with an unconfined water table to the lakeshore. However, it clay lenses or other continuing layers intervened, effluent could emerge away from shore or at other unexpected locations.

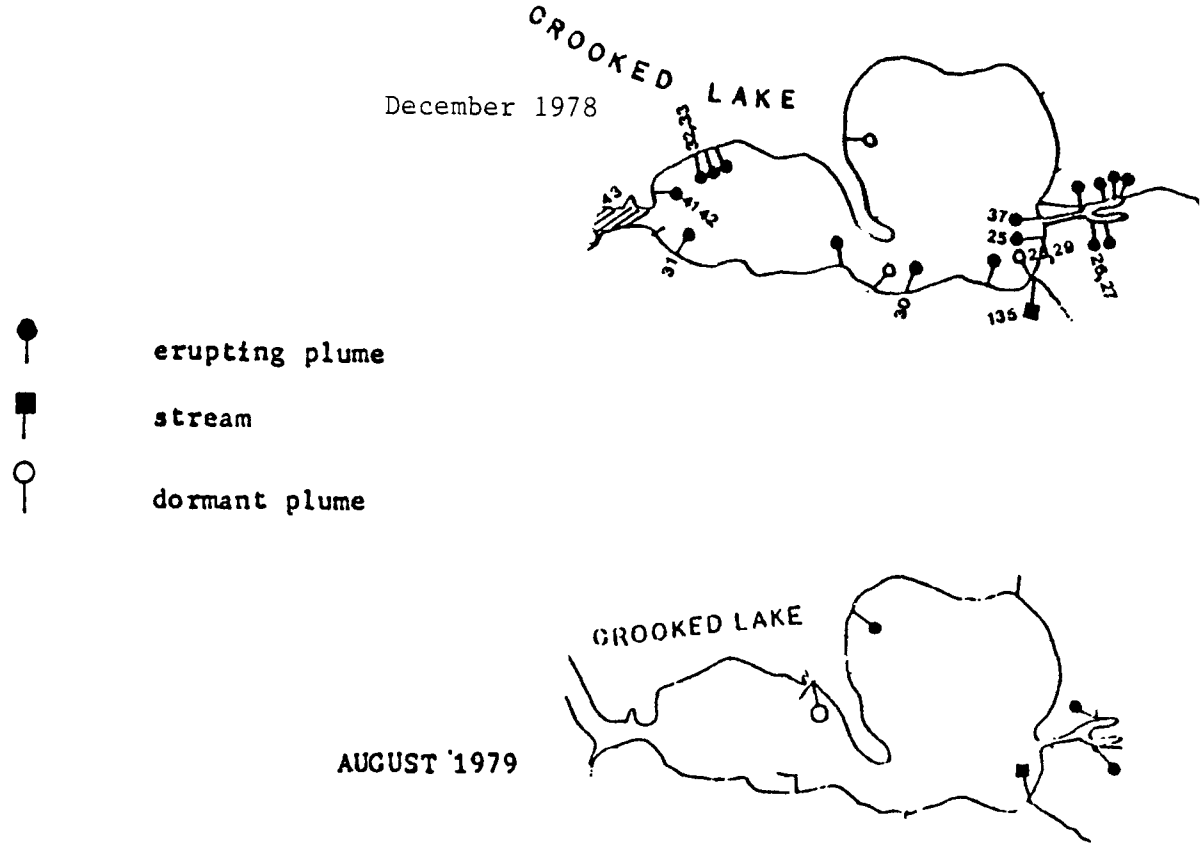
To address these concerns and to gain operational familiarity with the equipment, field investigations were conducted from August 26 to September 15, 1980. The site selected for this work was the first and second basins of Crooked Lake, Indiana. Crooked Lake was chosen because 1) a good data base had been developed on it during preparation of one of the seven rural lake environmental impact statements, and 2) the percentage of lakeshore homes with plumes during previous surveys was especially low.

a. Plume Stability

Two leachate surveys had beem conducted on Crooked Lake and groundwater flow patterns had been determined (K-V Associates, 1979c and d). Figure II.D.15 shows the outline of this lake with the plumes located during these earlier surveys and the groundwater flow information. The December 1978 survey, even though conducted several months after the summer recreation season, detected more plumes (19) than the subsequent August 1979 survey (4).

To evaluate plume stability during the 1980 field investigations, three heavily developed shoreline segments facing the main body of the lake were surveyed on multiple occasions. Segments are illustrated in Figure II.D.16. During initial scans of these segments only the organic channel's signal was recorded. Later scans were recorded on the combined mode for comparison. During all shoreline scans the span adjustments on both channels were set for maximum response to changes in conductivity and fluorescence.

Figure II.D. 15 Effluent Plumes and Groundwater Flow Recorded during Previous Studies (K-V Associates, 1979 c and d)



Direction and rate of Groundwater Flow at selected stations around the Steuben Lakes Shoreline, August 1979

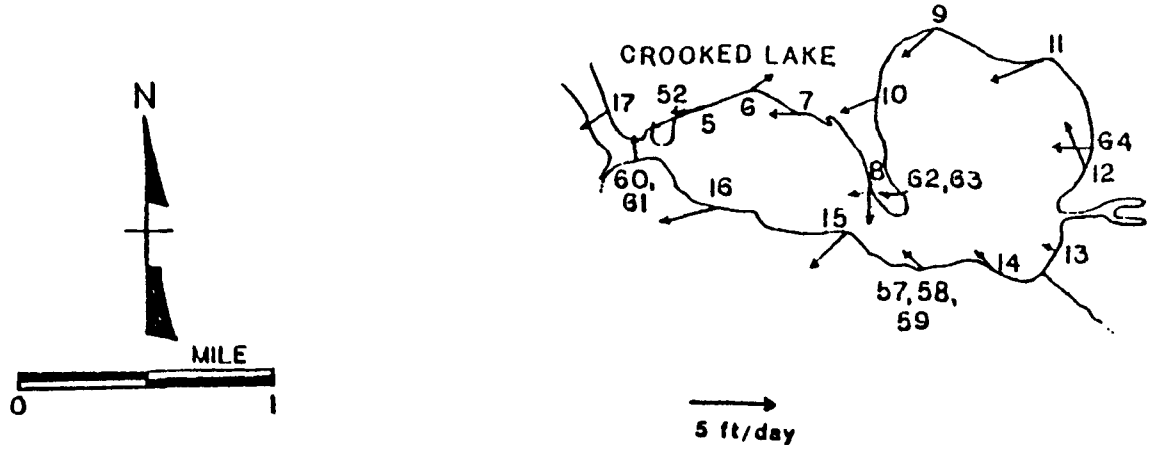
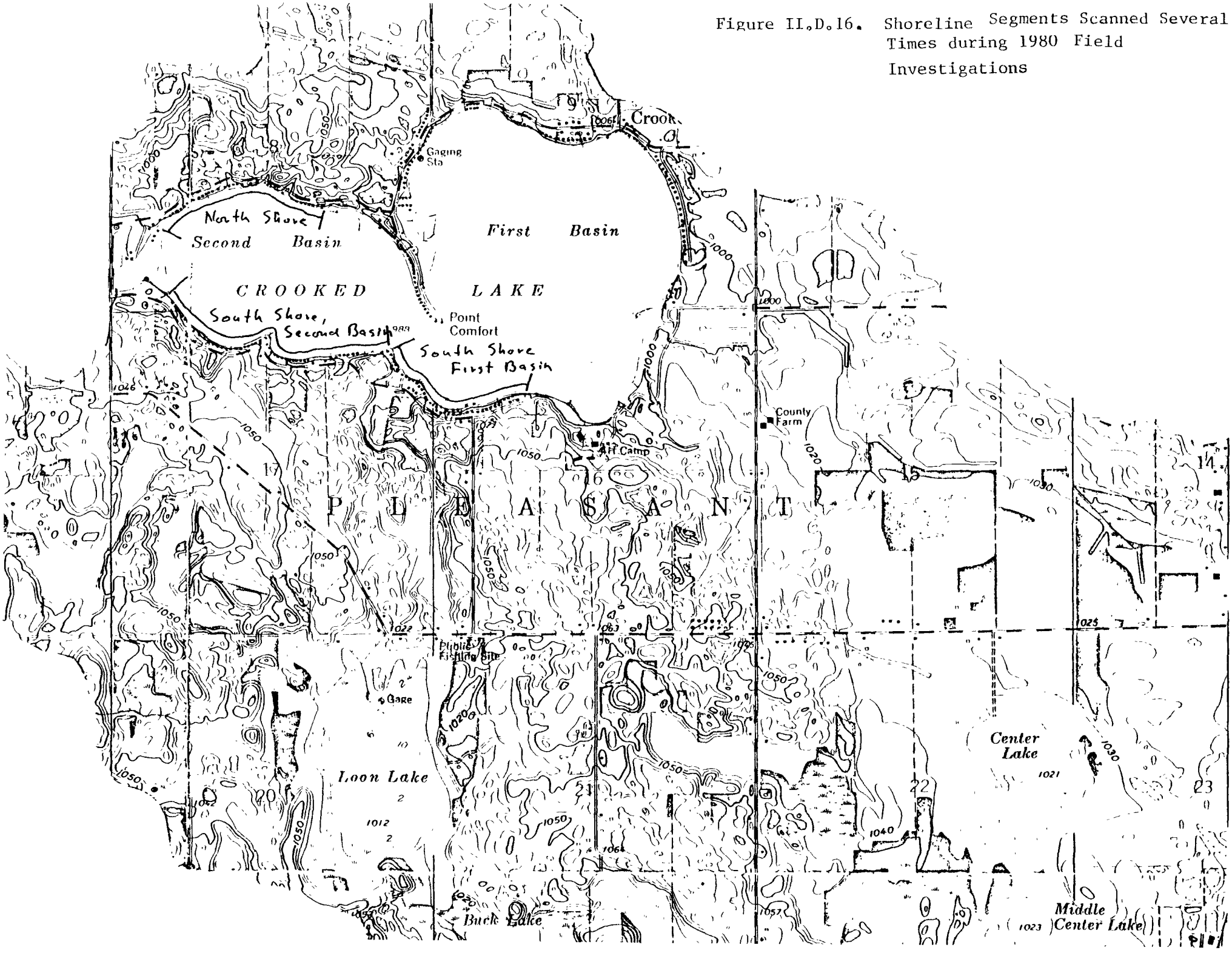


Figure II.D.16. Shoreline Segments Scanned Several Times during 1980 Field Investigations



II-D-36

TABLE II-D-10. SAND AND CLAY COLUMN #3 - PERMEATE DATA

Date	Time	PO_4 (mg/L-P)	NO_3 (mg/L-N)	NYY_4 (mg/L-N)	Conductivity	Fluorescence (Meter reading)
16 July	9:00 a.m.	.4	1.20	<.01	169	-
	7:35 p.m.	.4	.60	<.01	188	58
17 July	9:50 a.m.	-	6.12	<.01	226	61
18 July	8:45 a.m.	.3	103.68	~.01	335	49
19 July	4:00 p.m.	.3	26.88	<.01	244	53
20 July	4:30 p.m.	.4	55.44	<.01	303	-
23 July	4:55 p.m.	.02	24.48	<.01	321	-
25 July	8:30 a.m.	.15	13.80	3.20	294	-
27 July	7:50 p.m.	.94	1.89	5.04	271	-
30 July	9:20 a.m.	<.05	.97	6.05	330	-

Figure II.D.17 shows where recordings typical of effluent plumes were obtained within these segments at one time or another during the three week period. Most of these recordings document very small changes in organic or combined signals. Examples of the smallest and the largest responses judged in the field to be attributable to wastewater sources are illustrated in Figure II.D.18. For comparison, Figure II.D.18 also shows part of a recording made at a fixed location where no wastewater influence was suspected.

Only 5 of the 25 plumes identified in the three segments were present during every scan. Logs for each segment and discussion of significant observations suggest reasons for this.

South Shore, First Basin

Date	Time	Weather	Recording Channel	Comments
29 Aug.	8:30-9:00 a.m.	Very slight breeze	Organic	Flat response except 2 distinct peaks.
2 Sept.	4:50-5:26 p.m.	Strong wind in morning; moderate during scan.	Organic	Flat response except small peak at a place recorded 29 Aug.
14 Sept.	10:04-10:30 a.m.	Moderate wind NNW.	Organic	Flat response.

Access to the shoreline was difficult in the middle of this segment due to a shallow, pebble-paved bottom. Some plumes in this stretch may have been missed, but none were found on either side where shoreline was easily accessible.

One of the two plumes was particularly strong during the first scan and was the only plume detected in the second scan. But it was not detected during the last scan either because of the wind or because the wastewater source is a direct discharge. The building was not in use the day of this last scan.

South Shore, Second Basin

Date	Time	Weather	Recording Channel	Comments
30 Aug.	8:31-9:25 a.m.	Moderate breeze from south.	Organic	Shoreline in the lee of wind. 7 plumes.
30 Aug.	2:02-3:10 p.m.	Slight breeze	Combined	12 plumes. Under similiar conditions, the combined mode located 5 additional plumes.

Figure II.D.17 Suspected Effluent Plumes Detected within Designated Segments, 26 August - 14 September, 1980.

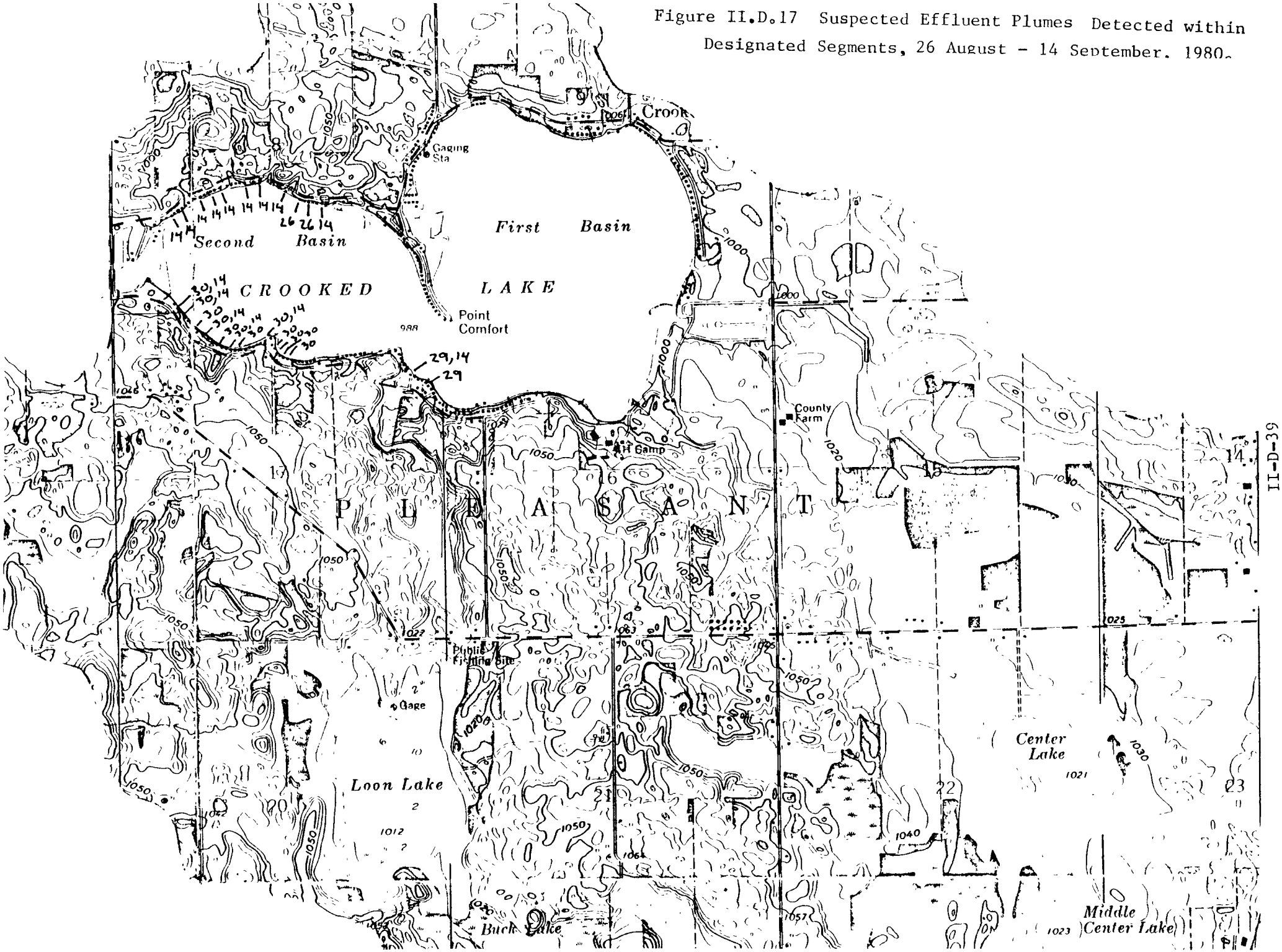
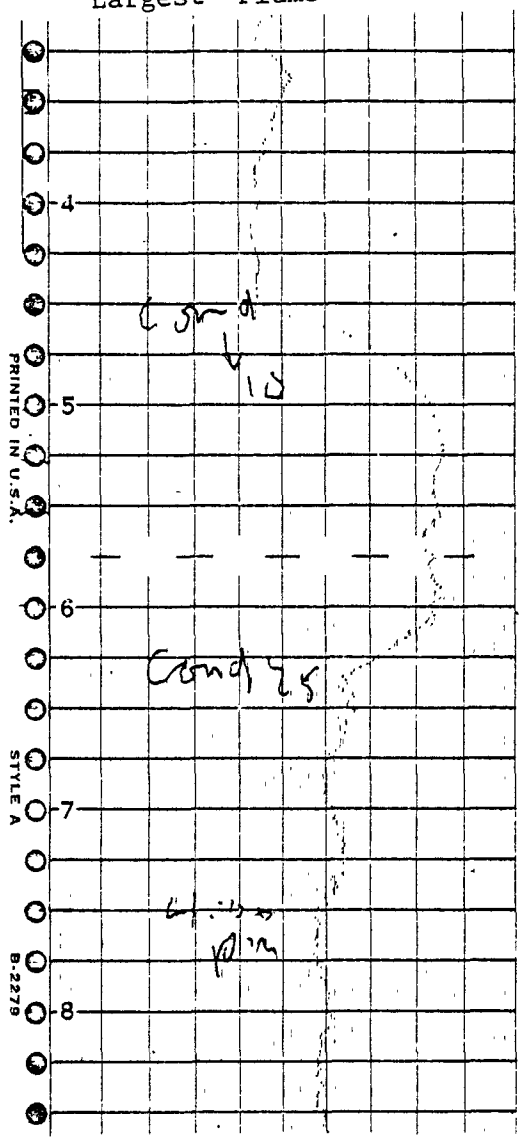
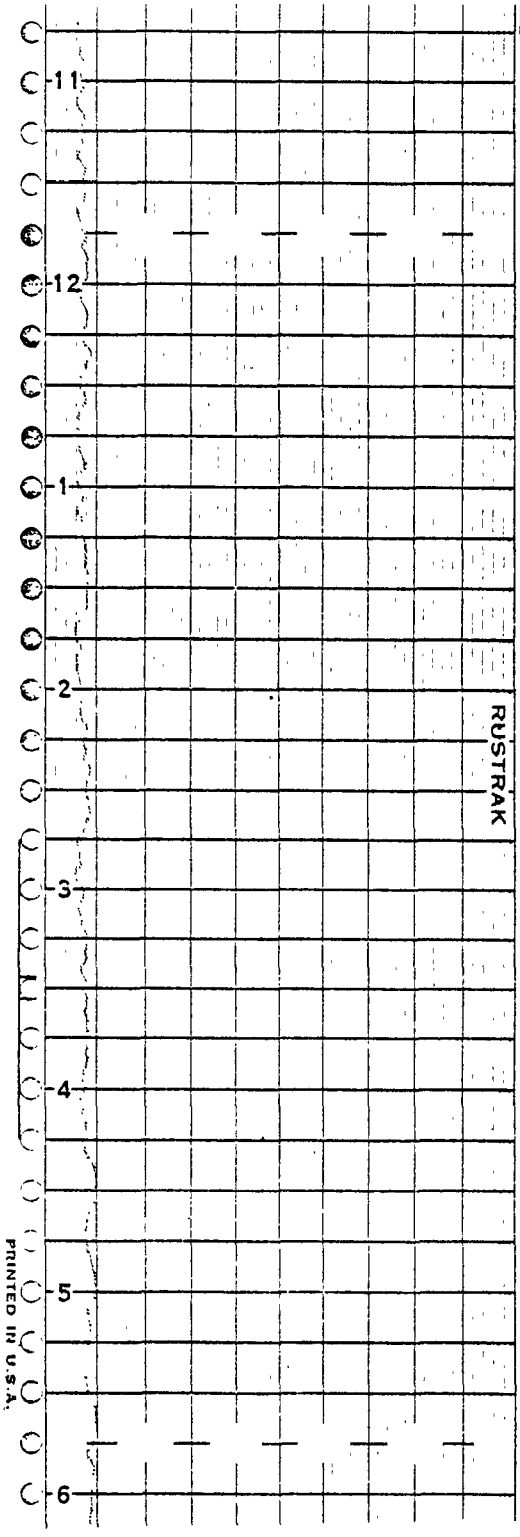


Figure II.D. 18 Examples of SLD Responses Smallest Plume, Largest Plume and Constant Background

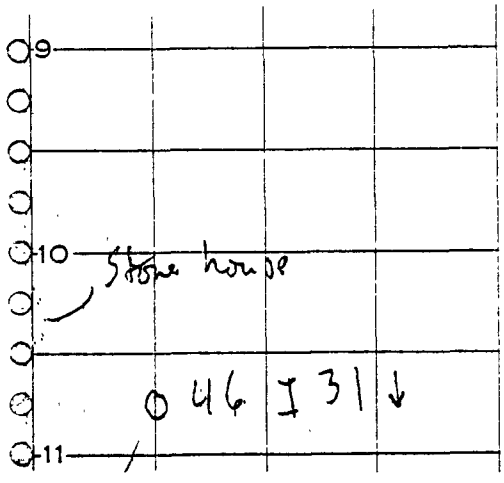
Largest Plume



Background



Smallest Plume



South Shore, Second Basin (continued)

Date	Time	Weather	Recording Channel	Comments
14 Sept.	10:33-11:15 a.m.	Strong breeze from north.	Organic	5 plumes detected despite strong on-shore breeze.

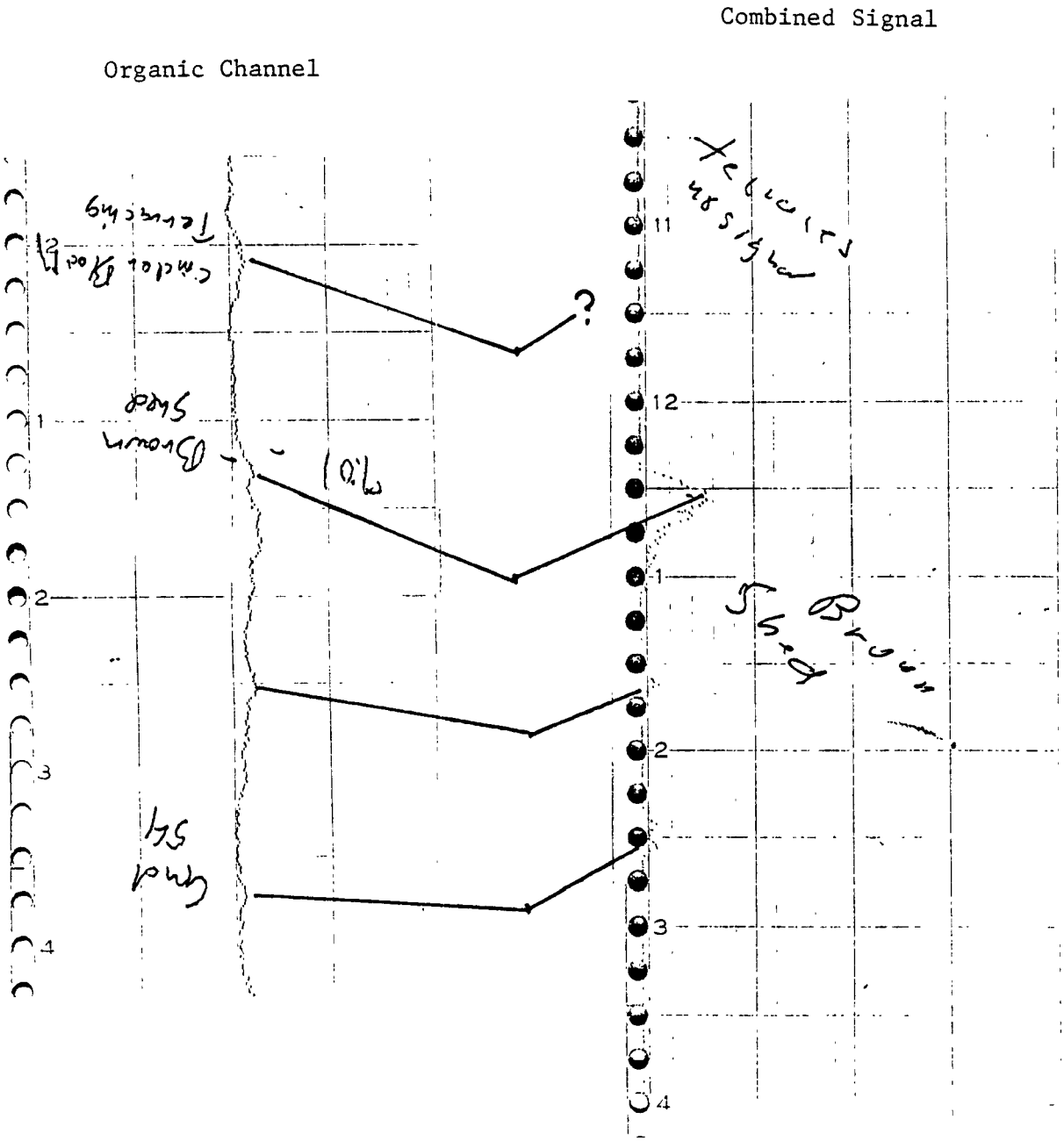
The morning and afternoon scans on 30 August provided the first of two planned comparisons between organic channel and combined channel recording. Figure II.D.19 shows the two recording modes for the same part of this segment. While the organic channel recording is useful in identifying large-scale variations in fluorescence that may be due to streams or wetlands, the combined channel performs rapid data analysis that allows the operator to devote greater attention to making and recording observations about house locations, shoreline conditions, weather, etc. The greater number of plumes detected in the afternoon may have been due either to calmer winds or to enhanced data display by the combined channel.

The five plumes detected on 14 September were the only ones on Crooked Lake that were detected during every scan.

North Shore, Second Basin

Date	Time	Weather	Recording Channel	Comments
26 Aug.	3:40-4:55 p.m.	Not recorded	Organic	Two peaks, one abnormally large off of a lot with construction activity.
30 Aug.	11:08-11:44 a.m.	Moderate breeze from west.	Organic	No discernable wastewater peaks. Wind-generated waves.
31 Aug.	5:25-6:33 p.m.	Very light breeze from southwest. 1" rain mid-day.	Organic	No discernable wastewater peaks. Heavy boat waves.
14 Sept.	11:45 a.m.-12:20 p.m.	Strong breeze from northwest	Organic	Shoreline on lee, protected by hills. High background fluorescence with broad variations. 7 plumes.
14 Sept.	12:30-12:55 p.m.	Wind shifting to west.	Combined	Many peaks at beginning of scan in lee. No peaks toward end of scan out of lee.

Figure II.D.19 Comparison of Organic Channel and Combined Signal Recordings; South Shore, Second Basin; 30 August



North Shore, Second Basin (continued)

Date	Time	Weather	Recording Channel	Comments
14 Sept.	12:58-1:21 p.m.	Wind shifting to west.	Combined	Similiar to prior scan.
14 Sept.	3:29-3:46 p.m.	Moderate breeze from southwest.	Combined	Similiar to two previous scans.
14 Sept.	4:07-4:55 p.m.	Moderate breeze from southwest.	Combined	Walking scan getting probe very close to shore. Fewer and smaller peaks than previous scans. Much lower background fluorescence than in morning.
14 Sept.	4:55-5:09 p.m.	Slight breeze from southwest.	Combined	All but 3 peaks disappeared.

The large peak recorded during the 26 August scan was the largest wastewater source detected during the field investigations. Later discussions with the occupants of the nearby house revealed that repairs had been made to the on-site system earlier that day and some of the wastewater may have entered the lake. This source was not detected during any later scans.

No peaks were identified during scans of this segment 30 and 31 August. Wind and waves may have dispersed any plumes.

The series of six scans on 14 September produced a considerable amount of information and raised some interesting problems in interpretation of recorded data. Large numbers of apparent plumes were detected during the first four scans in the western two-thirds of this segment. This part of the segment was protected from strong, steady winds by hills and trees next to the lake. Few recording peaks were generated outside of this lee. As the wind shifted to the south and calmed somewhat in late afternoon, the number and intensity of peaks decreased until very few could be detected by 5 p.m. Also, the level of background fluorescence decreased in the afternoon necessitating a recalibration of the organic channel before the next to last scan.

There are two interpretations that can be made of the recordings for this segment. First, the absence of many plumes during all scans except for the lee area on the 14th of September could be explained by dispersal of plumes by waves and wind-generated currents. The emergence of effluent plumes may even have been induced by a lowered lake elevation on the lee side caused by the wind which accelerated groundwater, and effluent, flow toward the lake.

Alternatively, the source of fluorescence and conductivity increases may have been the lake's hypolimnion, raised and disturbed by wind-generated currents in the epilimnion. The presence of high fluorescence and conduc-

tivity in the hypolimnion was demonstrated by exercises described later in this section. If this interpretation is valid, there may have been no wastewater plumes in this segment at all on the 14th. Unfortunately, this possibility was not recognized until after the opportunity to do verification exercises had passed.

b. Plume Emergence Points

The possibility that effluent plumes might be entering this lake some distance from shore was investigated by running transects with the SLD perpendicular to the shore. To do this a modified probe was constructed with a 1,200 gallon per hour submersible pump and twenty feet of flexible tubing. The boat was towed by a SCUBA equipped diver who held the probe approximately one foot above the sediments while swimming toward the center of the lake.

Locations of the transects are shown in Figure II.D.20. These locations were chosen because 1) groundwater flow was previously determined to be toward the lake, 2) the shorelines are developed, but 3) few or not effluent plumes had been detected.

At the end of two transects from the south shore, the direction of tow turned to follow ledges to check the possibility that the drop-offs might intersect clay lenses or other confining layers.

No localized increases in fluorescence were detected. Along transects from the east shore of the First Basins, the organic recordings increased gradually along the shallow, even grade. Figure II.D.21 reproduces the recording of the middle transect.

A similar but more dramatic increase was recorded from transects on the south shore. In particular, the recording for the transect starting at the point between the two basins went off-scale within 50 feet of shore and 19 feet depth. A similar rapid increase for the transect from the south shore of the First Basin is also reproduced in Figure II.D.21. Conductivity, which was noted separately, also increased rapidly at this depth.

The depth where the increases occurred, and improved transparency at this depth noted by the diver suggested that the signals were not caused by localized effluent plumes but were characteristic of the lake's hypolimnion.

To check this possibility, the probe was lowered to 20' in deep water 250' off shore in the southwest part of the First Basin, the zero adjustment for the organic channel was increased to bring the meter back on scale, and the boat was allowed to drift westward with the wind. Figure II.D.22 reproduces part of the recording. The wide, repaid fluctuations suggest that the probe was skimming in and out of the upper surface of the hypolimnion.

As discussed in Section II.D.3.c. a sample of water drawn from the hypolimnion had absorption and fluorescence spectra similiar to laboratory grade humic acid and tannic acid, suggesting that decaying vegetation in the lake or in wetlands which drain to the lake was the source of this strong fluorescence. The alternative source would be wastewater from shoreline residences. Indeed, the spectra of septic tank samples are also very similar to those of the hypolimnion sample.

Figure II.D.20 Locations of Bottom Scan Transects

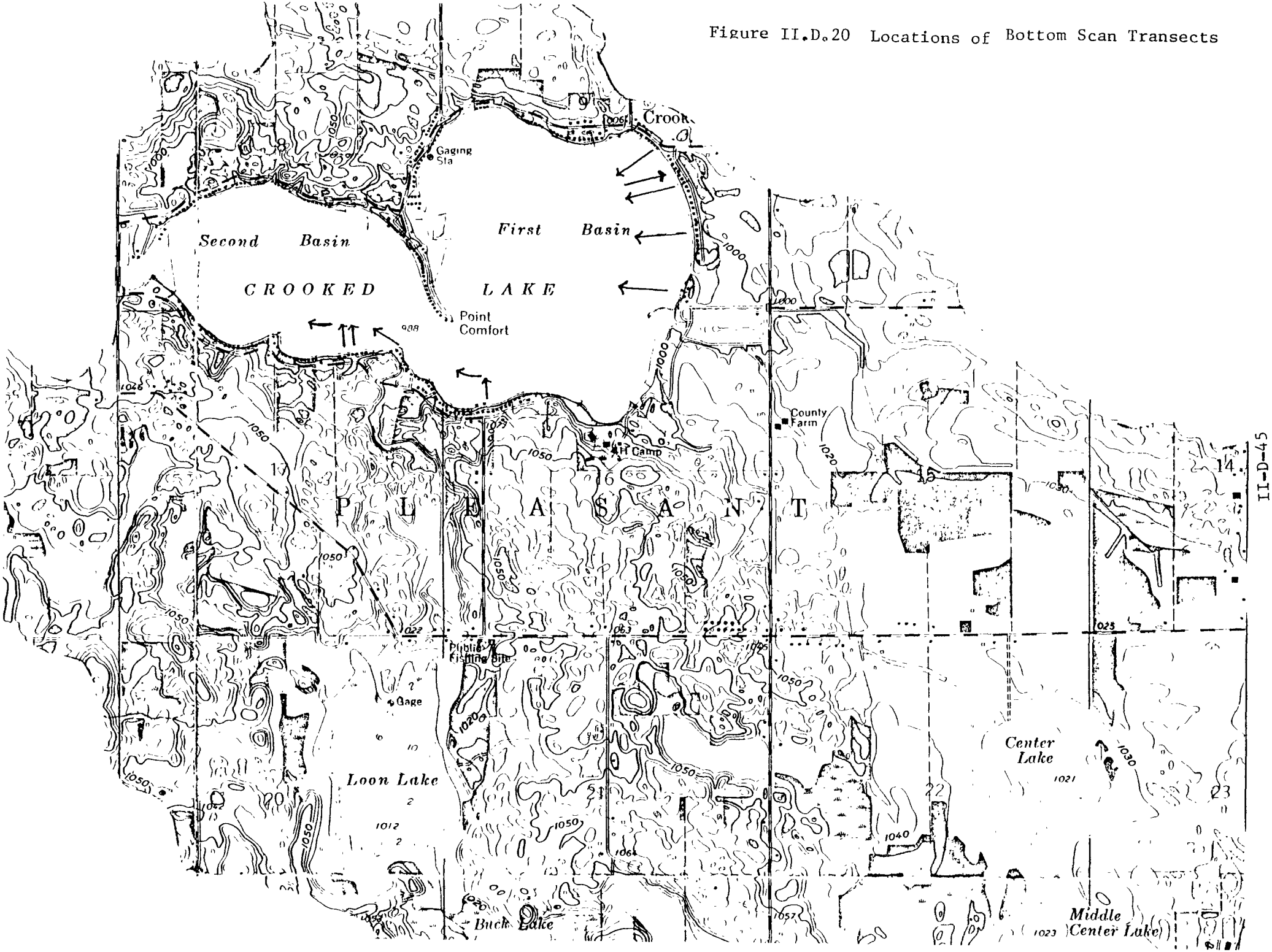
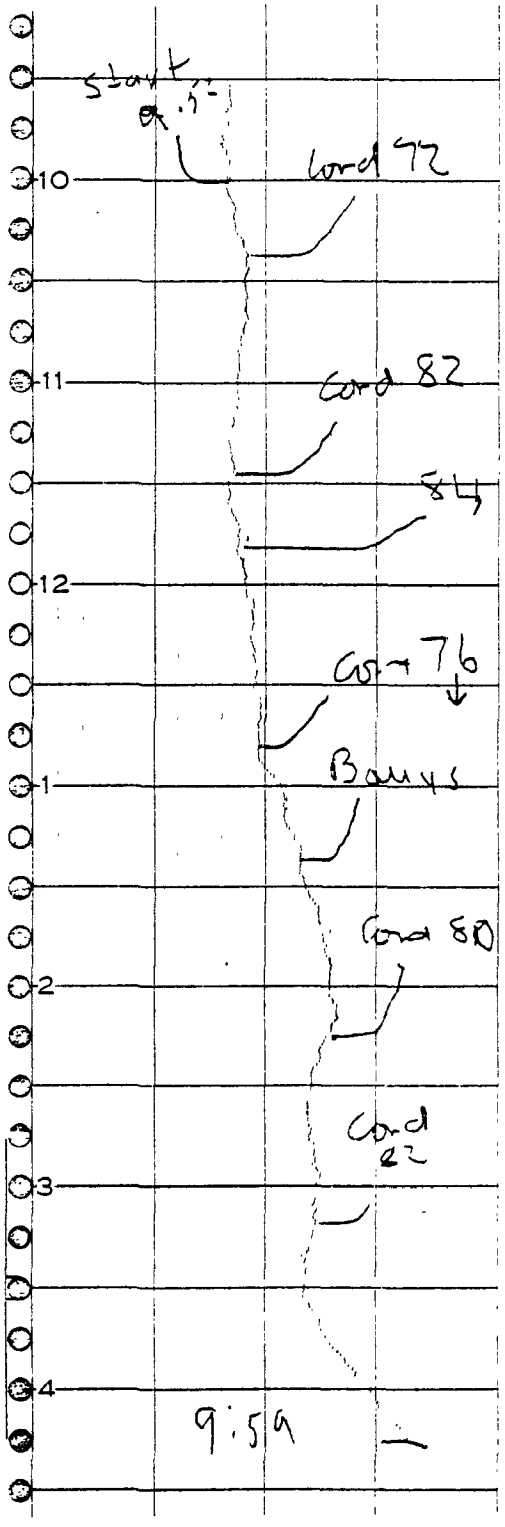
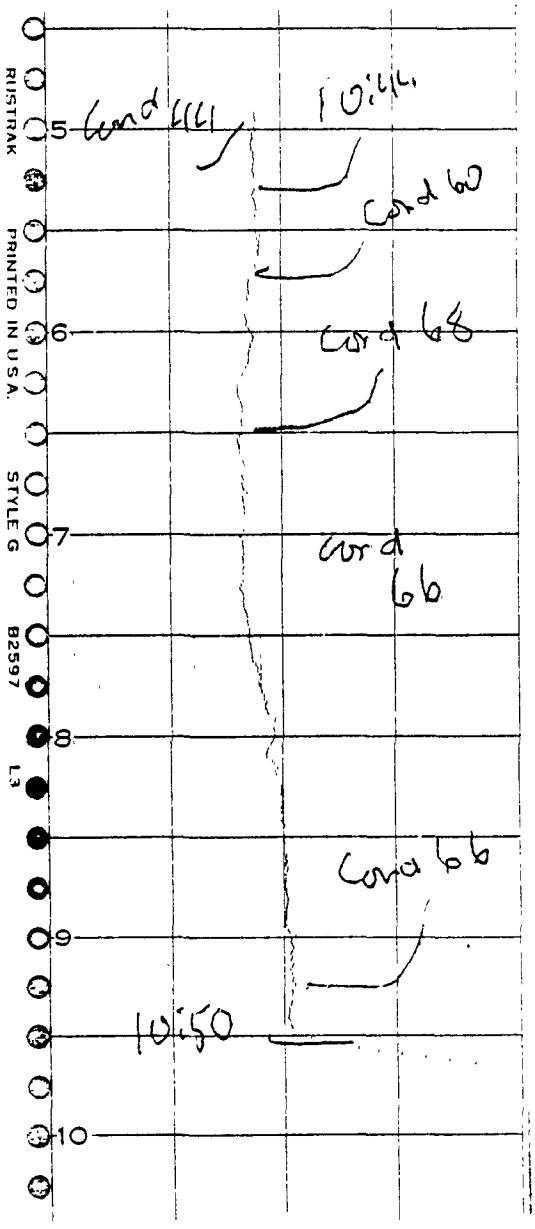


Figure II.D.21 SLD Organic Channel Recordings on Plume Emergence Point Transects

Middle Transect, East Shore, First Basin



South Shore Transect, First Basin



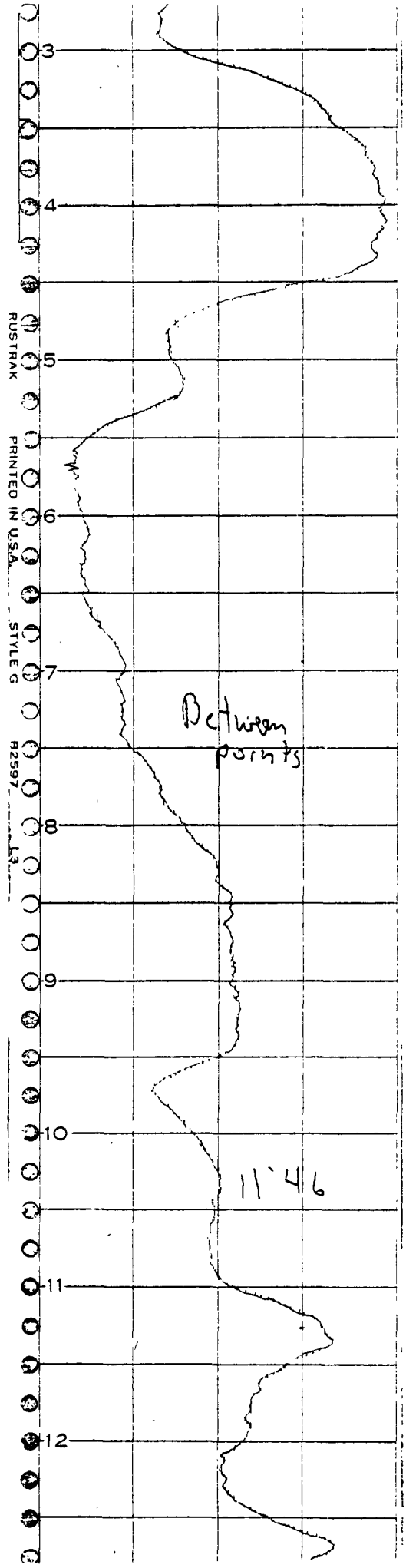


Figure II.D.22

Fluorescence Response of SLD to Crooked Lake Hypolimnion

The diving transects, instead of demonstrating alternative routes for effluent migration and discovering why so few plumes are detectable around Crooked Lake, revealed a massive reservior of strongly fluorescent water, the source of which may have nothing to do with wastewater or shoreline development.

c. Stream Plumes

The SLD surveys conducted in support of the seven Rural Lake EIS's, including the two surveys on Crooked Lake, detected increased conductivity and fluorescence in streams where they flowed into the lake. Shoreline scans during the 1980 field investigations also detected plumes caused by Carpenter Ditch which flows into the southeast corner of the First Basin and the stream flowing from Loon Lake to enter the First Basin on the south shore just east of the point dividing the basins. Both streams drain smaller lakes that are surrounded by wetlands.

The SLD responses to these streams were of greater magnitude than to suspected effluent plumes. Figure II.D.23 reproduces several recordings obtained where the Loon Lake stream enters the First Basin. Note that even as large a source as this can be missed or does not flow some days. These recordings were made at the same channel adjustment setting as the shoreline scans illustrated and discussed in Section II.D.6.a. above.

To quantify the conductivity and fluorescence of these streams before they enter Crooked Lake, the meter was transported by van to six stream locations shown in Figure II.D.24. Since the readings go off-scale of the meter when channel adjustments are set for lake scans, the calibrated channel adjustments themselves were used to determine strength of response.

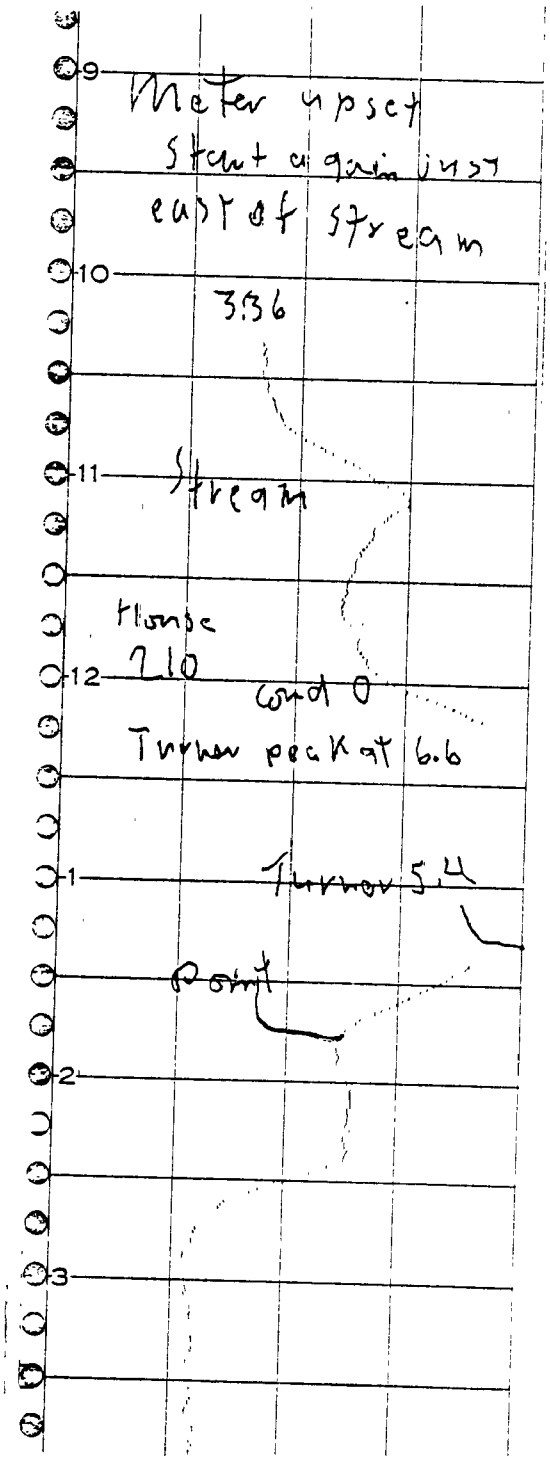
The readings are presented in Table II-D-11. All of the organic signals were too high to be brought back on scale without changing the fluorometer's polarizing filter. Inorganic channel readings were higher than lake settings in Carpenter Ditch but lower in the Loon Lake stream. The flow in these streams and their high fluorescence could probably explain all of the fluorescence in Crooked Lake's hypolimnion.

Stream plumes can produce recordings that would deceive even experienced surveyors. An example was recorded during a survey of Lake George, NY. The recording is reproduced in Figure II.D.25. It was not until the tape for this scan was examined days after the scan that the several peaks, which by coincidence were recorded near houses, were recognized to be eddys moving along the shoreline in the direction of the wind from the mouth of a small plume. Examples such as this highlight the need for careful attention to potential sources of fluorescent materials and a good understanding of physical and biological limnology.

d. Feasibility of Detecting Effluents with Fluorometry Alone

The SLD detects two parameters: fluorescence and conductivity. In theory, conductivity distinguishes between wastewater (which has more ionized, dissolved solids, and, therefore, higher conductivity that the water supply) and natural sources of fluorescence which do not release as much ionized

Figure II.D.23 Shoreline Scan on South Shore Crooked Lake Where Stream from Loon Lake Enters (Organic Channel Recording)



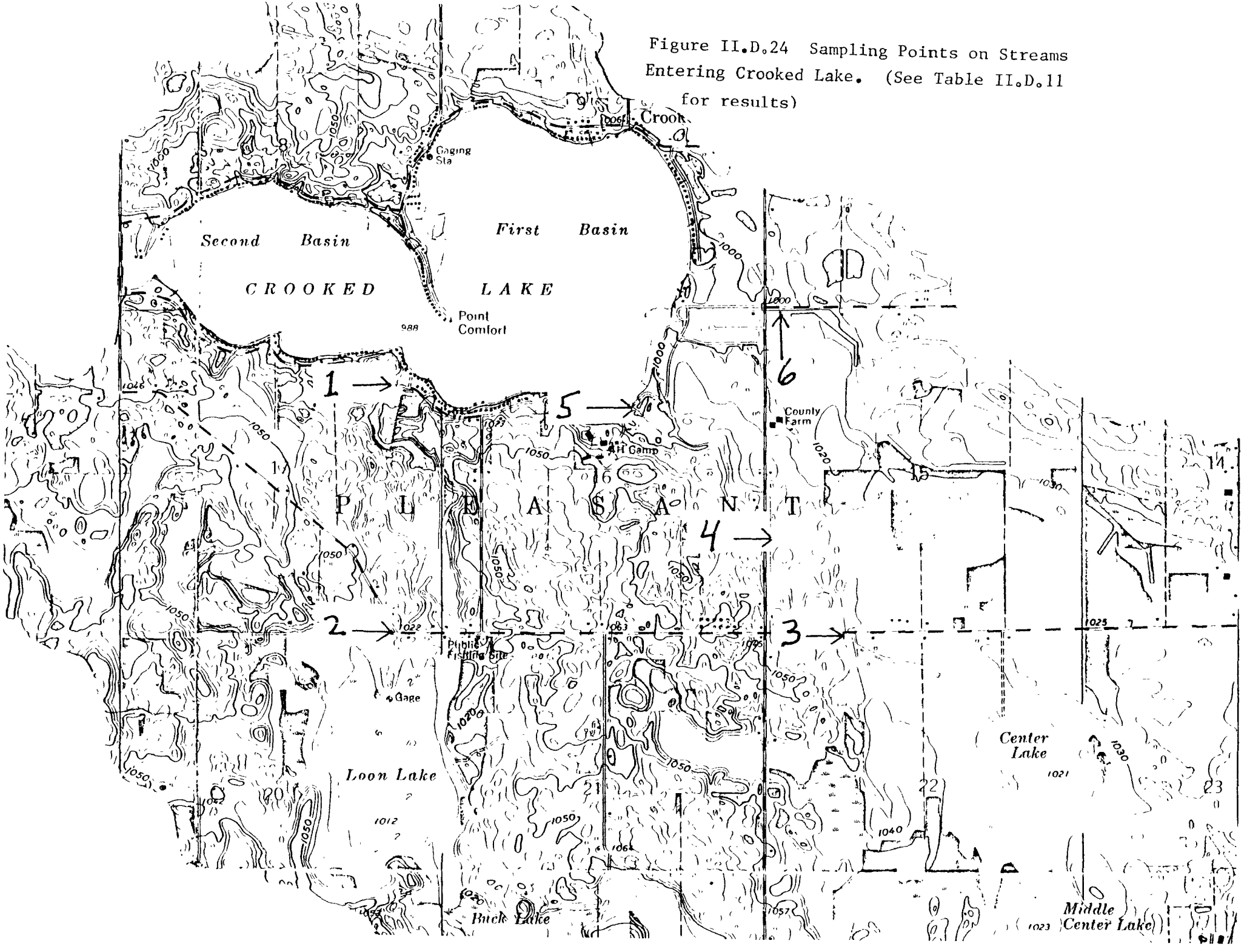


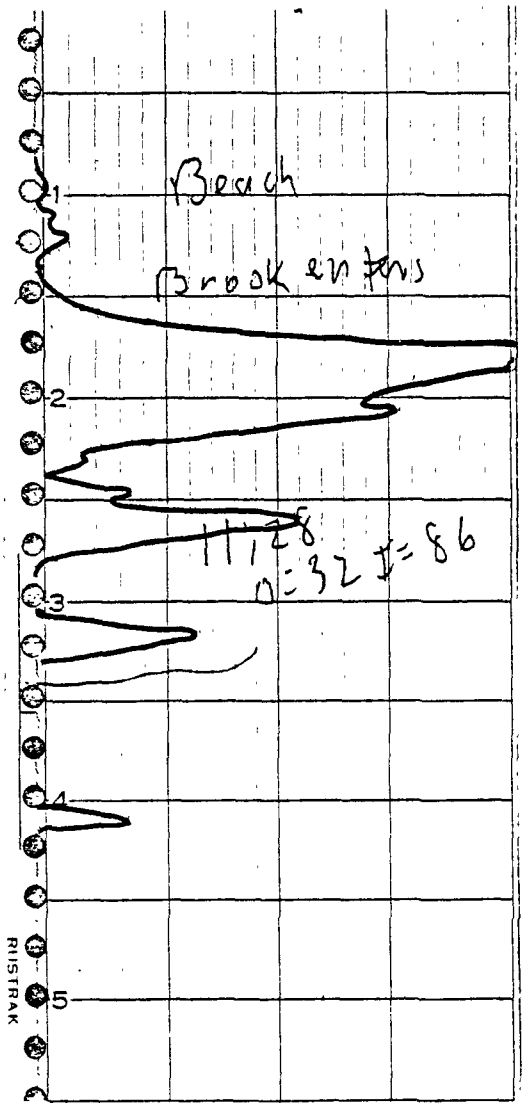
Figure II.D.24 Sampling Points on Streams Entering Crooked Lake. (See Table II.D.11 for results)

TABLE II-D-11. SLD READINGS FOR INFLOWING STREAMS

No.	Stream and Location	Organic Zero	Inorganic Zero
1	Stream from Loon Lake just before entering Crooked Lake	off-scale[a]	506
2	Stream from Loon Lake at first road before lake	718	501
3	Carpenter Ditch at road below Center Lake	off-scale[a]	805
4	Carpenter Ditch mid-way to Crooked Lake	off-scale[a]	802
5	Carpenter Ditch just before entering Crooked Lake	off-scale[a]	814
6	Ditch feeding canals just east of Highway 200N	off-scale[a]	678

a Reducing span adjustment would not bring reading on-scale.

Figure II.D.25 Stream Plume and Eddy Currents, Lake George, NY, August 1981



material. The SLD's "combined" signal responds to rapid (although not necessarily large-scale) changes in these parameters and automatically screens out slowly changing background variations in either parameters. These features provide the benefit of automatic data interpretation which allows the operator to devote attention to other important survey information that will later be essential to interpreting the recordings. The cost of this automatic data interpretation is that potentially significant information on background levels of fluorescence and conductivity are not recorded unless separate notes based on visual meter readings are taken. Also, the benefit of automatic data interpretation is somewhat diminished because the quantitative relationship between rates of change in conductivity, fluorescence and the resulting combined signal are not published.

Because of these concerns and the high cost of currently available SLD's (the ENDECO 2100), interested investigated have asked whether a fluorometer alone or perhaps a fluorometer operated simultaneously but separately with a conductivity probe may provide as much information at a much lower cost.

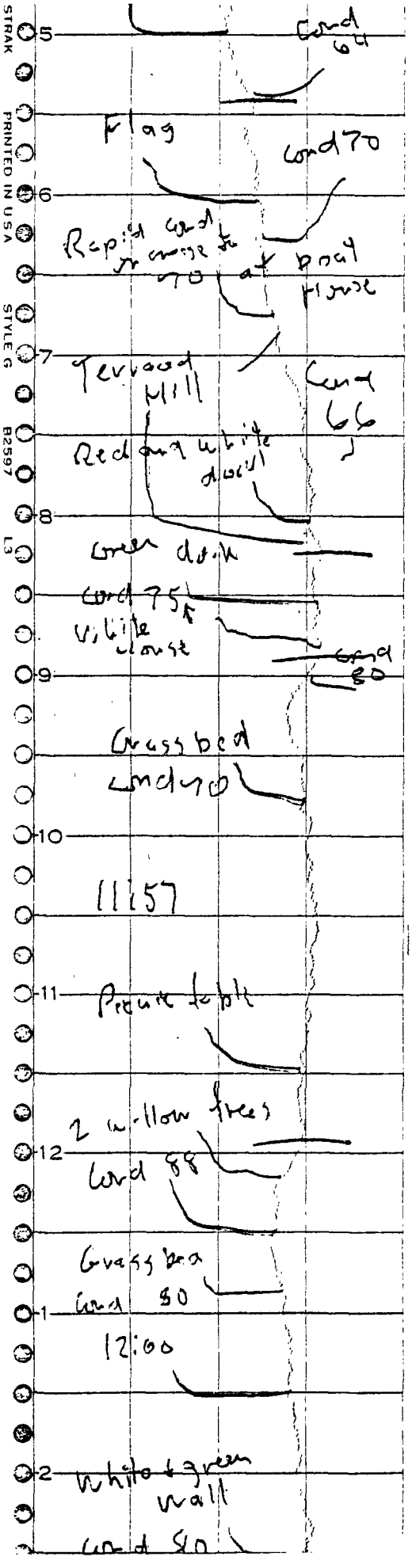
To address this concern during the Crooked Lake field investigations, two approaches were taken: 1) repeat scanning of shoreline segments recording fluorescence only one time, then combined signal another and 2) operation of the SLD in series with a laboratory model fluorometer (Turner Model 10) equipped with the same type light source, filters, and photomultiplier tube.

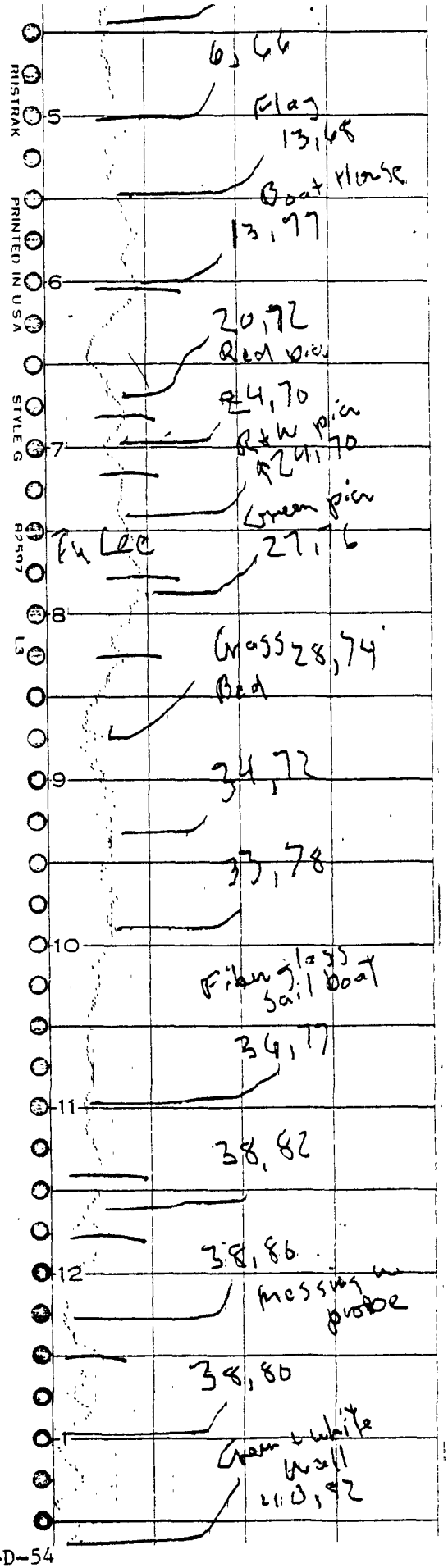
Comparison of fluorescence and combined recordings on the same stretch of shore was illustrated in Figure II.D.19. The same type of comparison is made in Figure II.D.26 for another stretch.

In general, the combined recordings indicate effluent plumes in some locations where the fluorescence recording along was interpreted as reflecting small background or equipment-related fluctuations. The combined signal will, therefore, indicate that more plumes are present than will fluorescence alone. Whether any of the plumes "lost" by recording fluorescence alone are significant in terms of bacteria or nutrients was not evaluated.

The SLD and the laboratory fluorometer were connected with tubing such that water from the probe flowed through the laboratory fluorometer, then through the the SLD. The laboratory fluorometer responded in tandem with the SLD fluorometer as would be expected. However, the meter changes on the laboratory fluorometer were much smaller than those of the SLD. Only the streams and the strongest effluent plumes produced recognizable peaks by inspection of the laboratory fluorometer's meter. It was subsequently determined that changes could have been made to increase the sensitivity of this instrument. However, a direct comparison was not accomplished in the field because the necessary modifications were not made.

Assuming that a fluorometer can be modified to respond like the SLD's fluorometer and a fast-response conductivity probe could be connected to a two-channel recorder with the fluorometer, researchers could obtain much of the data provided by the SLD. The main differences between such an equipment combination and the available SLD is automatic data interpretation and detectability of weak or dispersed plumes.


Cond 64
Flag
Cond 70
Rapid cond increase to 70 at boat house
Tervoort Mill
Cond 66
Red and white diamond
Green dot
Cond 75
White cloud
Cond 80
Grass bed
Cond 80
11:57
Picnic table
2 willow trees
Cond 88
Grass bed
Cond 80
12:00
White & green wall
Cond 80


6, 66
Flag
13,68
Boat House
13,77
20,72
Red pic.
24,70
R&W pic
21,70
Green pic
27,76
Fy Lee
Grass 28,74
Bed
34,72
33,78
Fiberglass Sail Boat
34,77
38,82
38,86
Missing w/ number
38,80
Green & white wall
40,92

The automatic data interpretation function of the SLD is patented and the design details are proprietory. Particularly when limited time is available or when requested scans are not feasible, this feature can be very valuable. However, a person with extensive experience with a fluorometer alone or a fluorometer/conductivity meter/recorder combination may not be at a serious disadvantage regarding time and scheduling.

Regarding the detectability of weak or dispersed plumes, insufficient work has been done to discount their importance. However, detailed sampling of a number of easily detectable plumes shown that only a fraction also have nutrients or bacteria at levels to be of concern. Weaker plumes would be expected to have even fewer pollution problems. Therefore, if the septic leachate survey's purpose is to find and sample the worst effluent discharges, the enhanced plume detection may not be necessary. On the other hand, if the purpose is to find all possible wastewater sources or if surveys might be done under suboptimal conditions of wind and waves, the apparent ability of the SLD to discern weak plumes would be valuable.

Acknowledgement: Spectrophotofluorometry and thin layer chromatography were performed by Dr. Daniel W. Armstrong of Georgetown University, Washington, DC. His advice and expertise are greatly appreciated.

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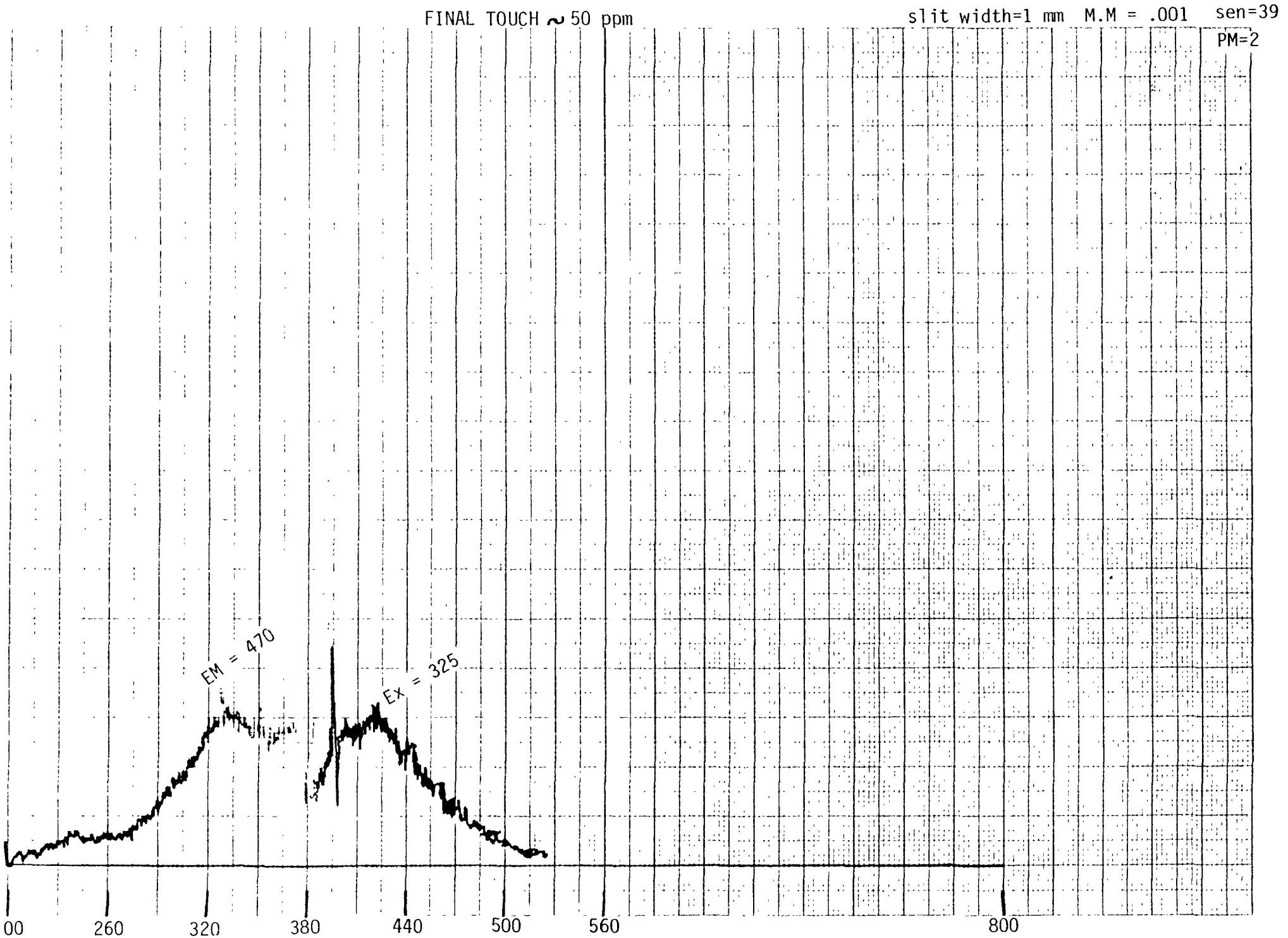
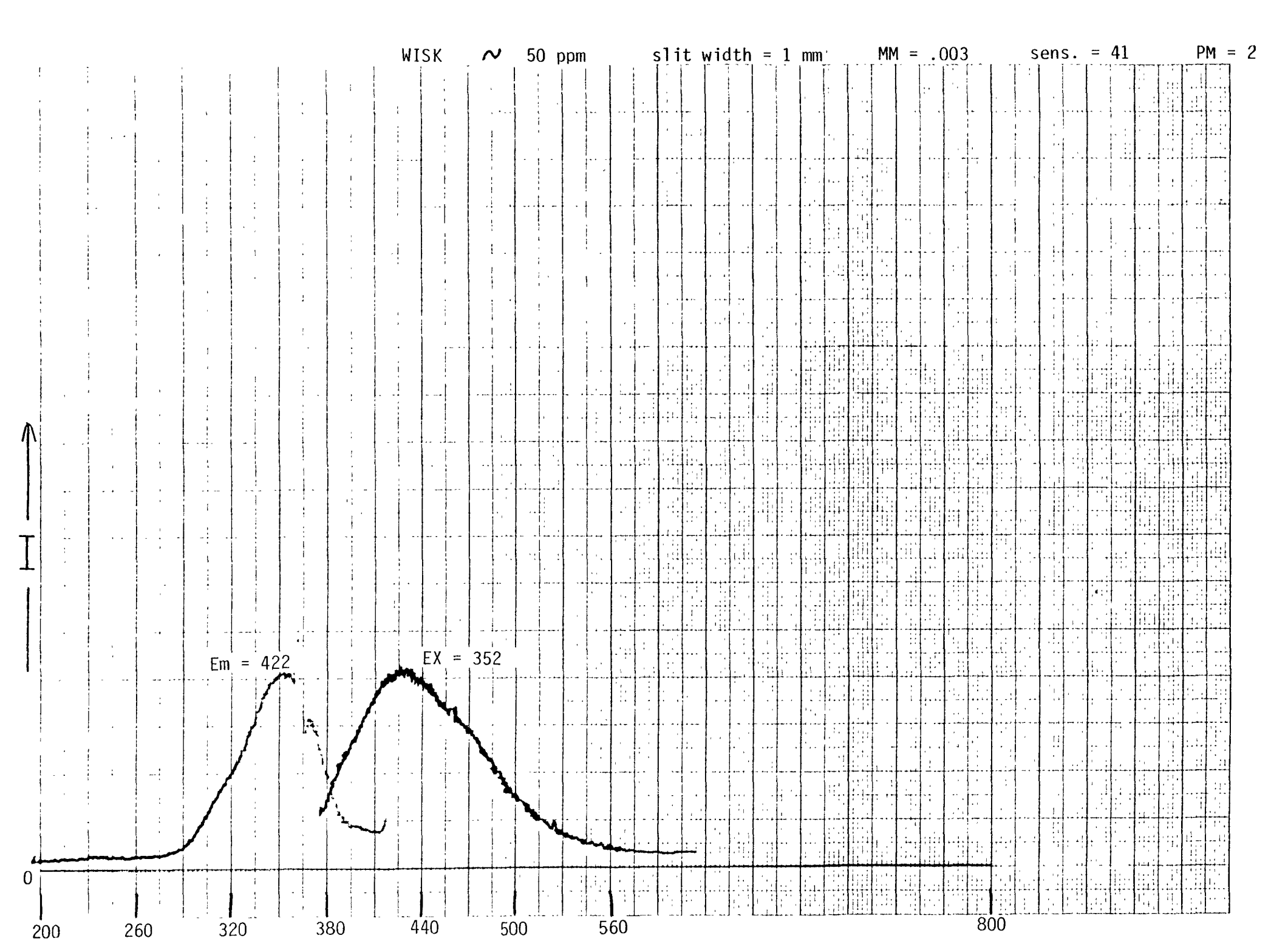
K-V Associates, Inc. 1979c. Investigation of Septic Leachate Discharges into Steuben Lakes, Indiana, December, 1978. Published in: Draft Environmental Impact Statement Appendices, Alternative Waste Treatment Systems for Rural Lake Projects, Case Study Number 4: Steuben Lakes Regional Waste District, Steuben County, Indiana. U.S. Environmental Protection Agency, Region V, Chicago IL.

K-V Associates, Inc. 1977d. Septic Leachate and Groundwater Flow Survey, Steuben Lakes, indiana, 1979. Published in Final Environmental Impact Statement, Alternative Waste Treatment Systems for Rural Lake Projects, Case Study Number 4: Steuben Lakes Regional Waste District, Steuben County, Indiana. U.S. Environmental Protection Agency, Region V, Chicago IL.

APPENDIX II-D


FINAL TOUCH ~ 50 ppm
slit width=1 mm M.M = .001 sen=39
PM=2
EM = 470
Ex = 325
00
260
320
380
440
500
560
800


WISK ~ 50 ppm
slit width = 1 mm
MM = .003
sens. = 41
PM = 2
I
Em = 422
EX = 352
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200
260
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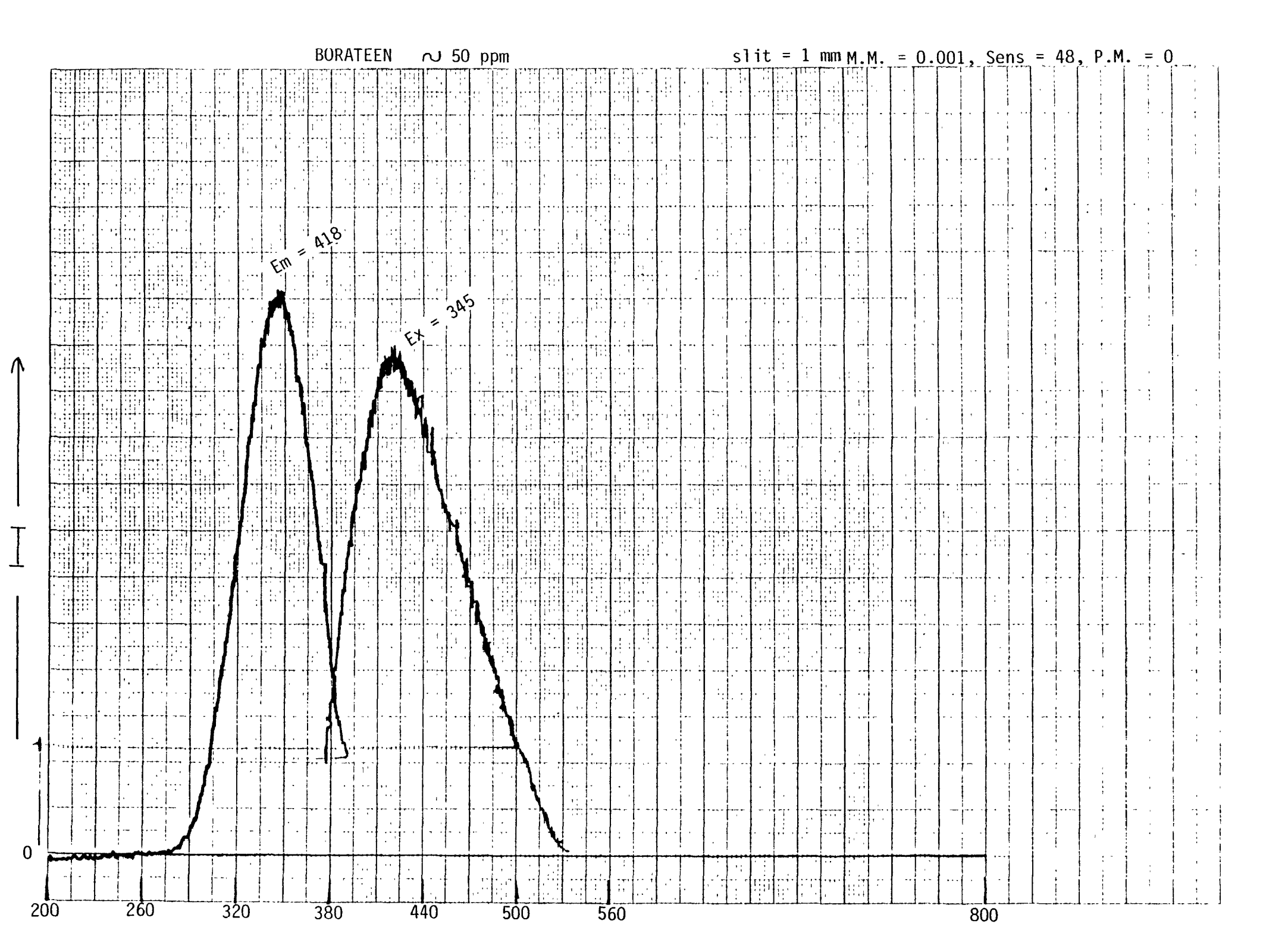
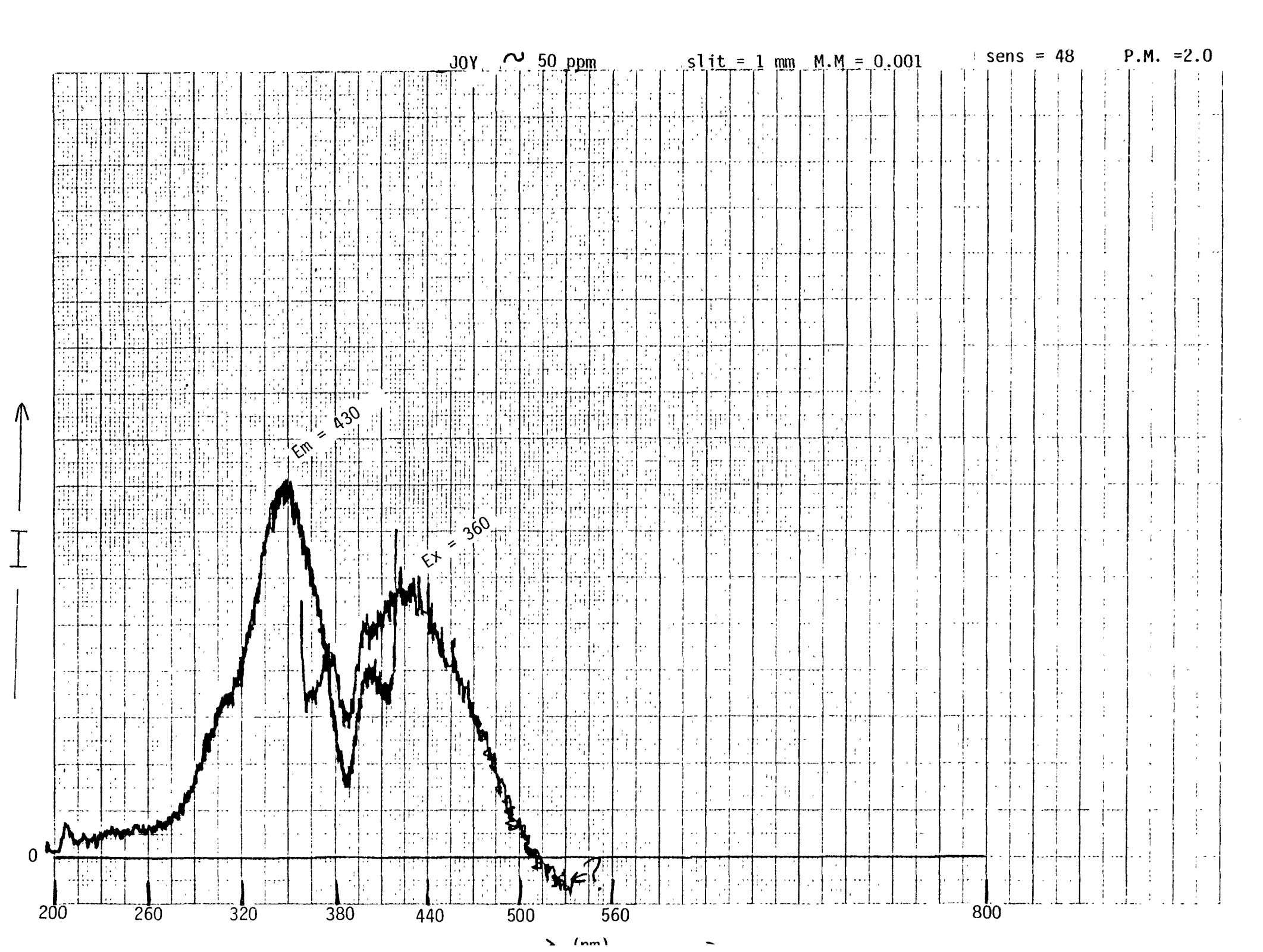
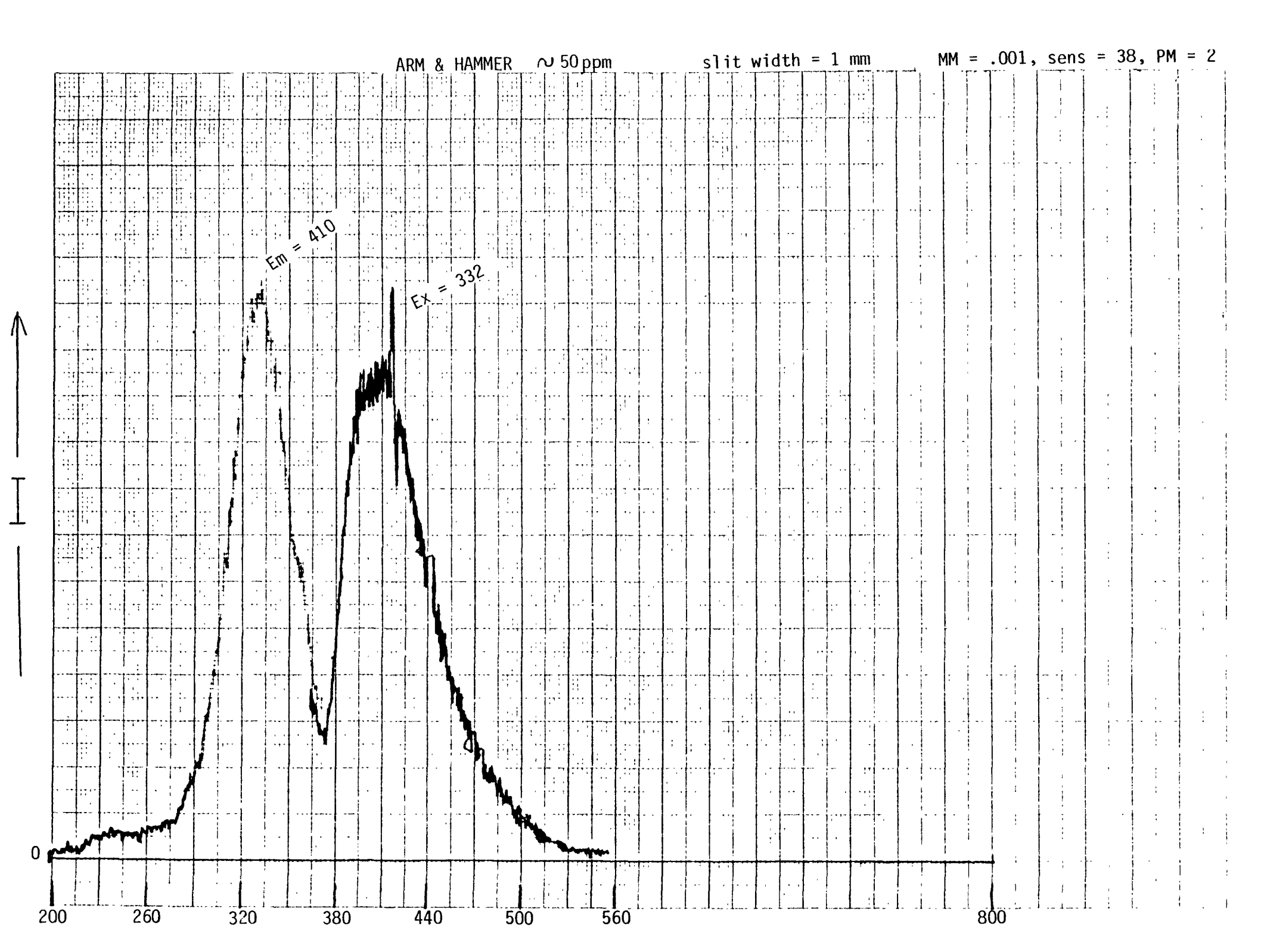
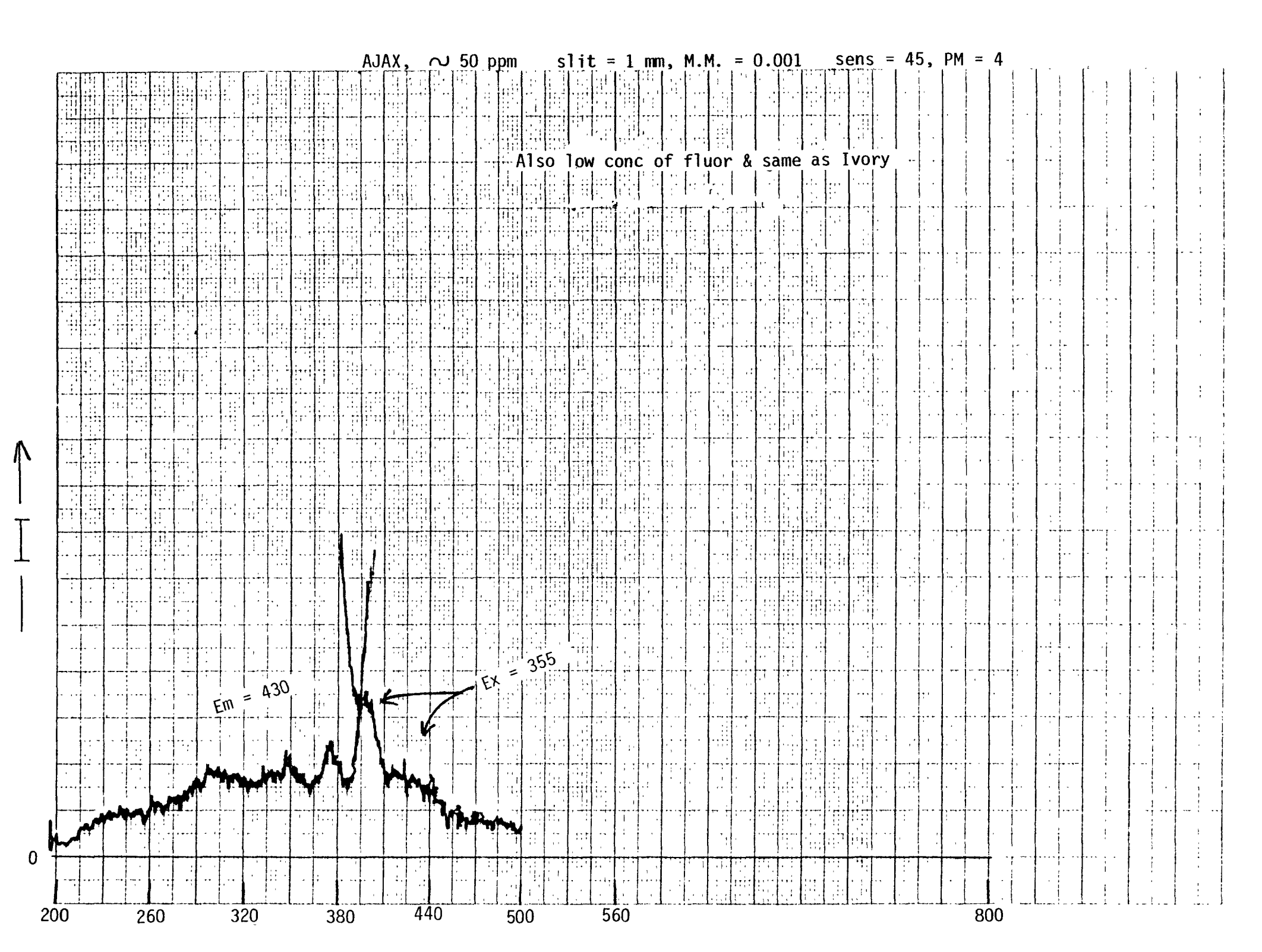
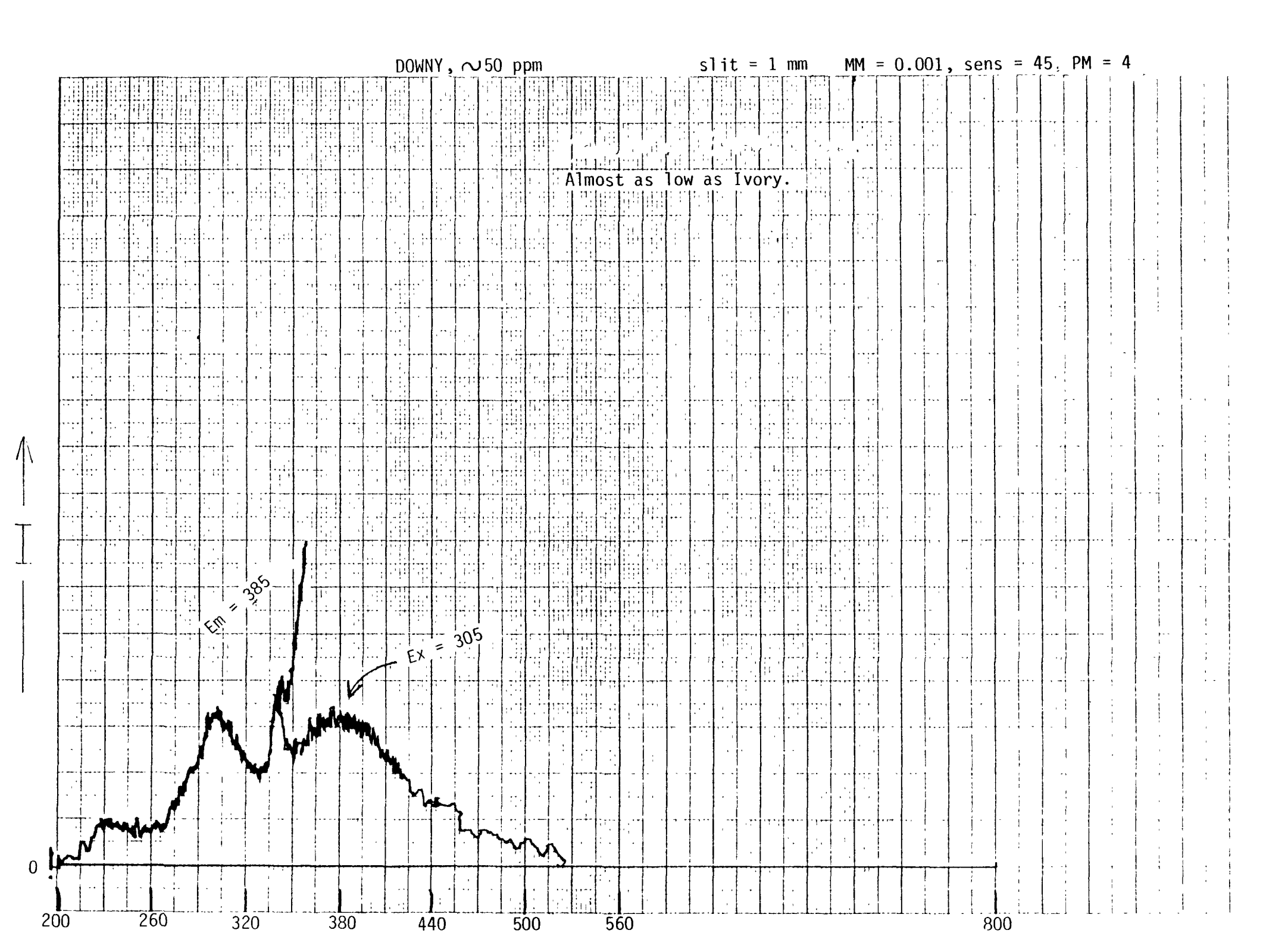
380
440
500
560
800

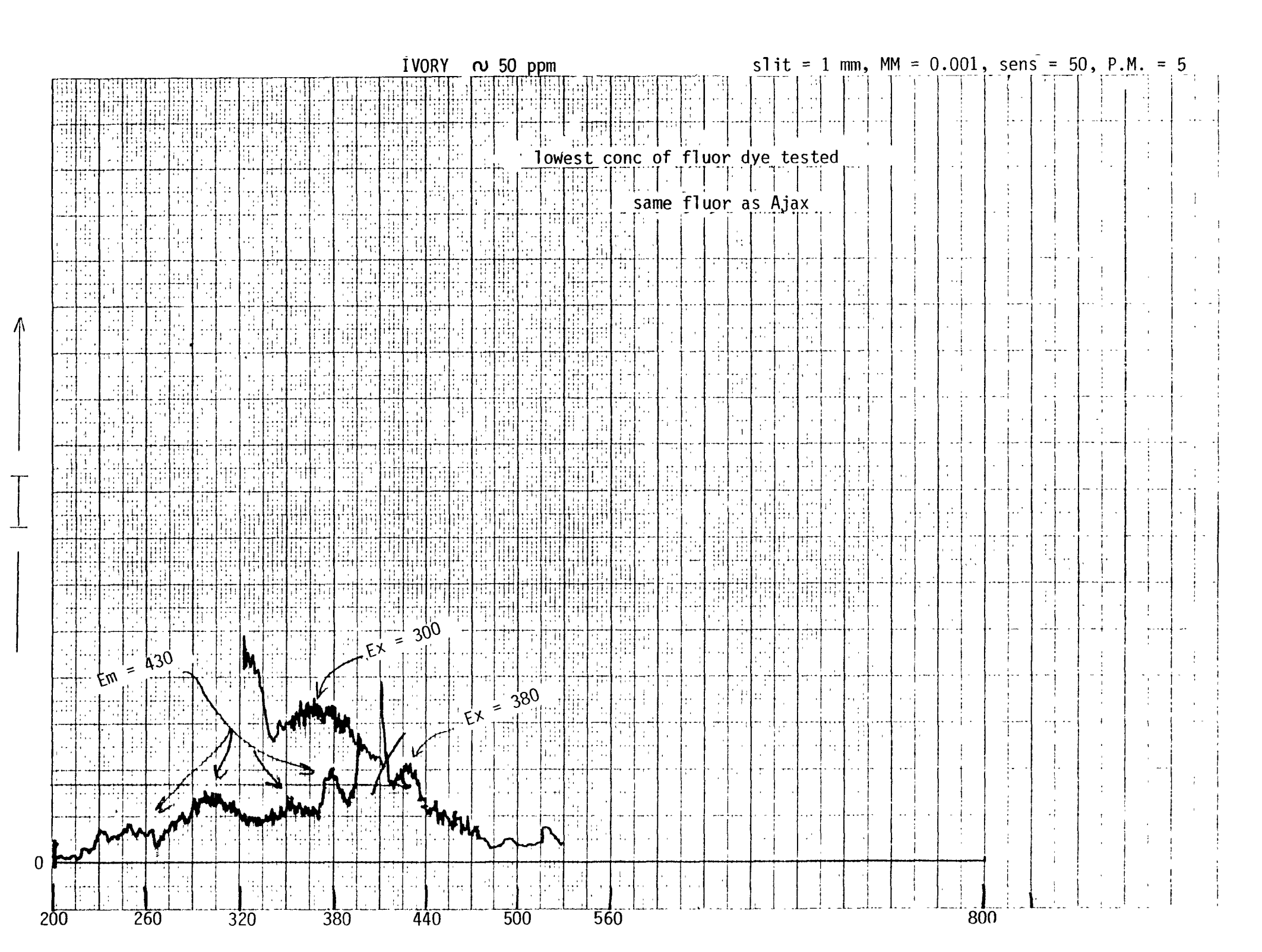

BORATEEN ~ 50 ppm
slit = 1 mm M.M. = 0.001, Sens = 48, P.M. = 0
Em = 418
Ex = 345
I
1
0
200
260
320
380
440
500
560
800


JOY ~ 50 ppm
slit = 1 mm M.M = 0.001
sens = 48
P.M. =2.0
Em = 430
Ex = 360
I
0
200
260
320
380
440
500
560
800
λ (nm)


ARM & HAMMER ~ 50 ppm
slit width = 1 mm
MM = .001, sens = 38, PM = 2
Em = 410
Ex = 332
0
200
260
320
380
440
500
560
800


AJAX, ~ 50 ppm slit = 1 mm, M.M. = 0.001 sens = 45, PM = 4
Also low conc of fluor & same as Ivory
Em = 430
Ex = 355
0
200
260
320
380
440
500
560
800


DOWNY, ~50 ppm
slit = 1 mm
MM = 0.001, sens = 45, PM = 4
Almost as low as Ivory.
I
Em = 385
Ex = 305
0
200
260
320
380
440
500
560
800


IVORY ∿ 50 ppm
slit = 1 mm, MM = 0.001, sens = 50, P.M. = 5
lowest conc of fluor dye tested
same fluor as Ajax
Em = 430
Ex = 300
Ex = 380
0
200
260
320
380
440
500
560
800

E. SEPTIC LEACHATE DETECTOR POLICY

Research and field studies with septic leachate detectors sponsored by U.S. EPA, Region V to date have been based on an ENDECO Model 2100 Septic Leachate Detector and its prototypes. The recommendations made here regarding its use and its eligibility for use or purchase with federal grants are not intended to apply to other devices for locating wastewater in natural water bodies. Neither support for nor restrictions on other specific makes of equipment should be inferred from these recommendations, since the authors have not tested devices other than the one identified. In general, however, Region V encourages development of any device or method that facilitates location and remediation of insufficiently treated wastewaters.

1. INTERPRETATION OF DATA

As discussed in the preceding Technical Reference Document, the septic leachate detector is subject to certain limitations that must be recognized during use and in interpretation of the data it generates. The most significant limitation is that it cannot quantify the strength of wastewater in a sample or body of water. The organic and inorganic substances that it monitors can be transported through soil and water quite independently of other wastewater constituents. Even fluorescence and conductivity are recorded in relative, not quantitative, units. In order to quantify the concentrations of nutrients, bacteria, or other wastewater constituents, flow through the meter can be sub-sampled, or samples can be collected by conventional means for later analysis. The advantage of the detector is that it permits collection of samples in demonstrated effluent plumes.

Aside from the limit on quantification, septic leachate detector surveys are subject to false positives and false negatives. Most of these potential errors are due to the dynamic nature of the natural systems involved, and to variability in wastewater characteristics. False positives can be caused by:

- Naturally fluorescent decay products from dead vegetation. Swamps, marshes, and peat deposits can leach tannins, lignins, and other compounds that fluoresce in the detection range of the fluorometer. The conductivity measurements provided by the detector are intended to differentiate such signals, but in practice dilution may eliminate detectable conductivity changes expected from wastewaters, thus making a wastewater plume appear similar to natural decay products.

- Sediment or air drawn through the detector can cause dramatic changes in the monitor readings. This is usually noted by the operator and recorded on the recorder tape.

- Eddy currents carrying large wastewater or bog plumes can appear to be individual plumes from on-site systems.

The more serious errors are false negatives, since these may indicate that there is no problem when in fact a problem exists. Notable errors caused by false negatives included:

o High dilution of wastewater in a lake or in groundwater, as mentioned above, may reduce conductivity differences to the level of normal

background variations. The absence of conductivity differences will cause the detector to electronically mask fluorescence signals that are detected.

- Mixing of lake water by wind and waves can disperse leachate very rapidly, causing normally strong effluent plumes to be missed altogether. The time it takes for leachate to accumulate along a shoreline to detectable concentrations is dependent on several factors that have not yet been studied.

- Fluctuations in lake level can slow or even reverse normal groundwater flow, temporarily eliminating leachate emergence at a shoreline.

- Groundwater recharge by rainfall, snowmelt, or irrigation will also affect the dynamics of leachate movement.

- Seasonal use of dwellings may result in only periodic emergence of leachate at a shoreline.

Due to these factors, the data generated by septic leachate detectors has to be carefully interpreted before it can be considered to be useful information. Interpretation is aided by the following supplementary data collected or recorded before, during, and after the shoreline scan as noted:

Before

- Surface and subsurface watersheds, as suggested by topographic maps and available groundwater hydrology reports

- Groundwater flow, as determined by meters or other methods

- Soil types

- Wetlands and other sources of organic decay products in the watershed

- Locations of surface malfunctions identified by aerial photography interpretation

- Design, usage, and performance of on-site systems identified by on-site sanitary inspections

- Changes in lake elevation up to the time of the survey.

During

- Weather conditions, especially wind speed, wind direction, and recent rainfall

- Lake stratification and surface currents

- Observed non-wastewater sources of fluorescence or conductivity, such as culverts, drainage ditches, salt storage areas, landfills, abundant organic material in near-shore sediments

- Observed direct discharges
- Likely proximity of on-site systems near shoreline
- Operational mishaps such as stirred-up sediments, air drawn through meter, and engine backwash
- Sensitivity and zero adjustments for fluorescence and conductivity channels
- Frequent notation of visual meter readings
- location, time, and conditions of water sample collection.

After

- Water sample analysis results.

All of these types of supplementary data need not be collected for every shoreline scan. The person responsible for a survey must have the ability to select appropriate data collection methods so that the equipment responses can be meaningfully interpreted.

2. ELIGIBILITY OF SURVEYS AND EQUIPMENT FOR CONSTRUCTION GRANTS FUNDING

Because of the possibilities for error and the many factors influencing the results of septic leachate detection, the validity of surveys rests heavily on the experience, knowledge, and judgment of the surveyor. Until additional evaluation is made of the factors influencing survey results, septic leachate surveys will be eligible for Construction Grants funding only when:

1) the person in charge is experienced in operation and maintenance of the detector model being used. At least two weeks of field experience is necessary for assisting someone who is already expert with the model;

2) the person in charge is present during any shoreline scans that are reported;

3) data is interpreted by a person who has a professional background in liminology; and

4) approximate wind speed and direction are noted during the survey, and reported.

Septic leachate detectors should prove to be valuable monitoring tools for communities managing shoreline on-site systems. Purchase of detectors will be eligible for Constructon Grants funding. Grantees will be required to show that comparable instruments are not available on a timely basis from other nearby grantees. Funded instruments will be made available to other grantees.

F. AERIAL PHOTOGRAPHY METHODS AND POLICY

Properly acquired and interpreted aerial photography can provide data on surface malfunctions of on-site systems. With this technique, a community survey can be made rapidly and at relatively low cost without intruding on private property. Aerial photography detection of surface malfunctions is a three-step process involving acquisition of the photography, identification of suspected malfunctions by an experienced photointerpreter, and field checking of the suspected malfunctions.

Optimum coverage, resolution, and signature recognition can be achieved using fine-grained color infrared film flown to a scale of approximately 1:8000 (1 inch = 1667 feet). Other image types can be acquired in conjunction with the color infrared film, such as true color, thermal infrared, or thermal scans. However, experienced photointerpreters (Evans, 1981) feel that color infrared film will be adequate.

Timing of the flight is an important consideration in remote sensing of surface malfunctions. These failures can best be detected when groundwater elevations are highest and foliage is minimal. Therefore, best results for permanent residences are obtained during spring or late winter when the ground is not snow-covered. Tree cover present during the remainder of the year can limit detection of surface malfunctions. In cases where aerial photographs must be taken during summer months, such as in communities with seasonal populations, the subsequent interpretation and field checking phases must be conducted more cautiously. Also, flights can be completed with substantial overlap of photos, thus affording stereoscopic analysis of on-lot features. With sufficient overlap, interpreters can actually see under some of the taller trees.

Suspected malfunctions should be identified from the photography by an experienced photointerpreter. This experience is needed to distinguish valid signatures from those of unrelated phenomena such as shade, natural vegetation and wet soils, and artificial surface drainage features. Surface manifestations of surface malfunctions include:

- Conspicuously lush vegetation,
- Dead vegetation (especially grass),
- Standing wastewater or seepage, and
- Dark soil, indicating excessive accumulation of organic matter.

The suspected malfunctions should be field checked. The ideal person to do this is the photointerpreter, although others may perform the task. By inspection and, if feasible, by interview with the residents, the suspected malfunctions are reclassified as:

- Confirmed malfunctions - standing wastewater from an on-site system is visible on the land surface,

- Marginal malfunctions - accumulation of excess organic matter or the presence of dead vegetation, indicating that wastewater had surfaced in the past, or

- Irrelevant signatures - visible surface or vegetative features which mimic the visual characteristics of malfunctions but are not caused by wastewater.

Aerial photography acquired for this purpose can be used for other purposes during facilities planning, such as:

- House counts,
- Land use, vegetation and wetlands analysis, and
- Layout of wastewater collection and transmission facilities.

To accomplish the last purpose listed above, precision flights may be necessary to overcome resolution problems that can result from the normal tilting of the airplane during photo missions. No special preflight measures (establishment of reference points, etc.) are required. Available maps can serve as a guide on precision flight missions. Precise photo missions enhance the three-dimensional effect already characteristic of aerial photographs, thereby enabling facilities planners to complete detailed design of wastewater collection and transmission facilities. These data supplement those contained in USGS topographic maps. The cost of precision flights can be expected to be approximately 50% greater than normal photographic missions. The decision to make a precision flight should be based on the likelihood that large portions of the facilities planning area will require centralized collection and treatment. Otherwise the extra cost cannot be justified.

G. SANITARY SURVEY METHODS AND POLICY

1. PURPOSES OF SANITARY SURVEYS

Sanitary surveys are among several data collection and analysis methodologies that can be used in unsewered communities to assess the need for improved wastewater facilities. If continued use of on-site systems is planned, the sanitary survey is a fundamental step in selecting and designing the improvements.

Design of a sanitary survey depends on the expected uses of the information to be collected. For this report, it is assumed that the study is concerned primarily with existing on-site wastewater systems, and secondarily with nearby water supply wells and springs that may be affected by the wastewater systems. Trash, rodents and other matters that could be included in a community sanitary survey are not considered here.

Within this defined scope, sanitary surveys can achieve several specific objectives:

1) Identify possible sources of water quality and public health problems. This is a fundamental requirement for facilities planning in unsewered communities. Other methods, such as aerial photography interpretation, review of health department files, or interviews with septic tank contractors, might meet this purpose or augment sanitary surveys in doing so.

2) Evaluate causes of system malfunctions. Causes for some malfunctions can be determined readily by interviewing the resident or inspecting the site. The interviewer's experience and the homeowner's knowledge of his or her system will affect the proportion of malfunctions that can be explained during the first visit.

3) Assess the feasibility of continued use of on-site systems. Cost studies have shown that where continued use is feasible, it is nearly always cost-effective compared to installing sewers. The performance of existing systems is the best indicator of this feasibility.

4) Provide a quantitative information on the types and frequencies of malfunctioning systems, as a basis for projecting renovation and replacement needs and for estimating the costs of community-wide optimum operation alternatives. Decisions made to sewer or not to sewer an area depend on meeting this objective at least approximately, and even a partial survey can furnish sufficient data for projections.

5) Collect information on individual properties and their on-site systems, for later use in the process of designing renovation and replacement facilities if continued use of these systems is the selected alternative. The first step in the design process, which is normally to collect such information, will thus already have been completed for surveyed properties.

Because the design of the survey will determine in large part how successfully these objectives are met, the persons responsible for the survey should initially assess the relative importance of each of these categories of information in their local planning situation.

2. TERMINOLOGY

Before discussing the design of sanitary surveys, conventions on the use of terminology will be stated.

In its common usage, the term "sanitary survey" can apply to any evaluation of hygienic conditions on a specific property or within an entire community. However, "sanitary survey" as used here is defined more specifically, and refers to resident interviews, visual site inspections, and (as appropriate) water supply inspections for all or a portion of the dwellings and businesses in a community. A "complete sanitary survey" is one intended to reach all occupied places; and a "partial sanitary survey" is intended either to sample a representative portion of the community, or to survey places that had been previously selected on the basis of past problems, poor site conditions, or other factors. A sanitary survey includes preparation, on-site sanitary inspections, and analysis of the information acquired. An interview, site inspection, and water supply inspection for an individual property is termed an "on-site sanitary inspection" to distinguish it from the broader meaning of sanitary survey.

When sanitary surveys are performed in the context of facilities planning for the EPA Construction Grants program, the terms "needs documentation" and "detailed site analysis" establish a valuable distinction. "Needs documentation" refers to data collection for the purpose of community-wide or segment-wide alternatives development and selection. "Detailed site analysis" refers to the type of data collection that is required to analyze the condition and performance of an individual system so that renovations or replacement facilities can be designed. On-site sanitary inspections can be the pivotal data collection effort in either needs documentation or detailed site analysis. However, when it is clear from available data that continued use of on-site systems will be the selected alternative, performing a sanitary survey during facilities planning is optional. In such cases, the sanitary survey can be deferred so that it becomes an integral part of the detailed site analysis.

In some cases, available data, together with the information collected in a sanitary survey, may not be sufficient to describe the performance of on-site systems. This may occur where the suspected problems or their causes are not obvious either to the resident or to the surveyor. Sampling of soil conditions, groundwater quality, or groundwater hydrology may be needed in such cases to reveal the fact or the causes of poor system performance. Limited sampling intended to explain typical problems is called "representative sampling," and normally is conducted concurrently with a sanitary survey.

3. PREPARATION FOR THE SURVEY

The design of a sanitary survey can vary considerably, depending on its purposes (see Section 1 above), available funds, the expertise of available personnel, and the extent of existing information on the performance of on-site systems in a community.

The first decision to be made will often be whether the survey should be partial or complete. If decisions to sewer significant parts of a community cannot be supported with available data or inexpensive data collection methods, a partial sanitary survey reaching anywhere from 10% to 80% of occupied buildings may provide sufficient information while costing less than a complete survey. Data from a survey will have little future utility if the survey (along with cost, feasibility and environmental analysis) indicates that sewers are necessary. If conditions are that severe, a partial survey should reveal that fact while avoiding unnecessary work and expense. On the other hand, if it is reasonably clear that most or all of a community will continue to use on-site systems, then a complete survey may be more efficient in the long run. The quality of data on system performance available prior to the survey will affect the decision on whether a survey should be partial or complete. If comprehensive and up-to-date data are available, the outcome of needs documentation (to sewer or continue using on-site systems) will be predictable and the chances of over-surveying or under-surveying will be reduced. The most common sources of available data are health department permit records, soil maps, and interviews with local septic tank contractors, well drillers, sanitarians, zoning officials and soil scientists.

If a partial sanitary survey is called for, good available data may also enable the survey designer to emphasize analysis of the causes of failures. If the available data already identifies most of the failures, then the survey can be "targeted," that is, can concentrate on analyzing known problems and making preliminary identification of the required replacements and renovations. However, if the failure rate cannot be estimated from available data, it might be better to design the survey to estimate this statistic and to pinpoint the most frequent failures. A random survey, aimed at inspecting some predetermined fraction of occupied buildings, can accomplish this aim and provide an estimate of overall failure rate.

Two closely related factors in survey design are the personnel and the funds available to do on-site sanitary inspections. If funds are very limited, and the main purpose of the survey is to locate problems, then local volunteers or low-cost, part-time employees might be trained in a day or two to find the obvious failures. Such an approach would be useful, for instance, where available data is poor but does suggest that failure rates are high and that sewers will be necessary. More experienced surveyors or small system experts can be sent in later if they are needed to analyze specific problems.

On the other hand, many planning situations will dictate that experienced personnel be used from the start. If expected problems are not obvious (for example, well contamination) or are difficult to analyze without disrupting the property (as with some plumbing backups), it may not help the facilities planner to have to deal with conjectures from inexperienced surveyors. If the survey designer expects that the project will end up implementing on-site renovation and replacement, then having highly qualified individuals responsible for the entire process from survey design through facilities construction might be less expensive in the long run.

Another variable to be weighed is the length of time available to complete the survey. For instance, a community that has made a long-term commitment to manage on-site systems for its property owners may be in a hurry to locate only the obvious problems at first, problems which can be located well

enough by inexperienced surveyors. If the planning situation is complex, however, or financial assistance depends on the timely completion of application documents, then a team of experts might be needed to complete the survey and deliver their results quickly so that other necessary decisions and actions may proceed.

The validity of an on-site sanitary inspection rests in large part on the willingness and ability of the person being surveyed to respond accurately and completely to the surveyor's questions. An experienced surveyor can maximize the information gained by knowing what questions to ask and being able to respond to a variety of questions about the project that the resident might ask.

However, any surveyor's efforts will be more productive if residents have been prepared for the inspection by advance publicity that answers the following questions:

- What are the purposes of the survey?
- What are the types of information that will be sought?
- How will this information be used?
- How can residents cooperate in the survey?

Information that can accomplish this advance preparation may be disseminated in a variety of ways. Local newspapers, radio, and television are effective means of publicizing a survey. Notices sent to civic associations may also be valuable. Another method of publicizing a survey, which can also be used to gather information of value in planning the survey, is that of direct mailing of questionnaires and fact sheets to residents. An example of a mailed questionnaire previously used by EPA Region V will be found in Appendix A. The cost of duplicating, mailing, and analyzing mailed questionnaires may be justified by the enhanced ability to target inspections in areas with reported problems. The response rate may also be taken as an index to public support for a project.

Prior to the survey, suitable forms for guiding the on-site sanitary inspections and for recording information should be developed and reproduced. A useful form that EPA Region V has developed to standardize its own surveys is included herein as Appendix B. This form incorporates the following features:

- It is general enough for use with nearly any site.
- It is designed to be used by surveyors who have received training in its use. Many of the entries must be made in blank spaces--the surveyor must know what to ask and what to record.

- The format follows the progress of a typical survey. The first page records information from the interview, the second from the site inspection, and the third from a well inspection, if one is made.

The extent of variation in recorded responses can be controlled by providing each surveyor with a separate list of possible entries, particularly for such items as "Type of Sewage Disposal System" and "Surveyor's Visual Observations of Effluent Disposal Site." Because terminology changes from place to place, one of the steps in preparing for a sanitary survey may be developing such a "crib sheet" based on interviews with local officials and contractors.

Lastly, the logistics of getting together surveyors, forms, sampling equipment and containers (if any), and suitable transportation for the surveyors must be worked out. For quick-turn-around surveys, proper planning of such small matters may be critical to the survey's success.

4. ON-SITE SANITARY INSPECTIONS

The basic elements of an on-site sanitary inspection are an interview with the resident and an inspection of the property, focusing on the locations of on-site wastewater facilities. To these may be added as appropriate: a well or spring inspection, sampling and analysis of the water supply, sampling and analysis of groundwater that directly receives wastewater effluent, soil sampling and analysis, and follow-up inspection by more experienced personnel.

At the beginning of an interview, it is imperative that the surveyor clearly state the purpose of the interview, the name of the sponsoring agency, and the name of the project. An explanation of how the information will be used should also be offered if the resident is not already informed.

Since many sanitary surveys will be conducted prior to the establishment of mandatory access provisions, the resident's permission to conduct an inspection will be required. A signed letter of introduction from the head of the sponsoring agency may help obtain permission.

The surveyor should judge whether the resident is competent to provide the needed information. Small children, visitors, short-term renters, baby-sitters, etc., may be willing to allow access and to answer questions, but may not be able to provide adequate responses. A given resident may not be the person who is best informed about the property, but may nevertheless be able to describe past problems with the on-site system, or provide data on occupancy and water-using fixtures. Depending on the purposes and design of the survey, a surveyor may decide to terminate the inspection, to use incomplete or unreliable data from anyone present in the residence, to return when the best-informed resident will be available, or to search for landlords or other informed persons. Some surveyors have used a system of response ratings in order to quantify the willingness of residents to respond and their ability to provide information. These ratings will be discussed in Section 5 below, in relation to data interpretation.

The recommended survey form reproduced in Appendix B shows data that can be aquired during resident interviews. The data categories are explained on pages immediately following the three-page form. Information acquired during the interview should be entered on the form immediately, since to delay recording this information can lead to unnecessary errors and often means that the information is never recorded.

As mentioned earlier, the recording of information from the interview will be aided by a "crib sheet" containing the most likely responses to selected items. Wide variation exists in local terminologies referring to plumbing fixtures, parts of on-site systems, and system failures. To facilitate discussions with residents, and to make survey entries as uniform as possible, lists including both local and more widely accepted terms might be prepared for each surveyor. Drawings of typical parts of on-site systems and wells would also aid communication.

The next step is an inspection of the property, with the resident's permission, and preferably, accompanied by the resident. Known or suspected locations of all sanitary facilities should be observed. If the resident is not able to identify facility locations, it will normally be appropriate for the surveyor to estimate relevant information based on such factors as the topography of the lot, the locations of wells, and previous knowledge of systems serving houses of similar age. The degree of uncertainty in such estimates should be noted by comments in the margin of the survey form.

The property inspection should not be limited to reported sanitary facilities. Streams, drainage ditches, field tiles, lakes, and ponds on or adjoining the property should be inspected for signs of illegal discharges, nutrient enrichment, and possible impact on drinking water supply. Property boundaries should be walked in search of unreported wastewater discharges. Surface drainage that may interfere with proper soil absorption should also be noted. Any nearby water supplies that may be affected by existing systems or by possible replacement facilities should be located.

Page 2 of the form in Appendix B provides space for recording visual observations in narrative form and in a sketch. A check-list of drainage characteristics is also provided. Surveyors should be provided with ruler, compass and pencils to sketch the property, buildings, water bodies, and sanitary facilities.

Many buildings served by on-site wastewater systems also receive their fresh water from on-site wells or springs. Decisions to continue using on-site wastewater systems must consider the impact these systems are having on such water supply sources. These sources will be located during the property inspections, and should be examined to check for the integrity of casings, adequacy of surface drainage, proper venting, and the presence of grouting (in some geologic settings). During initial surveys, wells that may also be subject to contamination from sources other than on-site wastewater systems should not be sampled, since positive analysis for coliform bacteria or nitrates might be due to inflow from surface runoff or other non-wastewater sources, and decisions to modify or abandon wastewater systems based on well sampling results would then be in error. Results of all well inspections and other data that the resident may have on well depth and depth to water may be recorded on the third page of the survey form in Appendix B.

During intensive surveys, a number of surveyors may be working at the same time. Some or most of them may be inexperienced. Two devices have been used to maintain the quality of information developed from such surveys. One is a daily feedback meeting in which surveyors are brought together to summarize and discuss difficulties in conducting inspections, and to report notable results. These meetings provide additional training for surveyors, and reinforce their use of uniform terminology, while providing survey leaders with an opportunity to gain input on survey progress and results. Such meetings also provide opportunities for adjustments of survey design while the surveyors are still in the field.

The other device is a follow-up inspection by more experienced personnel. If most of the survey staff are inexperienced, they may be able to find malfunctions, but be unable to assess the causes of the malfunctions or to prescribe supplemental investigations. Such reinspections by experienced surveyors may yield optimum survey results for the funds spent.

5. INTERPRETATION

Each property that is surveyed is unique, and may vary widely from other properties in terms of site conditions, system design and usage, and resident responsiveness. It may seem to a surveyor that to distill the many bits of data from such a survey into useful planning information is not possible with just a partial sanitary survey and some representative sampling. It could even be argued that precise descriptions of necessary on-site remedies could be made only after replacement facilities and renovations are constructed--and even then we would not know whether the best designs had been selected until the systems had been in operation for several years! How, then, should a facilities planner proceed?

This question can be answered on two levels. The first level is that of the individual system. If continued use of on-site systems is proposed, it is at this level that a community's wastewater problems will be solved--one system at a time. The on-site sanitary inspection forms record the first step in the process of designing and constructing appropriate remedies for individual on-site systems. Subsequent steps will also be taken for individual systems, one system at a time. Following these steps for all the systems that need them will result in economies of scale, as well as comprehensive and effective abatement of problems.

The second level is that of the community. It is possible that a decision may be made to inspect and upgrade individual systems in a community-wide program without specific or conclusive knowledge as to what will be done on each property. At this community-wide level, the task is not to select specific remedies for individual properties, but to assess for the community as a whole (or significant parts of it) the following:

- The feasibility of abating most or all of the community's water quality and public health problems with on-site remedies;
- The comparative long-term costs of doing nothing, of constructing on-site remedies, of constructing centralized sewers, or of some combination of these options; and
- The environmental and social impacts of the major options.

If facilities planning before and after a sanitary survey is conducted with these assessments in mind, and the survey is designed to provide appropriate (even though approximate and partial) information, then good facilities planners can make the essential, community-wide decisions using survey data. Of course, in arriving at these community-wide decisions, some determinations must be made as to whether selected individual systems should be abandoned, replaced, renovated, repaired, or left along. But it is not essential that these assessments be conclusive for individual systems, or that all individual systems be surveyed.

In the interpretation of individual on-site sanitary inspections, two types of possible responses from residents should be considered: those based on poor knowledge of their systems, and deceptive responses. Many residents, even heads of households and property owners, can provide only vague information on the design and maintenance of their systems. This, of course, is a significant factor in the failure of many systems. But it also limits the amount of information that a surveyor will have to assess the system, it may increase costs by requiring further work to complete the assessment, and it increases the burden on the surveyor to understand and interpret other clues to system design and performance. As time and expense allows, and the purpose of the survey demands, the absence of knowledgeable persons can be overcome by return visits, written or telephone inquiries, review of official permit records, or requests for information from contractors who may have installed or serviced the wastewater system. If no resident or owner has the necessary information, only the latter two methods would be fruitful.

A more insidious problem is that of deceptive responses. Individuals may not disclose the requested information if they do not understand the purpose of an on-site inspection, or if they are concerned about costs that may be incurred as a result of what is learned. However, some information, such as problems with plumbing backups, can be obtained from no other source. On the other hand, residents who support abandonment of on-site systems may exaggerate the frequency or impact of problems.

In some cases, deception may go undetected by the surveyor. However, there will often be some evidence or clue that such deception is occurring. These clues can include unwillingness on the part of a resident to allow inspection of the property, signs of surface malfunction where none was claimed, or behavioral clues that indicate a resident's attitude. Again, the surveyor's response to perceived deception depends on the purpose of the survey. If the purpose is just to locate problems by a partial survey, the inspection may be completed but not included in a statistical description of the problems in a community. However, if the survey is part of an ongoing program to upgrade systems, follow-up inspections by senior project members or by local officials may be necessary. The training and judgment of surveyors are critical factors in proper handling of possibly deceptive responses.

Whatever the reason, the quality of information provided by residents will vary. To account for this variable, some surveyors use a rating system, of which the following is an example. This system was tested during a targeted, partial sanitary survey at Lake George, New York. After each interview, the surveyor placed a number from 0 to 5 on the first sheet of the survey form. These numbers corresponded to the following assessments of residents' responses:

5- Resident has design drawings for system construction or repairs or witnessed construction; has lived in dwelling long enough to know of any problems; can communicate knowledge of system accurately.

4- Resident has hearsay knowledge of system design; has lived in dwelling long enough to know of any problems; can communicate knowledge so as to be understood.

3- Resident has lived in dwelling long enough to know of surface malfunctions or backups, and has general understanding of system components and their location. Not familiar with technologies used or appropriate terminology. Explanations from surveyor required to complete interview.

2- Resident can report on gross problems with system but can provide little information on age, location, or design. Possible communication problems.

1- Resident can provide no information on system design and little, if any, relevant information on surface malfunctions or backups.

0- Resident uncooperative. Information suspect or not provided.

Such a qualitative rating system for resident responses can assist survey leaders in directing efforts to other sources of information or in extrapolating the inspection data to unsurveyed sites (see below).

Conclusions can be drawn from on-site sanitary inspections on the two levels discussed previously, i.e., for individual systems and for the community. Many inspections will reveal no potential or ongoing problems. Of the remainder, the appropriate course of action to remedy or forestall problems will be readily apparent for some properties without additional inspections or testing. Some fraction, however, will require one or more special measures before an appropriate course of action can be selected and designed. Interpretation of an on-site sanitary inspection at the individual system level, then, includes categorizing the system according to whether there is no problem, a problem with obvious solutions, or a problem which requires additional work before a solution can be selected or designed. For this latter category, interpretation should include prescribing the appropriate tests and inspections (See Technical Reference Document II.J. - Site Analysis and Technology Selection for On-Site Systems). To this end, the recommended survey form in Appendix B has resident approval blanks at the bottom the of the first (interview) page.

Informed judgement is required to reliably interpret survey results in terms of 1) assessing the causes of surveyed failures, 2) prescribing, to various degrees of certainty, what the appropriate solutions will be, and 3) extrapolating these data and judgments from surveyed properties to unsurveyed properties. The degree of certainty to which these conclusions can be made will depend on the same factors that were considered in designing the survey, i.e., the purposes of the survey, the expertise and time available to perform the survey, and prior knowledge of systems' performance. The strength of conclusions drawn from the survey will also depend on whether the causes for system failures can be adequately analyzed from the information gathered.

APPENDIX A

WASTEWATER TREATMENT QUESTIONNAIRE

Please indicate your answer by checking "√" the appropriate response where applicable.

1. What kind of wastewater treatment system does your home primarily use?

____ Septic tank
____ Cess pool
____ Direct discharges } → Skip to Question 3
____ Privy (outhouse) }
____ Holding tank
____ Other (describe) _____

2. Where does your system discharge?

____ Seepage field only
____ Seepage field plus surface discharge
____ Surface discharge through tile line
____ Other (describe) _____

3. Does your system discharge to a ditch or tile line?

____ Yes
____ No . → Skip to Question 5

4. Where does this ditch or tile line eventually flow? (specify end point)

_____
_____

5. What is the age of your present system?

____ years (or) ____ months

6. Approximately how many times has your treatment system had solids removed from it in the last five years?

____ times

7. What is the depth of your wall?

____ feet

8. Is your well proceeded from run-off by being covered?

____ yes
____ no

9. What water sources, other than for the bathroom(s), are connected to your treatment system? (√ all that apply.)

Question 9 continued

____ laundry
____ garbage disposal (grinder)
____ dishwasher
____ other (describe) _____

12a. Are there any liquid wastes which discharges some place other than your primary treatment or sewage system?

____ yes
____ no → Skip to Question 13

12b. Which wastes are these (sump pump, footing drain, roof drain, etc.)

_____
_____

12c. Where do these wastes go?

_____
_____

13. When do you usually perform maintenance tasks on your treatment system?

____ on a regular basis (for example, every summer.)
____ after a problem develops
____ never
____ other (describe) _____

14. What serious problems have you experienced with your treatment system?

14a. ____ Backup of wastes in the house
Caused by: ____ Unusually heavy water use
____ Plugged pipe to septic tank
____ Full septic tank
____ Other (describe) _____
_____

Solved by: ____ Reduce water use
____ Clear obstruction from pipes
____ Pump septic tank
____ Rebuild system
____ Other (describe) _____
_____

14b. ____ Odorous water surfacing at tile field
Caused by: ____ Unusually wet weather
____ Unusually heavy water use
____ Full septic tank
____ Plugged tile lines
____ Other (describe) _____
_____

Solved by: ____ Drier weather
____ Reduce water use
____ Pump septic tank
____ Install new tile lines
____ Other (describe) _____

_____

14c. ____ Other problems (describe) _____

_____

Caused by: _____

_____

Solved by: _____

_____

15. Are you aware that the Environmental Protection Agency would fund (if the money is available) 85% of the cost of upgrading septic tank systems if the Sanitary District would assume responsibility for them?

____ yes
____ no

16. Is your home a permanent or seasonal residence?

____ Permanent → Skip to Question 18
____ Seasonal

17. Approximately how many weeks out of the year is your house inhabited by you, your family or friends?

____ weeks

18. Approximately how long have you owned this home?

____ years (or) ____ months

The following information is needed so that sewage treatment needs in different areas can be identified. Again, all answers are confidential.

19. Which subdivision is your house located in?

_____ (name)

20. What is the address of your property?

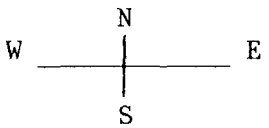
_____

_____

Question 20 continued

Below, please make a sketch of your lot showing the following items:

____ the lake
____ the roads
____ your house
____ the well
____ the wastewater treatment system
____ the appropriate distance in feet between the treatment system and the lake
____ the appropriate distance in feet between the wastewater treatment system and the well



Thank you very much for your help on this study.

APPENDIX B

SUGGESTIONS FOR USE OF
"SANITARY SURVEY OF CONSTRUCTION GRANTS APPLICATION"

This three-page form is intended to be used during initial sanitary inspections of on-site wastewater systems. The intent of this form is to guide surveyors in interviewing residents and inspecting their properties so that results are recorded as uniformly as possible, and include factors that explain system performance as well as document satisfactory or unsatisfactory performance.

This form assumes a certain level of knowledge on the part of surveyors about on-site systems and the terminology used in describing them. Therefore, instead of lengthy checklists for some entries, blanks are left to be completed at the surveyor's discretion. This form should not be used as a mailed questionnaire, and should not be used by untrained surveyors. However, experienced has demonstrated that a one-day training program can adequately prepare people as surveyors. Such a program would include three to four hours of instruction and two or three on-site sanitary inspections in the company of a trained surveyor.

Recommended conventions in completing the entries are discussed below in the order they appear on the form.

Resident - Enter the name of the person interviewed. If later correspondence might be sent to a head-of-household who has not been not interviewed and is not the owner, also enter his or her name in parenthesis. Establish local conventions for completing this entry for non-residential properites.

Owner - If different from the person interviewed, enter name. Also enter address of owner if he or she does not reside here.

Address of Property - Self-explanatory.

Lot Location - Provide sufficient detail to enable others to find the property.

Tax Map Designation - Enter tax map identification code, file number or locally developed code to enable property to be located on available maps such as soils maps, topographic maps, and tax maps.

Study Area - Enter name of community being surveyed, facilities planning area, or locally developed identifier for the sanitary survey.

Surveyor/Date - Self-explanatory.

Weather - Note recent wet or dry periods or other climatic factors that might influence on-site system performance at the time of inspection.

Approximate Lot Dimensions - Field estimates arrived at visually or by pacing may be sufficient for preliminary surveys. Measurements from tax maps,

tape measurements, or land surveys may also be called for in more intensive surveys. Enter road or water frontage first. Local convention may call for entering acreage here instead of dimensions.

Age of Dwelling - For preliminary surveys, the resident's estimate will normally be sufficient. For determination of eligibility for federally funded collector sewers, it is necessary to know whether the dwelling was built prior to October 1972. For federally funded individual sewage disposal systems or cluster systems, it is necessary to know if the dwelling was built before December 1977.

Age of Sewage Disposal System - If available, this information should come from the construction permit for the system. Resident's estimate may be the only available information.

Type of Sewage Disposal System - Information on the existing system provided by the resident may have to be supplemented by construction permit records. Of specific concern are:

- Use of holding tank. If used, cite size, frequency of removal and arrangements for removal.

- Size and construction of the septic tank, including material of construction, whether open-bottomed or water-tight, and type of sanitary tees and baffling.

- Size, construction, and type of aeration tank if used instead of septic tank.

- Means of effluent transport to the disposal site, whether by gravity or pump. If pumped, note presence of alarm device, provision of back-up pumps, parallel pressure lines, type of pump pit, and reason for pumping, whether to overcome topography or for dosing disposal site.

- Type of effluent disposal. Examples are cesspools, dry wells, shallow drainage beds, lateral trenches with equal distribution, lateral trenches with serial distribution, sand mounds, evapotranspiration beds, and sand filters with surface discharge. Record dimensions of cesspools and dry wells. Record number of trenches or beds and square footage of trench or bed bottom, if available.

- Presence of multiple disposal systems such as dry well for laundry waters, composting, recycling or incinerator systems for toilet wastes or separate grey water and toilet wastes systems.

Maintenance - These entries are useful not only for the expected value of the information, but also as an index to the user's knowledge of maintenance requirements. Therefore, note in the right hand margin your assessment of the interviewee's being "responsible" for the system's maintenace (owner or head of household) or "not responsible" (children, visitors, etc.). Record interviewee's responses, including "don't know."

Dwelling Use: Permanent Residents - Record the number of adults and children normally living in dwelling. Note whether any changes in occupancy have occurred in the recent past, i.e., up to one year.

Dwelling Use: Number of Bedrooms - The number of potential bedrooms in a dwelling generally limits its maximum occupancy rate, and so should be noted for planning purposes. Questions arise as to the classification of some spaces. A bedroom is defined for survey purposes as a room which can be closed off from the rest of the dwelling and is not equipped specifically for other purposes. The number of rooms normally in use as bedrooms are to be counted as "actual" bedrooms on the survey form. Unfurnished basements or other space which could easily be converted should be discussed with the resident to determine whether there exists a possibility for future "potential" bedrooms and their number. Bedrooms the resident is considering adding by constructing new floor space should be entered as "planned."

Dwelling Use: Seasonal Use - Many variations in the level of occupancy are found, especially in areas with recreational potential. Increased use by weekend visitors, summer use by weekly renters, second home use on weekends and full-time during the summer, and temporary use by migrant workers are examples of the many types of variation in usage. Note the pattern of such uses and approximate occupancy. Assume two people per bedroom if no better information is available.

Water-Using Fixtures - Residences in rural areas often are not provided with community water supplies. Water use data for use in predicting wastewater flows will, therefore, not be available for many rural residences. Knowledge of the dwelling use and of water-using fixtures can provide a basis for estimating wastewater flows. Indicate the number of each type of fixture in the spaces provided. Note which of the fixtures are designed to conserve water. Also, if the residence is served by more than one sewage disposal system (dry well, separate drain field, etc.), identify which fixtures discharge to the smaller system by circling the names of the fixtures.

Problems Recognized by Resident - Problems with the existing sewage disposal system will depend on the type of system. For typical septic tank-subsurface drain field systems, problems recognized by the resident may be limited to plumbing backups and leakage into basements. Odors and ponding over the absorption field may be as apparent, but may occur only periodically. Malfunction of aeration devices, clogging of sand filters, discharges from broken hoses or effluent sewers, and seepage at the edges of mounds are other examples of inadequate operation. The seasonality of such problems should be noted. If wells are known or suspected to have been contaminated by on-site or neighboring sewage disposal systems, note this with the resident's explanation of the route of contamination, i.e., through groundwater or surface flows.

At this point in the interview, the resident may describe other sewage disposal problems in the neighborhood. This could be useful information, but the interviewee may not want this information recorded. This could become an ethical or legal problem. The leader of the sanitary survey should set a consistent policy on this issue prior to the interviews.

Resident Will Allow Follow-up Engineering Studies - As part of the sanitary survey, detailed site analysis may be required on selected lots to evaluate operation of the effluent disposal area and to determine the local groundwater impacts of effluent disposal. These studies may include probing the disposal area, augering soil samples, or installation of shallow groundwater observation shafts. As these studies could be disruptive of the property and the resident's privacy, written permission to conduct the studies would be necessary. Determining the resident's cooperativeness and noting it here on the survey form does not constitute that permission. However, knowledge of the resident's cooperativeness will facilitate implementation of follow-up engineering studies, and so should be noted.

Page Two - Water Supply

The interview should continue at least through the first two entries concerning water supply source and cost and usage data. Remaining entries on this page and the third page shold be completed during inspection of the site and its water supply, wastewater and drainage facilities.

Surveyors may require training on local well construction techniques.

Page Three - Site Inspection

The information recorded on page one is derived primarily from an interview with the resident as supplemented by data from tax assessor's records and health department records. In contrast, information to be recorded on page three will be based on the surveyor's inspection as supplemented by resident's input and possibly health department records.

Optimally, the resident will allow a visual inspection of the property and will be able to identify the location of parts of the sewage disposal system, surface drainage facilities and the water supply. In this case the information for Page Two should be readily obtainable.

In some cases, the resident may not allow the surveyor to walk the premises. The surveyor will be limited to recording observations made from the property boundary. If the survey is limited in this manner, this fact should be noted.

If the resident is not able to identify facility locations, and if health department records do not fill such information gaps, it will normally be appropriate for the surveyor to estimate relevant information based upon lot topography, well location, etc. However, where the surveyor has to make educated guesses about the type, size or location of facilities, the degree of uncertainty should be noted by comments in the margin.

Surveyor's Visual Observations of Effluent Disposal Site - This space is to be used for a narrative description of problems indicated by any part of the sewage facilities or the property. The lack as well as the presence of visible or olfactory evidence of malfunction should be noted. The surveyor should attempt to determine whether past malfunctions have been disguised by directing excess flows to drainage ditches, storm sewers or neighboring

properties. Any site conditions which may limit the effectiveness of soil absorption units, such as steep slopes, proximity to surface waters, etc., should be noted.

Drainage Facilities and Discharge Location - Soil-dependent effluent disposal systems can be hydraulically overloaded by improper diversion of groundwaters or surface waters. Careless diversion can also cause well contamination. Careful inspection of drainage facilities may lead to effective, easily implemented measures to remedy malfunctions and prolong the useful life of the system. The resident may have to provide information on the presence and discharge location of basement sumps and footing drains. Discharge from roof drains and driveway runoff can normally be located visually. Note the presence and discharge locations of these types of drainage narratively in the space provided. Draw the discharge locations on the Property and Facilities Sketch below.

Property and Facility Sketch - In the space provided draw the following items:

- Arrow pointing north
- Location and name of road access
- Lot boundaries, indicating approximate lengths
- Dwelling location and orientation
- Well location, indicating approximate distances to nearest lot boundary, dwelling, septic tank or other storage/treatment unit, and soil disposal area.
- Septic tank or other storage/treatment unit location and capacity, if known.
- Soil disposal area or other disposal site location and dimensions, depth, and design features if known.
- Drainage discharge locations.
- Surface drainage features such as storm sewer grates, ditches, streams or lakes. Show direction and approximate distance to surface drainage features if not adjacent to lot.

APPENDIX B

ON-SITE SANITARY INSPECTION FORM

SANITARY SURVEY FOR CONSTRUCTION GRANTS APPLICATION

Resident: Study Area:

Owner: Surveyor/Date:

Address of Property: Weather:

Lot Location: Approximate Lot Dimensions:

Tax Map Designation: _____feet by _____feet

Preliminary Resident Interview

Age of Dwelling: ___ years Age of sewage disposal system: ____years

Type of Sewage Disposal System:

Maintenance: ____years since septic tank pumped. Reason for pumping:_____
____years since sewage system repairs (Describe below)
Accessibility of septic tank manholes (Describe below)

Dwelling Use: Number of Bedrooms:____actual, ____potential, _____ Planned
Permanent Residents:____adults, ____children
Seasonal Residents:____, length of stay____
Typical Number of Guests:____, length of stay____

If seasonal only, plan to become permanent residents:____ In how many years?____

Water Using Fixtures (Note "w.c." if designed to conserve water):

____Shower Heads	____Kitchen Lavoratories	____Clothes Washing Machine
____Bathtubs	____Garbage Grinder	____Water Softener
____Bathroom Lavoratories	____Dishwasher	____Utility Sink
____Toilets	____Other Kitchen	____Other Utilities

Plans for Changes:

Problems Recognized by Resident:

Resident Will Allow Follow-Up Engineering Studies: ____Soil Borings _____Groundwater
____Well Water Sample

SANITARY SURVEY FOR CONSTRUCTION GRANTS APPLICATION

Water Supply

Water Supply Source (check one)
_____ Public Water Supply
_____ Community or Shared Well
_____ On-Lot Well
_____ Other (Describe)

If public water supply or community well:
_____ Fixed Billing Rate $ /
_____ Metered Rate $ /
Average usage for prior year: /

If shared or on-lot well:
_____ Drilled Well
_____ Bored Well
_____ Dug Well
_____ Driven Well

Well Depth (if known): _____ feet total _____ feet to water table

Well Distance: _____ feet to house _____ feet to septic tank

_____ feet to soil disposal area _____ feet to surface water

Visual Inspection: Type of Casing

Integrity of Casing

Grouting Apparent?

Vent Type and Condition

Seal Type and Condition

Water Sample Collected:

_____ No

_____ Yes

(Attach Analysis Report)

SANITARY SURVEY FOR CONSTRUCTION GRANTS APPLICATION

Surveyor's Visual Observations of Effluent Disposal Site:

Drainage Facilities and Discharge Location:

Basement Sump

Footing Drains

Roof Drains

Driveway Runoff

Other

Property and Facility Sketch

H. EVALUATION OF THE DOWSER™ GROUNDWATER FLOW METER

1. INTRODUCTION

In conjunction with septic leachate detection surveys in several of the Seven Rural Lake communities, a new groundwater flow meter was operated for the first time on a full-scale basis. While considered at the time to be an experimental, not a proven, device, the K-V Associates' Dowser™ groundwater flow meter provided information on the rate and direction of groundwater movement not otherwise available. The data generated, although subject to some concern over quantitative accuracy, insights to local groundwater hydrology. These insights helped explain other field study results and, in part, guide the description and impact evaluation of the EIS's recommended actions.

This report discusses the Dowser™ particularly in respect to traditional groundwater flow estimation methods.

The somewhat academic explanation of groundwater data gathering and analysis techniques that follows is to help the non-geohydrologist understand the assumptions and thus the limitations of the techniques. After the existing techniques are presented, the technical basis and applications of the Dowser™ groundwater flow meter is discussed. The Dowser™ flow meter is a tool useful for expanding and simplifying capabilities of geohydrologic monitoring and modeling, but not for replacing those existing techniques.

2. THE EXPERIMENTAL BASIS OF GEOHYDROLOGY - DARCY'S LAW

The most common geohydrologic models employed in site investigations are those predictive models of groundwater flow directions, rates, and discharge. These models may consist of physical, analog, or digital methodologies, but all are basically derived from the empirical relationships discovered by Henry Darcy in 1856.

Darcy is credited with being the first to recognize a relationship between the velocity of flow through a porous medium (earth materials) and the forcing function (hydraulic head differences). Using a column packed with different combinations of soils and hydraulic head differences, Darcy recognized the following experimental relationship.

$$V \propto \iota \qquad (1)$$

where V = groundwater flow velocity
and ι = hydraulic head gradient
or

$$V = K\iota, \qquad (2)$$

where K is the constant of proportionality and includes the properties of the fluid and the saturated, fluid-bearing zone. In geohydrology, the properties (especially kinematic viscosity) of the fluid (usually water) are assumed to be constant throughout the range of values normally encountered. The value K is thus usually referred to as the Coefficient of Permeability and written as k. The Coefficient of Permeability is a property of the earth material.

Thus defined, groundwater flow velocities may be calculated from the relationship

$$V = k\iota \qquad (3)$$

or the value of k may be determined in the laboratory by

$$K = \frac{V}{\iota} \qquad (4)$$

Groundwater discharge (Q) is determined by multiplying the cross sectional area (A) by the flow velocity:

$$Q = V \cdot A \qquad (5)$$

or

$$Q = (k\iota) \cdot A \qquad (6)$$

3. EXISTING GROUNDWATER FLOW MODELS

Applying this relationship to field investigations, groundwater gradient is established by measuring static groundwater elevations at three or more non-colinear locations (usually by measuring water levels in three wells). A site map of equipotentials is constructed. The maximum gradient found on this map is assumed to represent the groundwater flow gradient. Groundwater flow is assumed to occur parallel to streamlines at right angles to the equipotentials (Figure II-H-1).

The coefficient of permeability of the aquifer (water-bearing zone) is determined either by measuring samples of the material in the laboratory or by choosing a value from the existing literature (Table II-H-1). The discharge area is usually defined to include the saturated thickness of the aquifer between the extreme streamlines which bound the site.

Limitations of this simplified method stem from the distribution of the wells and from the determination of aquifer coefficient of permeability. Observation wells must be constructed with a distribution suitable to detect variations and singular points within the site. Some singular points which may exist on a site and might affect the geohydrology include pumping wells, pods or lenses of low permeability material, deep foundation, absence of lower confining zones or sites of surface or subsurface recharge. Determination of the coefficient of permeability from tables or discrete samples may introduce errors by neglecting the variability of the material, secondary permeability caused by bedding planes, joints or other structural features.

4. ALTERNATIVE DETERMINATION OF AQUIFER PROPERTIES - PUMP TESTING

An alternative and more accurate method of determining the hydraulic properties of a water bearing zone is through the use of a pump test.

In one type of pump test, a production well and at least one fully penetrating observation well are required. The production well is pumped at selected rate(s) and drawdown of the water table at the observation well(s) is recorded. After completion of this test, recovery of head in the pumping and observation wells is recorded. The result of this analysis is the determination of the transmissibility of the water-bearing zones. Transmissibility is a value derived from the test result through the use of other empirical relationships. It is defined as the rate of flow through a vertical section of an aquifer whose height is the thickness of the aquifer and whose width is one foot, when the hydraulic gradient is 1.00.

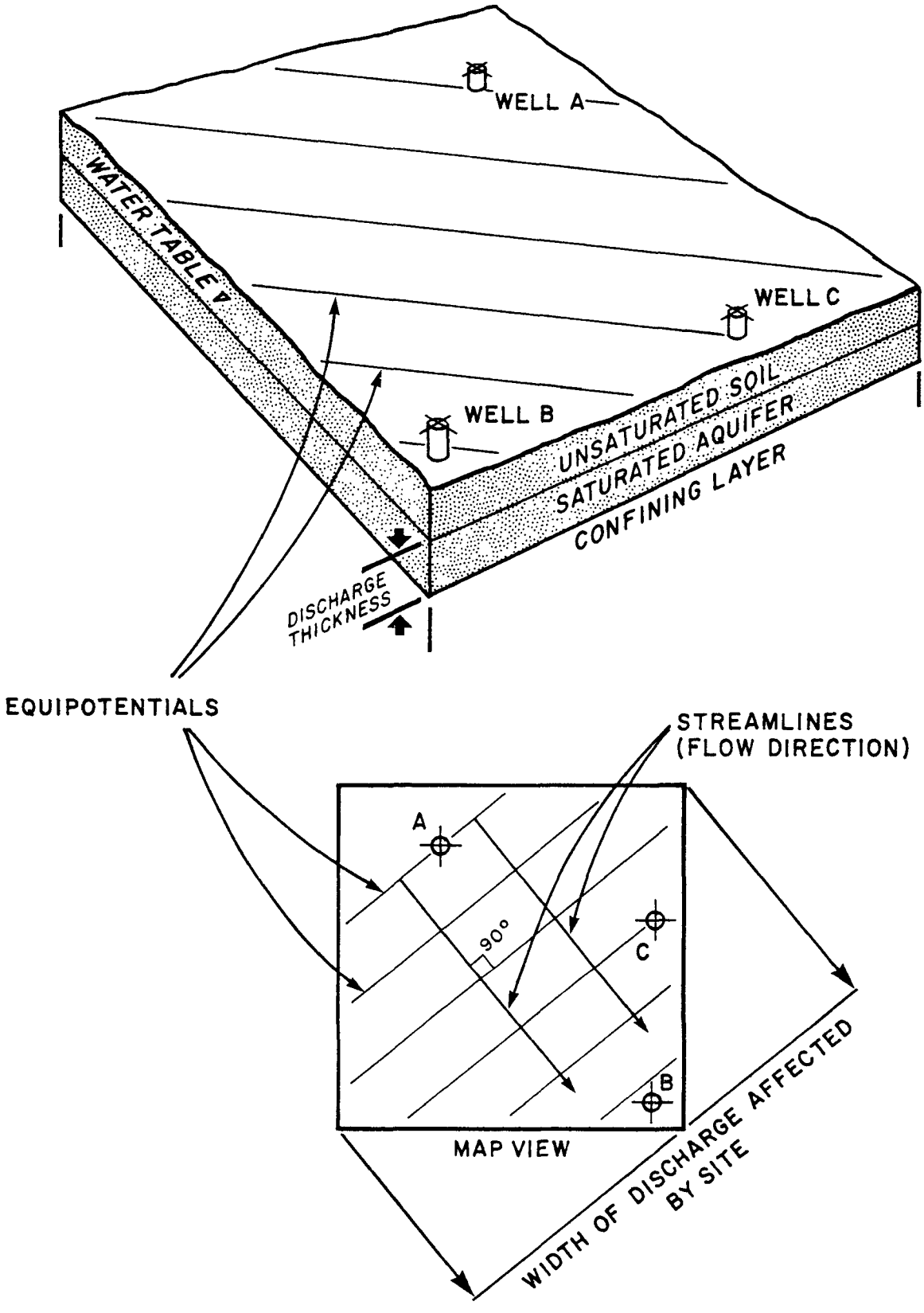


Figure II-H-1. Diagrammatic representation of basic site geohydrologic investigation for a water table (unconfined aquifer).

TABLE II-H-1. TYPICAL VALUES OF PERMEABILITY COEFFICIENTS

		Particle Size Range				"Effective" size		Permeability Coefficient--k		
		Inches		Millimeters						
		$D_{max.}$	$D_{min.}$	$D_{max.}$	$D_{min.}$	D_{20}in.	D_{10}mm.	Ft./yr.	Ft./mo.	Cm./sec.
TURBULENT FLOW	Derrick STONE	120	36	-	-	48	-	100×10^6	100×10^5	100
	One-man STONE	12	4	-	-	6	-	30×10^6	30×10^5	30
	Clean, fine to coarse GRAVEL	3	1/4	80	10	1/2	-	10×10^6	10×10^5	10
	Fine, uniform GRAVEL	3/8	1/16	8	1.5	1/8	-	5×10^6	5×10^5	5
	Very coarse, clean, uniform SAND	1/8	1/32	3	0.8	1/16	-	3×10^6	3×10^5	3
LAMINAR FLOW	Uniform, coarse SAND	1/8	1/64	2	0.5	-	0.6	0.4×10^6	0.4×10^5	0.4
	Uniform, medium SAND	-	-	0.5	0.25	-	0.3	0.1×10^6	0.1×10^5	0.1
	Clean, well-graded SAND & GRAVEL	-	-	10	0.05	-	0.1	0.01×10^6	0.01×10^5	0.01
	Uniform, fine SAND	-	-	0.25	0.05	-	0.06	4000	400	40×10^{-4}
	Well-graded, silty SAND & GRAVEL	-	-	5	0.01	-	0.02	400	40	4×10^{-4}
	Silty SAND	-	-	2	0.005	-	0.01	100	10	10^{-4}
	Uniform SILT	-	-	0.05	0.005	-	0.006	50	5	0.5×10^{-4}
	Sandy CLAY	-	-	1.0	0.001	-	0.002	5	0.5	0.05×10^{-4}
	Silty CLAY	-	-	0.05	0.001	-	0.0015	1	0.1	0.01×10^{-4}
	CLAY (to to 50% clay sizes)	-	-	0.05	0.0005	-	0.0008	0.1	0.01	0.001×10^{-4}
	Coloidal CLAY (-2 μ $\geq$ 50%)	-	-	0.01	10Å	-	40Å	0.001	10^{-4}	10^{-9}

The calculation of transmissibility, like other test data, is based upon certain assumptions and generalizations. For accurate results, field conditions must satisfy the assumptions upon which the formulae are based. A pumping test may be a very useful method of determining the hydraulic properties of water-bearing zones. It may yield reliable results which, in general, are representative of average conditions within the zone of influence of the pumping well. It does not give the details of groundwater flow at any point in that zone.

5. THE DOWSER^{TM} GROUNDWATER FLOW METER

The Dowser^{TM} groundwater flow meter is one of the first of an expected new breed of groundwater monitoring devices. Recent increased interest in groundwater supply and quality makes it seem likely that this market area will soon experience an explosion in the number of similar devices in the near future.

The Dowser^{TM} represents an attempt to measure groundwater flow directions and velocities without utilizing the Darcy law computations. A central thermister generates a pulse of heat which is transmitted to the groundwater. This heat pulse is transmitted through the groundwater more rapidly than the actual groundwater flow rate. Net movement of groundwater past the probe creates a bias in thermal conductance which is linearly proportional to the groundwater flow velocity.

The main advantages of utilizing this type of device include the rapidity with which measurements may be made, the availability in the field of groundwater flow direction data, the availability of additional information on groundwater flow at discrete locations on a site, and independence from reliance upon the assumptions in Darcy low calculations for flow velocity. Measurements of typically slow (on the order of tenths to tens of feet per day) groundwater flow velocities may be made rapidly because the device measures bias in the thermal conductance of groundwater rather than groundwater flow itself. Since the velocity of the heat pulse is much more rapid than the groundwater flow rate, single measurements may be made in a short time (less than 15 minutes). The thermal "tracer" used in the measurement is non-polluting and not likely to be masked by the presence of other properties of the earth materials or groundwater. The heat pulse tracer is also not likely to be a contaminant of the site. This contrasts with the use of dyes or ions which are otherwise used in groundwater tracer studies. These other tracers may be absorbed and desorbed from earth materials in a chromatographic sense which masks the true groundwater flow rate.

The availability of groundwater flow direction data in the field enables the hydrogeologist to modify the field investigation as it proceeds, reducing the need to remobilize drilling equipment or to leave gaps in the data uncollected. Determination of a groundwater flow vector at a single well location within a site provides more detail than would a typical site survey as presently conducted. The present site surveys describe average conditions over the entire site, while a Dowser^{TM} survey would supplement and provide confirmation of the general site conditions at discrete points. Finally, the device measures a flow phenomena which may be calibrated directly to the groundwater flow rate. It eliminates the need for the assumptions used in Darcy flow calculations.

The Dowser™ device requires little technical training for operation, but the interpretation of its results requires an experienced hydrogeologist who is familiar with its use and limitations. First, the device can measure accurately under laminar flow conditions only. This is not a problem for groundwater flow under most conditions although it may be a problem for very rapid flow through uniform coarse gravel where turbulent flow is possible. Under turbulent flow conditions, mixing of the heat pulse in the eddying groundwater would render measurements useless.

A second limitation for its use is in heat conductive earth materials, especially in clay size and gravel size ranges. In the clay fraction, groundwater flow velocities are typically very slow while the slaty texture and mineralogy may induce heat transfer through the clay particles sufficiently to mask the transfer in groundwater. In gravel, the probe thermistors may be in direct contact with a single sediment grain. The heat transfer through this single grain may mask the heat transfer in the groundwater. Groundwater flow paths, measured by this device in gravels, may also err in that the actual water flow paths may miss various detection thermistors as a result of their travel around the individual grains.

The physical size of the probe may introduce limitations on the statistical validity of a measurement. In well layered clays, groundwater flow may be concentrated in silt laminae which are continuous but very thin. The device has physical dimensions which would potentially require the penetration of numerous water conductivity laminae. This would limit the validity of the measurement. In gravel, the number of grains included in the field of the device is not a statistically valid sample. The problem is that discussed in the preceeding paragraph of this review.

The most serious concern over use of the flow meter is its calibration and the comparability of results from the flow meter with the hydraulic head--Darcy's law method of investigation. As more experience is gained with such devices, this problem will be resolved.

The temptation to use this device as a "cure-all" for groundwater monitoring will be great, especially in the hands of persons untrained in hydrogeology. The value of these devices is in providing additional important information for site hydrogeologic investigations. Specifically, while the typical hydrogeologic site investigations, using hydraulic head differences and Darcy's Law calculations, give average results over the entire site, the use of a groundwater flow meter would provide clues to local variations in the flow pattern and a cross-check of the results. This should help to further define hydrogeologic conditions at investigated sites.

As a part of this investigation, WAPORA, Inc. contacted other individuals with experience or exposure to the device. A summary of their experience as related to WAPORA follows:

Mr. Denis Wotterdrig, New York State Department of Environmental Conservation, Bureau of Waste Disposal, observed a demonstration of the Model 10 Dowser™ in October, 1980. The demonstration was held in a site where the DEC had performed previous investigations. Soils were silty sand. He felt that the Dowser™ gave a very accurate estimation of direction. The disadvantage was that to determine velocity, one had to have a good idea of the effective porosity of the soils.

Mr. Greg Vanderlaan, Chief, Superfund Site Survey, U.S. EPA Region V, Chicago, related his experience with the Dowser™ at the Seymore Recycling Plant contaminated waste site in Indiana. He felt that it was a very useful tool in the investigation of source, extent and dispersion of contamination of groundwater. In this case, he felt that the Dowser™ survey was more accurate than the hydraulic head--Darcy's Law methods also used at the site. Also particularly useful to him was the rapidity of results compared to a hydraulic head survey.

Mr. Joseph Fredle, Oil and Hazardous Material Coordinator, US EPA Region V, Cleveland, used a Dowser™ survey to determine the location and direction of plumes of contamination from septic systems and from old strip mines. He felt that it was comparable to other methods, but faster in results.

Mr. Mike Rowyer, Edison Laboratory, US EPA MERL, Edison, New Jersey, evaluated a demonstration of the Dowser™. He also called references supplied by Dr. Kerfot of K-V Associates. The hydrogeologists felt that great care must be taken in the interpretation of results but that results should be valid. Mr. Rowyer was concerned with the validity of the results until they are proven in court actions.

K-V Associates also produces a downhole Dowser™ probe. Specific concerns in the use of this probe were similar to those voiced for the surface unit. In addition, while Dowser™ advertises that it can be used to measure velocity profiles in a well, this is limited by careful sampling of the effective porosity of the materials penetrated by the well, gravel packing and grouting of the well to reduce flow between different water bearing zones through the gravel sack, proper selection of well screen, and packing of the well at each depth to be tested.

Objections to the use of the probe are the costs of drilling and sampling the well, which are needed to satisfy the sampling conditions. Additionally, some respondents stated flatly that if the packing material cannot be totally removed from the well, it could be inconvenient and costly, or it might eliminate any opportunity to prosecute groundwater contamination cases.

I. EVALUATION AND DESIGN OF OFF-SITE SMALL WASTE FLOW TECHNOLOGIES

Many unsewered communities will have areas where continued use of on-site systems is not the best alternative, either for cost reasons or because on-site systems cannot be made to work in those areas. Off-site treatment and disposal must then be considered. There are a number of different technologies available for off-site treatment and disposal of small waste flows.

1. SINGLE PROPERTIES

Usually the first off-site option to be considered for single dwellings or small businesses should be placement of a soil absorption system of appropriate design on a nearby parcel of land. If the transport distance is not great and a suitable site is available, this will often be the least expensive option. Site evaluation and system design will be comparable to similar on-site technologies.

Although holding tanks are typically located entirely on-site, treatment and disposal occurs off-site after transport by a tank truck. While holding tanks are discouraged by health authorities and can have operation costs as high as any other technology, they can be an effective and safe means of wastewater disposal. However, responsibilities for timely pumping of holding tanks and safe treatment and disposal of the wastes should be met by a person, agency, or private organization that has adequate financial means and technical capabilities. Despite the high operational cost of regular holding tank pumping, the use of holding tanks can help achieve the lowest overall cost for a neighborhood or community, if relatively few holding tanks are required. Costs to individual property owners can be kept to a minimum by serious efforts at water conservation or recycling, or by community subsidies for the hauling, treatment, or disposal of the wastes.

Communities that are faced with installing or managing a number of holding tanks for property owners would do well to consider standardizing holding tank design. Easy-access, quick-fit hookups and reliable alarm systems would significantly enhance holding tank operability. State agencies that regulate on-site wastewater systems usually suggest design objectives for holding tanks. Some agencies offer or even enforce specific design details.

2. MULTIPLE PROPERTIES OR LARGE WASTEWATER SOURCES

A wider variety of technologies is eonomically feasible when wastewater from a number of residences or other sources must be collected for off-site treatment and disposal.

a. Cluster Systems

Septic tank treatment and subsurface soil absorption disposal remain competitive with other technologies in the range from 2 to over 100 residences, depending on site availability and suitability, along with other factors. Septic tank treatment may be provided at each residence, or, after wastewaters are collected, at a central location. The effluent is further treated and disposed of by one of the several design variations available for soil absorption fields. Systems with multi-family subsurface soil absorption

fields are commonly referred to as "cluster systems." Unlike the off-site technologies discussed below, no state-of-the-art review has been prepared on evaluation and design of cluster systems.

Two basic approaches can be used to calculate the size of large soil absorption systems. One approach is based on controlling the groundwater mound that forms under groundwater recharge sources. This approach was used to analyze the suitability of two potential cluster system sites described in Technical Reference Document III.B., "Pickerel Lake, Michigan, Cluster System Site Analysis."

The other approach is an extension of on-site system design. Soil percolation rates are used to determine the number of square feet of trench bottom required. This approach is suggested in the University of Minnesota's Home Sewage Treatment Workshop (Machmeier and Hansel, 1980).

b. Land Application

Application of treated wastewater to the surface of the ground can provide better nutrient removal than subsurface disposal, and in larger facilities may be economically competitive.

The state of the art in evaluation and design of land application systems has been well described in Process Design Manual: Land Treatment of Municipal Wastewater (U.S. EPA, 1981). A special chapter in that document is devoted to land application systems handling less than 0.25 million gallons per day.

c. Wetland Systems

Both natural and artificial wetlands can be used to treat and dispose of wastewater. Although wetland systems are a relatively recent innovation, they have been studied extensively.

The reader is referred to several documents for detailed information:

- The Technology Assessment of Wetlands for Wastewater Treatment (Waste and Water International, 1982).
- Design Principles for Wetland Treatment Systems (Hammer and Kadlec, Undated).
- Use of Wetlands for Water Pollution Control (Chan et al., 1982).
- Wetland Systems for Wastewater Treatment: Operating Mechanisms and Implications for Design (Heliotis, 1982).
- The Effects of Wastewater Treatment Facilities on Wetlands in the Midwest (U.S. Environmental Protection Agency and WAPORA, Inc., 1983)

d. Conventional Treatment and Discharge Systems

Small-scale designs are feasible for all of the treatment processes conventionally used prior to discharge of wastewater to surface waters. In small communities, selection of one process over the others is usually based

on the closely related factors of low cost and minimal operation and maintenance requirements.

There is no lack of information on evaluation and design of conventional treatment systems. Any good engineering library should have adequate reference material on evaluation and design of such systems; therefore, specific references on this subject are not cited here.

e. Septage Treatment and Disposal

Wastes collected from septic tanks and holding tanks have to be properly transported, treated, and disposed of or recycled if the water quality and public health objectives of a wastewater management program are to be met.

The state of the art for evaluation and design of septage handling is summarized in Septage Management (U.S. EPA, 1980).

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J. SITE ANALYSIS AND TECHNOLOGY SELECTION FOR ON-SITE SYSTEMS

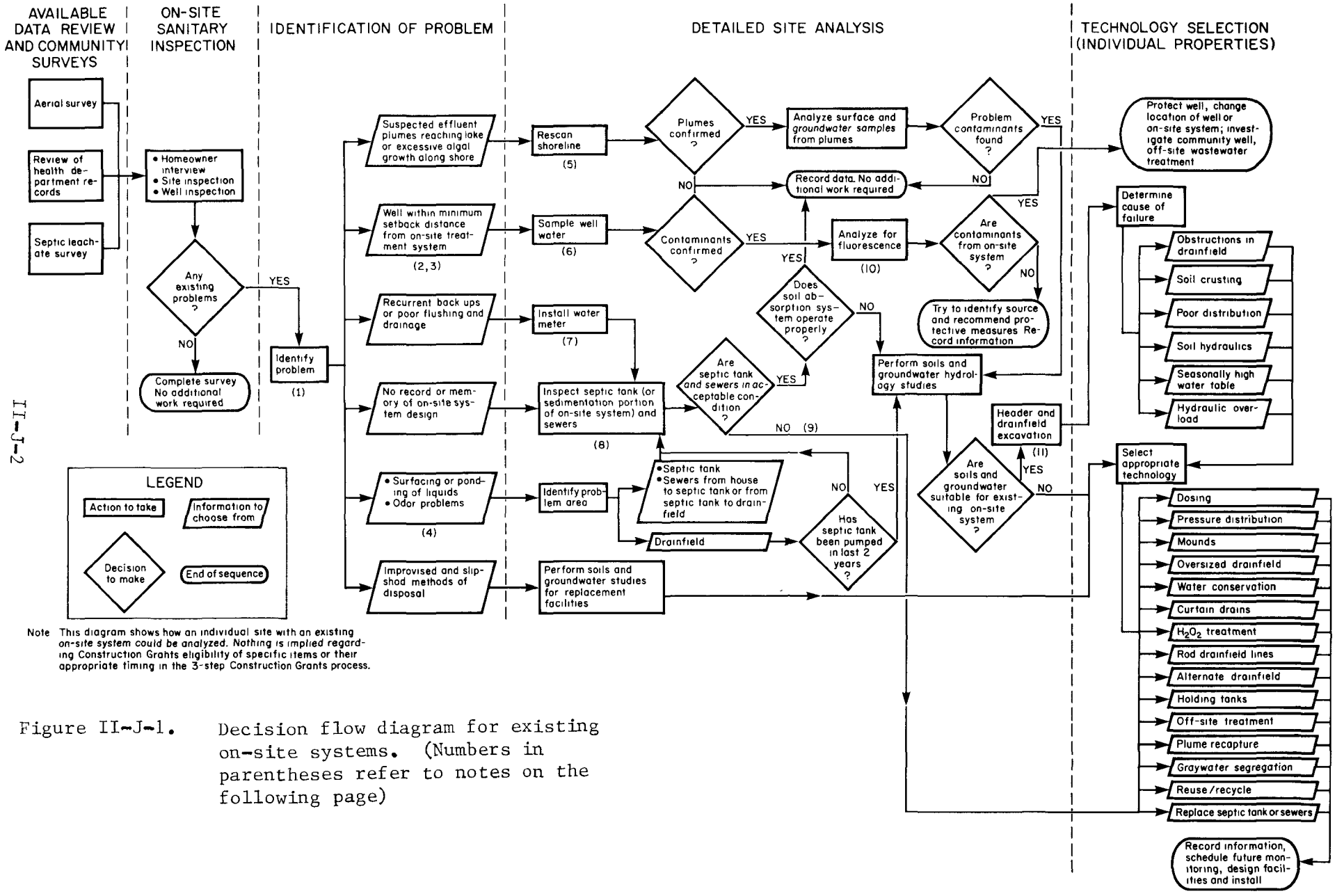
In communities served by on-site wastewater treatment systems, problems with existing systems must be identified. A systematic approach to isolating problem areas, identifying specific problems, and selecting the appropriate technology to solve the problem is presented in Figure II-J-1. The Decision Flow Diagram is divided into the following sections: 1) Phase I Information Gathering, 2) Sanitary Survey, 3) Definition of Problem, 4) Site Specific Analysis, and 5) Technology Selection.

Phase I Information Gathering includes reviewing existing or easily obtainable data (for more detail see Appendix A, Region V Guidance - Site Specific Needs Determination and Alternative Planning For Unsewered Areas). Data from Phase I can usually be obtained without going on-site and is useful for isolating potential problem areas.

The Sanitary Survey involves talking with individual homeowners about their on-site systems. The information generated is useful for defining problems with individual systems. After specific problems are defined the site specific analysis section of the decision flow diagram provides for various tests and inspections to find the source of the problem and to provide enough background information to select the most appropriate technology.

The Decision Flow Diagram provides for systems that do not meet current codes. For example, consider a septic tank that is found to be smaller than current requirements. If the septic tank is in satisfactory condition and operating properly, no additional work is recommended.

The Decision Flow Diagram cannot account for all situations that may be encountered in the field and should, therefore, be used as a guide along with common sense to determine the proper solution for on-site wastewater treatment system problems. The notes following Figure II-J-1 correspond to numbers listed on the decision flow diagram.



Note This diagram shows how an individual site with an existing on-site system could be analyzed. Nothing is implied regarding Construction Grants eligibility of specific items or their appropriate timing in the 3-step Construction Grants process.

Figure II-J-1. Decision flow diagram for existing on-site systems. (Numbers in parentheses refer to notes on the following page)

NOTES

1. If, through previous experience, the cause of the problem can be identified, the remaining steps can be bypassed by correcting the problem.

2. State standards for minimum setback distances should be used unless a hydrogeologic (or other) reason exists to use a larger distance.

3. In using contaminated wells as a criterion for delinating sewer service areas, only data from protected wells should be used.

4. Odors can come from a properly functioning septic tank/soil absorption system. Relocation of vent may solve the problem.

5. Shoreline scan should be repeated to ensure that plumes are found.

6. Well samples should be taken several times at various intervals to ensure accurate results.

7. If house drains are likely to be clogged, snaking drains may solve problem. Note: monitoring of watermeter is required after installation.

8. Septic tank and sewer inspection to include: excavation; pumping; inspection for size, structural integrity, outlet and baffle condition; rodding house and effluent sewers; measuring distance and direction to SAS using snake.

9. If septic tank and/or sewers (to and from septic tank) need replacement, and additional work on drainfield is required, follow "no" route and investigate other factors before replacing septic tank and/or sewers. This process will avoid replacement of septic tank/sewers when entire system is not functional.

K. GEOTECHNICAL INVESTIGATIONS FOR CLUSTER DRAINFIELDS

1. INTRODUCTION

Soil characteristics and surface and groundwater hydrology are the most important factors influencing the feasibility and performance of cluster drainfields. These characteristics must be evaluated throughout the site selection and evaluation process in order to arrive at a reasonable estimate of the feasibility and appropriate design criteria for a particular site. Soil cannot be tested or evaluated as can other engineering materials, such as steel, because it varies foot by foot both horizontally and vertically. The designer must conduct his evaluation and selection of design criteria with this attitude in mind. Simplifying assumptions must be made, average values selected, and limitations of data gathering, evaluation, and design criteria selection techniques kept in mind throughout these investigations.

In the analysis of soil characteristics and surface and groundwater hydrology, certain values should be estimated and quantified by field and laboratory testing when necessary. These values would be included in the following categories:

- Horizontal permeabilities - the rates for each distinctive soil layer at which water will move laterally from the drainfield
- Vertical permeabilities - the rates for each distinctive soil layer at which water will move downward from the drainfield
- Depth to water table - the distance below the ground surface at which free water is encountered; also a measure of the allowable rise in the water table that causes horizontal movement of the groundwater below the drainfield
- Depth to aquitard - the distance below the ground surface at which a layer is encountered that allows only a minimal passage of water downward; also a measure of the thickness of permeable material through which horizontal flow will occur
- Slope of the water table - the gradient of the pressure which indicates flow direction and when analyzed with the thickness of the saturated material and its hydraulic conductivity provides a measure of the present horizontal flow rate
- Slope of the aquitard - the slope of the underlying material which may determine the slope of the water table and groundwater flow direction.

In addition to estimates of criteria for the specific site itself, estimates of adjacent areas should be developed also to ascertain the effect of the applied wastewater.

2. PHASES OF THE GEOTECHNICAL INVESTIGATION

The investigations are carried out usually in three phases; the second and the third phases are dependent upon the first. The individual phases are outlined as follows:

a. Preliminary Site Investigation

The objective is to establish the general feasibility of the area for possible cluster drainfield sites. The primary source of information for the area assessment should be the detailed soil survey prepared jointly by the USDA Soil Conservation Service (USDA-SCS) and the state experiment station. These soil surveys can help to evaluate the suitability of a specific soil series for septic tank drainfields. It must be noted, however, that the survey's rating system is based on limitations. For example, although some soils may be rated "severe," workable systems may be designed but at a greater expense. Therefore, specific soil characteristics must be examined closely in order to arrive at suitable classifications for various drainfield designs.

Along with the soil survey information, topography, land use, and geology should be utilized also to optimize site selection. Topographic information is available from USGS quadrangles. More detailed information may be available from the USGS, local planning agency, community, or landowners. Land use information can be obtained from aerial photographs or from a windshield survey. Geological information can be obtained from special USGS reports, state or local reports, and well logs available from the local or state permitting agency or state USGS offices. Finally, landowners' knowledge of their property, often extensive in rural areas but also often ridden with gross misconceptions, should not be overlooked.

From the preliminary analyses of soils, topography, land use, and geology, a general site survey can be conducted. Cross-sections should be drawn through areas of greatest interest so that materials below the ground surface can be visualized and estimates can be made for the criteria enumerated in the preceding discussion. Then, the most promising sites could be investigated further.

At this point, estimates should be made of the type of drainfield that could function properly. The preliminary design must account for the mounding effect, a horizontal movement of water from under the drainfield. Sites with a limited depth to an aquitard or groundwater or low horizontal permeability may require a long, narrow drainfield rather than a square drainfield. A cluster drainfield may function on a particular site only if some means of artificial drainage is provided. Other designs might be considered also.

The most feasible sites and preliminary designs would be included in the cost-effective analysis and compared with other alternatives for wastewater treatment. If the cluster drainfield alternative is viable, the second phase of the geotechnical investigation would be implemented to determine critical soil and groundwater values.

b. Field Testing Program

Following the selection of the most suitable site(s), more detailed information may be necessary for preliminary design criteria and for assessment of the impact of the cluster drainfield on the environment. Groundwater mounding methods and flow effects should be selected prior to field investigations. The values necessary for the selected methodology can then be designed into the field investigation program.

The field investigation program should be designed to realistically account for the non-homogeneity of the soil mass and should neither grossly under- or over-estimate soil values and groundwater conditions. Thus, sampling methods that encompass the greatest soil mass would probably generate the most reasonable values. The most inexpensive and convenient testing procedures generally involve very small soil samples, and are, therefore, least representative of the actual soil body since a multitude of tests are not included (Reeve, 1965).

Where a detailed soil survey is not available or where the soil survey indicates considerable variability within a mapping unit, the contiguous lands and the land on the site should be mapped by a competent soil scientist familiar with the area. The USDA Soil Conservation Service will, cost and scheduling permitting, perform the mapping for requesting landowners, or will provide names of individuals who can provide the service if they cannot. The work is normally mapped on aerial photographs at a scale of 4 inches = 1 mile. If aerial photographs of a different scale were to be provided, for example 1 inch = 400 feet, they would be more useful, especially if the soil boundaries were variable.

Ideally, the soil borings and testing would be conducted by a firm that performs borings for structural purposes and for aquifer testing for water well purposes. Soil test firms generally collect small soil samples, visually classify soil material, and install piezometers. Well drilling firms generally visually classify soil material and conduct field tests on the saturated material. Frequently, the soil investigation program is limited by the equipment and experience of the drilling firms in the area.

The number and spacing of borings on a particular site would depend upon the complexity of the soils, the margin of suitability , and the contributing flow. While the actual number of borings may range from one to ten, three would usually suffice for most sites. At least one of the borings should be conducted with continuous split spoon sampling to identify any layers of limiting vertical permeability that would function as an aquitard (permeability less than the application rate). The soil material throughout the boring should be visually classified and blow counts kept on the split spoon sampling. The boring should be continued well down into the permeable material below the water table so that the area contributing to horizontal flow can be defined.

Piezometers should be installed in the borings in order to define the water table. Plastic well or piezometer screens perform best, along with plastic (PVC or CPVC) casings. The piezometers should be placed in both the upper and lower aquifers if a well-defined aquitard is present. The flow situation would be considerably different if water were moving upward from the

lower aquifer than if it were moving downward from the upper aquifer. Different water levels in the piezometers would define the direction of movement. The elevations of the water table in the piezometers should be established by a level survey to within an accuracy of 0.1 feet difference between the piezometers. Also, the elevation of surface water bodies in the area should be determined by the level survey or by topographic maps. The slope of the aquitard can be estimated from the borings and from level survey information.

Piezometers can be pumped out and utilized for obtaining water samples, and should be pumped until clear water is obtained and a sample taken. The sample should be tested primarily for nitrates and phosphorus, the nutrients of greatest concern. A complete chemical analysis should be conducted where groundwater chemistry is an identified concern.

Numerous testing procedures, which have been developed for determination of soil hydraulic properties, vary widely in equipment needs, bulk of soil tested, testing accuracy and reproducibility, and time required for testing (Luthin, 1957; Kirkham, 1965; U.S. EPA, 1977). A list of some of these procedures is provided in Tables 1 and 2. A soil test that determines the properties of a small volume of soil must be extrapolated to the larger soil mass. The approximations involved in doing that do not warrant extensive and expensive tests for accurate results. Certain tests are designed for specific soil materials and would not work in soil materials of greatly different textural classes. Also, the soil testing procedures are generally classified into above-the-water-table and below-the-water-table conditions. Some tests are applicable only where a considerable depth of saturated permeable material is located.

c. Analysis of Groundwater Mounding and Discharge

In order to assess whether an adequate distance between the water table and the bottom of the drainfield would exist under loading conditions, a method of predicting the water table rise, or conversely, of selecting design criteria for the drainfield, is necessary. Several means for conducting the analysis are available. Probably the most convenient, and the most conservative, is the Hooghoudt or elipse equation for two-dimensional flow. It can be derived for a level aquitard and a sloping aquitard and its derivation requires the assumption of an outflow ditch or drain tile. Several methods that model simple groundwater systems are available, including some with graphical and computer-assisted solutions (Spiegel, 1975; Bianchi and Muckel, 1970; U.S. EPA, 1977). Generally, the estimation of hydraulic conductivity entails considerable error and rigorous solutions are not justified. The modeling analysis should be evaluated against design criteria utilized and modeling techniques employed until a functioning cluster drainfield is developed. The cost analysis for this drainfield design can then be compared with other alternatives for sewage treatment.

Frequently overlooked in the analysis of the cluster drainfield is the effect it would have on the groundwater discharge locations and flow rate. In most areas the change may be negligible; nevertheless, an analysis of the impact of drainfield on groundwater discharge should be conducted if only in a qualitative manner. Occasionally mitigative measures must be employed to minimize the impact of greater groundwater discharge or of altered discharge locations.

Artificial drainage such as drainage wells or draintile may be necessary for a cluster drainfield to function properly. Draintile is preferred if a gravity outlet is available. In some situations, intercepting natural groundwater would be necessary to ensure satisfactory functioning of a cluster drainfield. Thus, potential sites should not be eliminated from consideration due to depth to groundwater or aquitard limitations.

3. Example Site Analyses

Three hypothetical sites, and boring programs for each site, are described herein. The first site is a 50-acre outward terrace with an elevation about 30 feet above an adjacent lake. The USDA-SCS identifies the soils on the site as nearly uniformly medium sands with a permeability of 6-20 inches per hour. Soil properties that cannot be estimated are depth of medium sands and potential of aquitards. The depth to groundwater and its flow direction (slope) could be defined also. One soil boring would suffice for the site, assuming that the medium sands are deep and the water table elevation is slightly above the lake elevation. The boring should be performed with a hollow-core auger and continuous split spoon sampling. The Hooghoudt equation for two-dimensional flow could show conservatively that the mounding effect would be no problem.

The second site is similar to the first but the soils have a lower permeability rating by USDA-SCS standards, two-six inches per hour. Wells from the area indicate variable depths to an aquitard, possibly discontinuous. The elevation difference between the land surface and the lake is about 25 feet and the distance to the lake is about one-fourth of a mile. The boring program would consist of three borings spaced equilaterally on the site extending to a depth 20 feet below the water table, or five feet into an aquitard. Continuous split spoon sampling would be conducted in one boring, and periodic split spoon samples taken at perceived soil texture changes in the other two borings. A limited number of double-tube tests for horizontal and vertical permability would be conducted below the expected bottom of the drainfield. At the piezometer location most suitable for a drainfield, a four-well permeability test would be conducted. Three additional peizometers would be installed for the test. The data from these tests would then be interpreted and an analysis of the mounding effect conducted. Different configurations of drainfields would be investigated until a satisfactory arrangement is identified.

A third site has three soil series mapped by the USDA-SCS, and soil textures range from silt loams to sands and gravels in apparently random patterns. One mapping unit is a complex of the other two mapping units. Within 600 feet on opposite sides of the site area small ponds. The elevation difference between the ponds is five feet, and the ground surface on the site is about ten feet above the lower pond water surface. First, the field investigation should consist of a more detailed soil mapping on a 1":400' scale aerial photograph performed by a locally qualified soil scientist. A boring program can be devised from the detailed mapping. Approximately ten borings and piezometers may be necessary to define the soil material and the water table surface. Double-tube permeability measurements could be conducted in the various soil layers above the water table as the borings are conducted. Below the water table the four-well or, if the saturated sand and gravels are of sufficient thickness, aquifer and recovery tests or a slug test could be

conducted. The effect of septic tank effluent application to the drainfield would probably indicate the need for artificial drainage. The analysis should concentrate on the feasibility of providing drainage and on the drainfield configuration. The irrigation test may be an alternative for evaluating limiting vertical permeabilities and effective horizontal permeability.

TABLE II-K-1. TESTING METHODS FOR DETERMINING HYDRAULIC CONDUCTIVITY OF SOILS GENERALLY APPLICABLE AT OR ABOVE THE WATER TABLE

Method	References	Field equipment	Laboratory tests	Office analysis	Limitation of method	Quality of data
Irrigation loading	Leach et al 1980	Piezometers, irrigation equipment	None	Interpretation of results and calculation of conductivity	Expense of test and length of test time; availability of water	Excellent for nearly all soil materials for both vertical and horizontal permeabilities
Auger hole	Boersma 1976a	Augered hole, bailer or pump, water level measuring device	None	Calculation of conductivity	Water table must be near the surface; small soil volume tested	Excellent if considerable tests are conducted for average horizontal permeability
Piezometer	Boersma 1976a Schmid 1967	Augered and cased hole, bailer, water level measuring device	None	Calculation of conductivity	Water table must be reasonably close to the surface; small soil volume tested	Excellent if considerable tests are conducted for horizontal permeability of individual layers
Shallow well pump-in	Boersma 1976b	Cased or uncased hole, water level maintaining device	None	Calculation of conductivity	Small soil volume tested; potential for smeared sidewalls; complexity of equipment; large quantities of water needed; length of test	Good if number of tests are conducted; horizontal permeability determined
Double-tube	Boersma 1976b Bouwer 1964	Augered hole, Tempe double-tube device	None	Calculation of conductivity	Small soil volume tested; complexity of equipment; no water table present	Excellent if number of tests are conducted; both horizontal and vertical permeability of a specific layer determined
Cylinder permeameter (Crust permeameter)	Boersma 1976b	Excavated hole, tensionmeters, cylinder, water level maintaining and flow rate measuring device	None	Calculation of conductivity	Small soil volume tested; complexity of equipment; shallow	Good if number of tests are conducted; generally for vertical permeability of restricting layers
Air permeameter	Grover 1955 Kirkham and Adams 1956 Kirkham and Grover 1960	Excavated hole, air pressure and application apparatus	None	Calculation of conductivity	Small soil volume tested; complexity of equipment; shallow depths only	Good if number of tests are conducted; vertical permeability measured
Four well	Snell and van Schilfgaarde 1964	Piezometers, pump, water level and flow rate measuring devices	None	Caluclation of conductivity	Water table or aquitard must be present; length of test; complexity of test	Excellent for large soil mass for horizontal permeability

(Continued)

TABLE II-K-1. TESTING METHODS FOR DETERMINING HYDRAULIC CONDUCTIVITY OF SOILS GENERALLY APPLICABLE AT OR ABOVE THE WATER TABLE

Method	References	Field equipment	Laboratory tests	Office analysis	Limitation of method	Quality of data
Permeameter (constant or falling head)	Klute 1965	Equipment to obtain soil samples	Permeameter	Calculation of conductivity	Extremely small soil sample; difficulty in obtaining undisturbed samples	Satisfactory if numbers of samples are tested and correct interpretations made; measures either horizontal or vertical permeability
Grain-size distribution	Davis and DeWeist 1966	Equipment to obtain soil samples	Sieve and hydrometer analysis	Calculation of conductivity	Extremely small soil sample; completely disturbed condition	Satisfactory if numbers of samples are tested, and soil material is structureless and unstratified

TABLE II-K-2. TESTING METHODS FOR DETERMINING HYDRAULIC CONDUCTIVITY OF SOILS GENERALLY APPLICABLE BELOW THE WATER TABLE

Method	References	Field equipment	Laboratory tests	Office analysis	Limitation of method	Quality of data
Pump-down aquifer test well	Johnson Div., UOP 1972	Well, pump, observataion wells, flow rate measuring device; water	None	Interpretation of data and calculation of conductivity	Complexity of field setup; length of test; usually for permeable materials	Excellent for deeper, permeable materials and horizontal permeability
Recovery aquifer test well	Johnson Div., UOP 1972; Bouwer 1970	Well, pump, water level measuring device	None	Interpretation of data and calculation of conductivity	Complexity of field setup; length of test; usually for permeable materials	Excellent for horizontal permeability in deeper, permeable materials
Slug test	Bouwer and Rice 1976	Well, pressure transducers, strip chart recorder, bailer or volumetric weight	None	Calculation of conductivity	Small soil volume samples; unfamiliarity, complexity of equipment	Good for permeable material and existing wells; rapid
Draintile outflow	Skaggs 1976 Hoffman and Schwab 1964	Draintile in place, piezometers, flow rate and water level measuring devices	None	Calculation of conductivity	Water table near the surface for draintile to function; length of test; complexity of equipment	Excellent for averaging large site, and for horizontal permeability

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L. IMPACTS OF WATER CONSERVATION ON ALTERNATIVE TECHNOLOGIES

1. INTRODUCTION

Flow conservation in individual residences directly impacts on-site wastewater treatment and disposal systems. However, the nature and extent of these impacts is not yet well defined. Existing literature on the subject is scarce, although ongoing research promises to fill in some of these gaps. One reason for the lack of information is that until the late 1960's and early 1970's, not much attention had been given to flow conservation because of lack of incentive. With the benefits now realized, such as lower costs for water supply and heating, more attention will be given to the effects on wastewater treatment and disposal.

2. EXISTING LITERATURE

Existing literature indicates that favorable impacts for on-site systems can be expected from flow conservation. Baker (1980) takes a total system approach in performing a cost analysis for households with and without flow reduction in the form of air-assisted toilets (0.5 gal/flush) and air-assisted showers (0.5 gpm). For a family of four, the reduced flow is assumed to allow a reduction in drainfield area from 750 ft^2 to 280 ft^2. This result is based on a design infiltration rate of 0.6 gpd/ft^2 and not on local codes that may not allow such a reduction. With the reduction of drainfield area and the reduction of energy required for shower water as the major factors, Baker shows a household with flow reduction devices to have a lower annual system cost than a household without flow reduction. The effect anticipated, therefore, is beneficial in terms of annual cost of the overall system and reduction of the size (or the extension of the life) of the drainfield.

Bennett (1975) indicates that flow reductions of 40% would reduce the cost of an average septic tank/soil absorption system by about 23%. The implication of the lower costs is that not as much treatment is required for a given system with reduced flows. If reduced flows result in smaller soil absorption systems for acceptable soils, perhaps flow reduction can be used in conjunction with marginal soils to make them acceptable for on-site disposal, or with failed systems for rehabilitation.

3. RESEARCH

The possibilities of rejuvenation are the subject of two research projects currently underway. Purdue University (Hudkins, 1980, personal communication) is investigating a number of on-site systems in Indiana that have failed and will be recommending flow conservation options or alternative treatment system options depending on the specific site conditions. Results from this study are not expected until the latter part of 1981. The Pennsylvania State University (Fritton, 1980, personal communication) is involved in a similar research project that includes monitoring water consumption and soil absorption system operations prior to implementation of a rehabilitation strategy. The systems will continue to be monitored after installation of flow conservation devices or alternate drainfields so that the effects of flow reduction and drainfield resting can be determined. Results from this project should be available in October 1981.

4. POSSIBLE EFFECTS ON SEPTIC TANKS

Water conservation will have some obvious impacts on the septic tank. One of these impacts will be an increase in the detention time of the wastewater. Theoretically, this increased detention time will allow better settling of solids and possibly improved digestion of pollutants. Flow reduction devices also reduce somewhat the hydraulic surge loads that conventional fixtures place on a septic tank. Smaller surges should reduce the mixing within the septic tank, thus reducing the possibility of solids carry-over, especially when the tank is ready for pumping.

For existing septic tanks not meeting current codes for size (smaller than standard size), the effect of flow reduction may be to increase the detention time to a point comparable to a standard size tank without flow reduction. Thus, savings could be realized by not having to replace an undersized tank. Undersized tanks would then have the potential for adequate treatment, while properly sized tanks could provide a somewhat better effluent.

Another effect of reduced flows is an overall increase in temperature of the wastewater. An increase in temperature is particularly evident when toilet flushing is reduced significantly. Table II-L-1 (Baker, 1980) shows flow rates with conventional and flow-reduction fixtures along with the expected temperatures for each use.

TABLE II-L-1. TYPICAL FLOW RATES AND TEMPERATURES FOR HOUSEHOLD WATER USES

Use	Conventional flow (gpd)	Reduced flow (gpd)	Temperature (F°)
Toilet	100	10	Ambient
Bathing	80	10	105
Laundry	35	35	105
Dishwasher			140
Kitchen (including dishwasher)	27	27	105
Lavatory	8	8	105
Utility	5	5	105
TOTAL	255	95	

The figures were based on a family of four with air-assisted showers and air-assisted toilets used to obtain the reduced flows. Figure II-L-1 shows the temperature of the wastewater as a function of ambient water temperature for conventional flows (255 gpd) as well as for reduced flows (95 gpd). The increase in temperature could theoretically mean an increase in biological activity within the septic tank. A higher digestion rate in the tank could

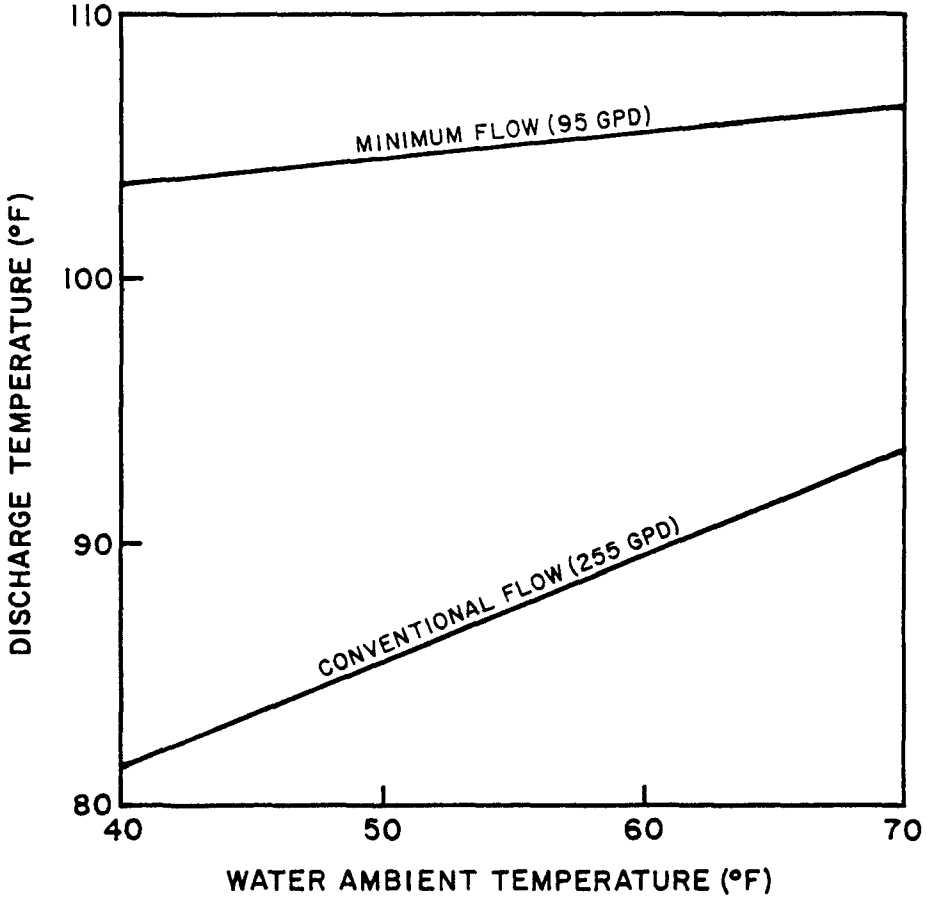

Figure II-L-1. Wastewater temperature vs. ambient water temperature.

mean less frequent pumping. However, this hypothesis must be tested along with testing for detrimental effects such as poorer settleability as a result of increased temperatures and increased gas generation.

Although increased detention time and temperature appear to be obvious effects of flow conservation on septic tanks, the impacts on treatment have not been scientifically proven. Similarly, soil absorption systems are known to receive less wastewater, but the impact on the quality of the effluent or the life of the system has also not been proven.

5. POSSIBLE EFFECTS ON SOIL ABSORPTION SYSTEMS

The most obvious effect of flow reduction is the decrease in hydraulic load to the soil absorption system. Because systems are sized on the basis of hydraulic load, a reduction in field size might be appropriate. A reduction in field size would be a tangible indicator for cost reductions of on-site systems.

Some states, such as Minnesota, allow a reduction in the size of the soil absorption system when wastewater flow is reduced (Machmeier, 1979). Soil treatment area can be reduced by about the same percentage as the reduction in flow. Minnesota requires that the amount of sewage applied to the soil be measured through the installation of a water meter or a cycle counter for well pumps. Many regulatory authorities do not allow a reduction in field size because they question the permanence of the flow-reduction devices or because they do not see the associated cost savings as being significant.

Another way of looking at the benefits woul be to consider the life of a soil absorption system in terms of total number of gallons treated. If a soil absorption system treats 255 gpd for 20 years, the total number of gallons treated would be 1,861,500 gallons. A flow-reduction system that handles 95 gpd would require a system life of 54 years in order to treat 1,861,500 gallons. This linear extrapolation is only an approach to assign economic benefits for soil absorption systems receiving reduced flows. No empirical or proven theoretical relationship exists relating hydraulic or organic loading to the useful life of a soil treatment system.

Organic solids loading to a soil absorption system may be low because of the increased detention time in the septic tank. On the other hand, the solids loading may remain the same because the total amount of pollutants entering the system will not be altered by reducing flows. Pilot studies or research, as mentioned earlier in this section, would help answer questions relating to such topics as solids loading to soil absorption systems. If the quantity of total dissolved solids in the supply water is high, the reduction in water use would definitely lower the solids loading to the soil absorption system.

6. CONCLUSIONS AND RECOMMENDATION

Many unanswered questions exist regarding the impact of water conservation on on-site treatment and disposal systems. Beneficial impacts of flow conservation on on-site systems appear likely, but testing and additional research is needed to confirm existing hypotheses. The following objectives should be considered as part of future research on flow conservation effects on on-site treatment systems in order to assess the feasibility and economic impact of flow reduction:

- Effects on soil absorption systems (size, loading, effective life),
- Effects on septic tanks (increased detention time, increased temperature, reduced surge loads, reduced mixing),
- Need for metering,
- Applicability to marginal soils,
- Applicability to failed systems,
- Effects on on-site systems not soil dependent (fixed film, package aeration, etc.), and
- Effects on overall system costs and effective life.

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CHAPTER III

USE OF SOILS DATA

A. SOILS RELATIONSHIPS STUDY

1. INTRODUCTION

Soil characteristics can be used to explain the performance of existing on-site sewage disposal systems and to predict the performance of new and future systems. Explanations and predictions may be based either on hypothetical relationships or locally verified relationships between soil characteristics and system performance. For planning-level decisions on developed and undeveloped properties and for site-specific decisions on undeveloped properties, the use of hypothetical relationships would be appropriate. For site-specific decisions relevant to existing private sewage disposal systems, however, the use of actual performance data and on-site soils information is imperative in order to maximize the service life of rural wastewater management systems and to optimize potential cost-effectiveness gains of these systems in comparison to sewers or other off-site facilities.

The benefits and limitations of applying soils data, based upon hypothetical relationships between on-site system performance and soil characteristics will be examined subsequently within the context of soil surveys and soil potential studies. Soil surveys are valuable wastewater management planning tools, containing information that is particularly useful in making preliminary estimates of new system feasibility and in designing field investigation programs. For site specific decisions, however, soil survey data, particularly soil limitation ratings, may preclude on-site sewage treatment techniques that are feasible and cost-effective. In effect, soil surveys recognize site limitations including low soil permeability and shallow depth to seasonal high groundwater as problems to avoid.

The use of soil potential ratings refine on-site wastewater management planning by recognizing the feasibility and cost-effectiveness of small waste flows techniques that may be used to overcome limiting site characteristics. This recognition, incorporated into soil potential indices, represents a departure from soil limitation ratings. Like the soil survey, however, soil potential data do not provide site-specific information that is essential to detailed decision making. Soil potential data effectively recognize site limitations as problems which can be overcome.

Empirical soils data are gathered, assessed, and retained in recognition of the fact that detailed wastewater management decisions must be based upon locally verified relationships, not hypothetical relationships, between on-site system performance and soil characteristics. The empirical approach recognizes that actual system performance does not always correlate well with soil limitation ratings. It further recognizes that system performance is not only influenced by local soil characteristics but also by other factors including system design, construction practices, usage, condition, and maintenance. In favorable soils, sub-optimum conditions in these other factors will not usually result in on-site system failures. In marginal soils, sub-optimum conditions will need to be improved in order to avoid malfunctions. In unfavorable soils, these conditions will need to be markedly improved in order to sustain efficient on-site sewage treatment. Some soils are so un-

favorable that no technique will be feasible and cost-effective. It is noted, however, that the empirical approach does not automatically recognize a soil limitation as a problem. Instead, it serves to probe reasons for system malfunctions, thereby suggesting corrective measures for existing systems that have failed and new or future systems that have the potential to fail under similar conditions.

2. USE OF SOIL SURVEYS IN RURAL WASTEWATER FACILITIES PLANNING

Since their first application to urban land use planning in the early 1960's, soil surveys have proven useful to public officials in the general siting of septic tank absorption fields. As basic land resource inventories, soil surveys provide information that cannot be obtained readily elsewhere. However, limitations inherent in their preparation and interpretation restrict the extent to which soil survey data, particularly soil performance ratings, may be applied in the planning and designing of on-site wastewater management systems. The ratings of soils for subsurface wastewater absorption, as presented in soil surveys, represent an assumed relationship between the performance of soil absorption systems and the soil characteristics upon which system performance depends. In many cases, this relationship is assumed to be valid. However, the relationship is not valid in enough cases to make it absolute and render the soil suitability ratings firm. Therefore, the use of soil surveys is optimized in the general planning and siting, rather than the detailed planning and design of on-site systems.

a. The Recent* Soil Survey

Soil surveys are prepared by the USDA Soil Conservation Service in cooperation with state agricultural experiment stations, and contain all soil maps of the survey area, descriptions of the soils, and interpretations for the use and management of the soils.

With the exception of Illinois, soil surveys are available for most counties in the midwestern states (see Table III-A-1). Current plans of the Soil Conservation Service call for completion of all new county soil surveys by 1997 (by telephone, Lloyd Wright, SCS, USDA, Washington, D.C., 13 January 1981).

TABLE III-A-1. APPROXIMATE PERCENTAGE OF COUNTIES BY STATE IN REGION V WITH COMPLETED SOIL SURVEYS AS OF 1978

Region V State	Percentage of Counties Surveyed
Illinois	30
Indiana	83
Michigan	83
Minnesota	56
Ohio	72
Wisconsin	71

Source: USDA SCS 1978.

* Assumed to address non-agricultural land uses (e.g., septic tank absorption fields).

Olson (1964) stated that the objectives of a soil survey are to:

- determine the important characteristics of soils;
- classify soils into defined types and other classifiable units;
- establish and plot on maps the boundaries among different kinds of soil;
- correlate and predict the adaptability of soils to various uses.

The soil survey is most effectively employed in reviewing the suitability of large land areas, such as plots and subdivisions, for wastewater disposal. The suitability of individual lots for sewage disposal is best determined through on-site evaluation rather than through use of the soil survey alone for reasons discussed later in this section.

Standard data contained in recent soil surveys that are important in small wastewater flows planning include land form, relief and drainage; parent material; soil profile characteristics; depth to groundwater; and depth to impermeable layers.

Both general and detailed soil maps are included in soil surveys. General soil maps are prepared in the field at a scale of one inch to the mile or less, with map units consisting usually of soil associations. At the community planning stage, a generalized soils map is essential for determining the potential suitability of septic tank-soil absorption systems (USEPA, 1980). Detailed maps, which delineate types or phases of soil series, soil complexes, and undifferentiated units, are usually published at a scale of 3.17 (approximately 4) inches to the mile. Detailed soil maps are superimposed on aerial photographs to show the location of various land uses and natural terrain features (see Figure III-A-1). These maps enable one to assess soil permeabilities and soil purification capabilities in general terms. Mellen (1975) noted that the accuracy of soil survey maps vary, depending on the location, due to the soil complexity, detail of the examination, scale of the base map, and the skill and experience of the soil mapper.

In an effort to satisfy the fourth objective of soil surveys, soil scientists make behavior predictions on the basis of soil properties such as permeability, water table fluctuation, and slope to determine the capacity of soil to absorb sewage effluent. These behavior predictions are manifest in soil limitation ratings.

1) Soil Limitation Ratings

Over the past 16 years, soil survey interpretations for non-farm land uses have included a rating of soils by limitations. The classifications used in rating soil limitations, as described in the National Soils Handbook (USDA, SCS, 1978), are defined as "slight", "moderate", or "severe" (Rogoff, 1979):

- Slight limitations are those which present no more than minor problems for the intended use and which can be overcome easily

Figure III-A-1. Detailed soils map

- Moderate limitations for the intended use can be overcome or modified by special planning and design, or maintenance for satisfactory performance

- Severe limitations are difficult and costly to overcome, requiring major soil reclamation, special designs, or intensive maintenance.

Soil limitation ratings are not quantitative. They are indicators of the kind of problem and degree of difficulty that may be confronted in the design of an effective wastewater treatment and disposal system. Because soil mapping units are not always homogeneous, the limitation classification is a good guide to areas in which more detailed investigation may be necessary (#6). It is noted that limitation ratings for septic tank drainfields are based only on conventional bed or trench designs.

The soil characteristics used in deriving soil limitation ratings are listed in Table III-A-2. They are based on a drainfield whose bottom surface

TABLE III-A-2. SOIL LIMITATIONS RATINGS USED BY SCS FOR SEPTIC TANK/SOIL ABSORPTION FIELDS

Property	Limitations			Restrictive Feature
	Slight	Moderate	Severe	
USDA Texture	---	---	Ice	Permafrost
Flooding	None, Protected	Rate	Common	Floods
Depth to Bedrock, in.	>72	40-72	>40	Depth to Rock
Depth to Cemented Pan, in.	>72	40-72	>40	Depth to Cemented Pan
Depth to High Water Table, ft below ground	>6	4-6	>4	Ponding, Wetness
Permeability, in./hr				
24-60 in. layer	2.0-6.0	0.6-2.0	>0.6	Slow Perc. Rate
layers >24 in.	---	---	>6.0	Poor Filter
Slope, percent	0-8	8-15	>15	Slope
Fraction >3 in., percent by wt	>25	25-50	>50	Large Stones

Source: U.S. EPA, Region V

is located 2 ft. (0.6m) below the soil surface (U.S. EPA Region V). Each soil characteristic has the potential to limit the capabilities of a soil mapping unit to the severe classification. This represents a distinct disadvantage in light of small wastewater technologies available (see Table III-A-3) to overcome these and other limiting soil characteristics.

TABLE III-A-3. ALTERNATIVES TO STANDARD SEPTIC TANK-SOIL ABSORPTION SYSTEMS

	Aerobic system	Alternative fields	Enclosed system	Evapotrans-piration	Grey water system	Water conser-vation
Steep slopes		X	X	X		X
Shallow bedrock		X	X	X		X
Space limitations		X	X		X	X
High water table			X		X	X
Iron & managanese in water		X		X	X	X
Soil permeability		X		X	X	X
No permeability	X	X	X	X	X	X
Re-use water	X				X	X
Water limited			X		X	X
Lot size			X	X	X	X

Source: Brian Duff, 1977

The advantages and disadvantages associated with the use of soil limitation ratings in wastewater management planning are discussed below.

2) Advantages of Soil Survey Data

Some key advantages to using soil survey data in wastewater management planning are listed below:

- Soil surveys represent a valuable source of areawide soils information which, if understood and used properly, have multiple planning and engineering uses
- Detailed soil maps are used in the preliminary selection of sites for subsurface absorption systems

- Soil surveys provide developers with a means for determining soil use before they purchase land. In effect, this enables developers to choose areas where odds are with, not against, success (Olson, 1964)

- Soil surveys indicate location of soils with impermeable horizons through which water does not readily pass

- Detailed soil maps illustrate site conditions that can be expected during field investigation of potential small wastewater flows sites. Expected conditions help determine equipment and time requirements for conducting field investigations

- Soil survey information enables health officials and small waste flows design specialists to quickly determine courses of action in problem areas

- A soil survey provides a method to determine readily whether a proposed subdivision can be expected to pass the soils evaluation test and approximately what percentage of lots can be expected to be served satisfactorily by on-site sewage disposal systems (Clayton, 1975)

- The use of information in soil surveys makes it possible to overcome most of the limitations of the percolation test, thereby increasing test value, as illustrated below (Mellen, 1975):

  - The soil map will permit an assessment of site suitability or limitations for the different kinds of soils regardless of the time of year at which the request for determination is made.

  - Interpretations of the soil mapping unit will indicate whether or not the soil at a particular site is subject to an intermittent high water table. This will avoid the criticism of variability of the percolation tests, which results from the tests taken at different seasons of the year.

  - The suitability of a site can be based on the performance of the soil type at many locations instead of merely the 6 test holes on a site. Therefore, the confidence limits would be greatly improved.

  - Soil maps show areas that are subject to flooding or that are too steep for construction and layout of septic tank filter fields.

3) Limitations Soil Survey Data

Maximum use of soil survey information can be made only when there is full understanding of the limitations inherent in obtaining the data. Some of the limitations to its use in small wastewater flows planning are:

- All criteria used in the assignment of limitation ratings (slight, moderate and severe) are based on assumed relationships between soil characteristics and performance of standard septic tank systems. Local verification of these relationships is seldom obtained. Allowance is not made in the ratings for system designs that overcome adverse soil characteristics.

- Some early soil surveys were completed prior to the present knowledge of soils and do not reflect current concepts and interpretations. Soil interpretations for non-agricultural uses, including septic tank drainfields, have been included in soil surveys only within the past two decades.

- Due to scale limitations, areas of 2 to 3 acres are seldom represented on a detailed soils map and are instead described as inclusions in soil mapping units.

- Soil mapping standards require that 85% of a soil area must represent the range of properties defined by a soil name. Therefore, 15% of an area within a soil boundary may contain slightly different soils from the main body. However, the accuracy of the soil map units is commonly much less than 85%, and may average only 55% in glacial landscapes throughout the midwest (Rogoff, 1979).

- Boundaries between soil types are seldom absolute in nature. Instead, soils are a continuum on the landscape (see Figure III-A-2). This is important when delineating areas suitable for septic tank drainfields.

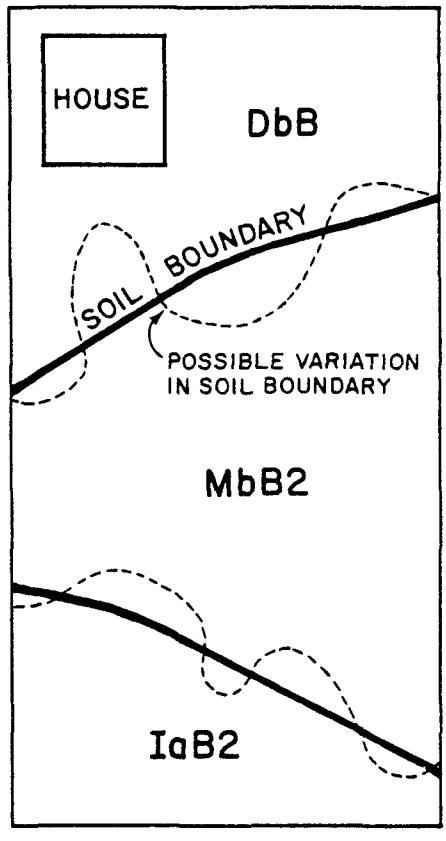

Figure III-A-2. Typical soil boundaries and variations in soil boundaries.

- Permeability data included or soil surveys are often based upon laboratory (column) analysis rather than field analysis (Bouma, 1974). Permeability rates so derived do not reflect the significant variations in groundwater movement known to occur in subsurface state.

- The 3-classification system of rating soils for subsurface absorption system suitability, as described above, focuses on the limitations rather than the potential of the soils in question. This is a marked disadvantage in the face of cost-effective small waste flows technologies available to overcome soil limitations.

- The single most limiting soil characteristic causes the soil to be placed in the most limited class.

- Soil interpretations included within soil surveys do not address when on-site systems fail. Therefore, the soil survey is not intended for use in resolving on-site system problems.

- The soil survey accepts the soil characteristics involving a seasonal high water table or slow percolation rates as limitations. Inexperienced users of the soil survey may accept the limitation as an insurmountable problem.

4) Summary

Soil surveys provide valuable land resource information at a scale of 2 to 3 acres or larger. They are particularly useful in making preliminary estimates of new use feasibility and planning site specific field work. However, in regard to making technology selections and sewering decisions, the cumulative effects of the listed disadvantages can often result in the rejection of techniques for wastewater treatment that are feasible and cost-effective.

3. USE OF SOIL POTENTIAL CONCEPT IN RURAL WASTEWATER FACILITIES PLANNING

In the early 1970's, it became clear to preparers and users of soil surveys that the employment of soil limitation ratings was an incomplete approach to predicting the behavior of on-site wastewater disposal systems (McCormack, 1974). The growing use of innovative and alternative small waste flows technologies, and increasingly available data on performance and cost of potential practices for overcoming soils limitations in specific soil types prompted development of the "soils potential concept." This concept takes a positive approach to soils interpretation by focusing on the quality of a soil after measures to correct limitations as related to other soils in an area have been applied. Innovative technologies employed to cope with high seasonal water tables and slow percolation rates can significantly increase the amount of land suitable for on-site sewage disposal (Rogoff, 1979).

The critical elements in the soils potential approach to small waste flows planning include the identification and recording of those practices which successfully overcome limitations, by soil type. While the soils potential concept incorporates local information about proven methods to overcome limitations, it does not eliminate the need for detailed field investigation of potential drainfield sites.

a. Soil Potential Ratings

Soil potential ratings are a form of soil interpretations which can assist wastewater management planners in their determination of the relative suitability of soils for on-site sewage treatment and disposal by (USDA, SCS 1978).

- Providing a common set of terms, applicable to all kinds of land use, for rating the quality of a soil for a particular use relative to other soils in the area.
- Identifying the corrective measures needed to overcome soil limitations and the degree to which the measures are feasible and effective.
- Enabling local preparation of soil interpretations, using local criteria to meet local needs.
- Providing information about soils that emphasizes feasibility of use rather than avoidance of problems.
- Assembling in one place information on soils, corrective measures, and the relative costs of corrective measures.
- Making soil surveys and related information more applicable and easily used in resource planning.
- Strengthening the resource planning effort through more effective communication of the information provided by soil surveys and properly relating that information to modern technologies.

Soil potential ratings are intended to be used in conjunction with other resource data (i.e., soil surveys) as a guide to making planning decisions. Application of soil potential ratings to general decision-making is not possible without previous identification of soil type. The soil potential rating or index for a particular soil is based on soil performance, cost of corrective measures, and costs related to continuing limitations. The soils potential index (SPI) is expressed as follows (USDA, SCS, 1978):

$$SPI = P - (CM + CL)$$

where

P = Index of performance or yield as a locally established standard.

CM = Index of costs of corrective measures to overcome or minimize the effects of soil limitations.

CL = Index of costs resulting from continuing limitations.

The definition of soil potential classes, which range from very high to very low, and the general concept of the SPI and procedures for preparing soil potential ratings are presented in Appendix A.

1) Limitations of Soil Potential Ratings

Although the soils potential concept represents a positive approach to wastewater management planning, several limitations to its use must be noted:

- Soil potential ratings are to be used with caution, as they are based on soil survey mapping. All the limitations due to scale and to soil variations within mapping units (see Section 2) are therefore extended into limitations in soils interpretation via potential ratings. These ratings do not reflect location, market trends, or social and political forces (Rogoff, 1979).

- Elements of the soil potential concept, including soil potential ratings, typical corrective measures, and typical continuing limitations are to be used only in general wastewater management planning and are not to be applied on a specific lot without on-site investigation for design and construction of the drainfield system (USDA, SCS Champaign Co., IL, 1979).

- Soil potential ratings are not interchangeable among counties. Application of ratings from one county to another must be preceded by field testing.

- Corrective measures to overcome soil limitations, and their attendant costs may become outdated quickly in light of a fast-paced state-of-the-art in small waste flows technology. Consequently, soils potential ratings must be updated to reflect increasing labor and material costs (Rogoff, 1979).

- Limitations within the soils potential context are viewed as problems, though to a lesser extent than within the soil survey.

- Soil potential ratings do not address why septic tank drainfields fail. Knowledge of problem causes is critical to the resolution of such problems.

- Soil potential ratings are more suited to interpreting soil suitability for future on-site systems than for existing systems. It is much more straight forward and conclusive to measure performance of existing systems directly.

- Soil potential studies are few in number. Available studies are not used extensively by sanitarians in the permitting of new or modification of existing systems.

2) Soils Potential Studies In Region V

A survey of the literature and of soil scientists throughout the midwestern U.S. indicates that while there is great interest in the soil potential concept as related to on-site sewage disposal, few soil potential studies have been completed in Region V. However, several soil potential initiatives within the region are scheduled for completion in the early 1980's.

Eaton County, Michigan

In his dissertation entitled A Computer Assisted Approach for Preparing Soil Potential for Urban Land Use Management, Rogoff (1979) developed a systematic procedure to numerically rate soil potential for five urban land uses, including septic tank drainfields in Windsor Township, Eaton County, Michigan. Soil limitations for septic tank system usage, as noted by the Eaton County soil survey, included slowly permeable soils, shallow depth to bedrock, and seasonal high water tables. Results of Rogoff's study indicated that of the 12,810 acres of land in Windsor Township that are rated as having severe limitations for on-site sewage disposal, over 11% or 2,857 acres may have good or fair soil potential, assuming the modification of conventional septic tank systems and construction of mound systems. This 2,857 acres represents a potential increase of 32% in the amount of land that is suitable for on-site wastewater disposal (Rogoff, 1979). On-site investigation of soil suitability will determine the actual amount of additional land that is available for development via decentralized systems.

Champaign County, Illinois

Soil potential ratings for septic tank absorption fields in Champaign County, Illinois were completed in 1979 by the USDA Soil Conservation Service. Evaluation factors by soil type that were considered in the preparation of soil potential indexes are listed below along with "standard" values[1] for these factors in Champaign County (USDA, SCS, 1979):

Evaluation Factor	Standard
Flooding potential	None
Depth to seasonal high water table	$\geq$ 48 inches
Percolation rate; (Permeability rate)	20 min./in. in upper 36 in. of soil; (0.6-2.0 in/hr)
Slope	$\geq$5%

The "standard" was also assumed in this study to include a three-bedroom residence with a private water supply, a 1,000 gallon septic tank with distribution box, and 750 square feet of drainfield.

Soil potential ratings for on-site systems were developed as a guide to land use planning only in Champaign County and were based on a consideration of 1) soil performance levels, 2) the difficulty or relative cost of corrective measures that would improve soil performance, and 3) any adverse environmental, economic and social impacts of soil limitations that cannot be feasibly overcome.

Soil potential ratings, in accounting for soil performance levels with respect to conventional septic tank systems in Champaign County, have been used successfully in the development of rehabilitative measures for existing

[1]"Standard" values for evaluation factors are associated with those soils have "slight" limitations for on-site sewage disposal.

systems that have failed (by telephone, Wyley Scott, USDA, SCS, Champaign County, IL, 2 Feb 81). Recommended corrective measures include the employment of aeration systems, mound systems, buried sand filters, and pressurized distribution systems. The amount of additional acreage that may be suitable for on-site sewage disposal as a result of modified conventional system and installation of alternative technologies has never been quantified, but it is likely to be significant in light of the excessive amount of system malfunctions and severely limited soils inventoried in the draft Champaign County soil survey (by telephone, Wells Andrews, USDA, SCS, Champaign Co. IL, 19 Dec 80; Wyley Scott, 2 Feb. 81).

It is noted that local sanitarians and geologists from the Illinois State Geological Survey involved in the Champaign County Study concluded that low density septic tank systems in sand and gravel soils do not cause pollution of groundwater resources (USDA, SCS, 1979). However, regular testing of wells in these soils should be conducted as a precautionary measure. This recommendation underscores the importance of field investigation as follow-up to general soil potential classifications.

Northeastern Illinois

Work on developing soil potential ratings for on-site sewage disposal has begun in the six-county area surrounding Chicago including Lake, McHenry, Kane, Cook, DuPage, and Will Counties. Efforts to date have consisted of meetings among multidisciplinary technical professional to organize the study. The format of the six-county soil potential report will be identical to that of Champaign County, Illinois (by telephone, Earl Voss, USDA SCS, Champaign Co. IL, 19 Dec 80).

State of Indiana

Small waste flows specialists in the On-Site Waste Disposal Program at Purdue University's Agronomy Department have undertaken a soil potential study in the Great Lakes Watershed of Indiana. A computerized approach is being developed so that soil potential information produced for counties in this watershed may be extended to other counties in Indiana. In addition, the On-Site Waste Disposal Program in concert with other midwestern states, will examine alternative ways of rating soils for septic tank systems (by telephone, Dr. Joseph Yahner, Purdue University, 2 Feb 81).

The objective of these studies is to compare field observed soil absorption system performance with system performance predicted in soil surveys. The degree of data correlation will be determined through field testing and laboratory analysis. Results of these studies have the potential to modify existing state regulations[2] regarding septic system construction in soils with "severe" limitations (by telephone, Stephen Hudkins, Purdue University, 23 Dec 80).

[2]Current state regulations prohibit the construction of septic tank systems on soils with "severe" ratings for such use as determined by soil surveys. It is estimated that over 70% of Indiana's soils have severe limitations or are unsuited for conventional septic tank-soil absorption systems. Unsuitable soil conditions include seasonal high water tables, slow soil permeability and slope by letter Stephen Hudkins, Purdue University to Alfred Krause, USEPA Region V, 25 June 1979).

3) Summary

The soil potential concept refines the wastewater management planning process, but like the soil survey, does not provide site-specific data that are essential to detailed decision-making. Particularly important among these data is information relevant to the causes of on-site system malfunctions. However, the main advantage of soil potential ratings is their recognition that some limitations can be overcome. Methods and costs of overcoming limitations are integral to the rating.

4. USE OF EMPIRICAL SOILS DATA IN RURAL WASTEWATER FACILITIES PLANNING

Many on-site sewage disposal systems installed in soils known to have severe limitations have continually served the wastewater management needs of rural communities without occurrence of system malfunction. The extent to which this apparent paradox exists today underscores the importance of incorporating site-specific soil data and empirical on-site system performance data, collectively termed "empirical soils data", into the wastewater management decision-making process. This phenomenom becomes less paradoxical when it is understood that the assignment of "severe" limitation to these soils was based upon an assumed or hypothetical relationship between soil characteristics and on-site system performance. Predictions of soil suitability for on-site sewage disposal based upon hypothetical relationships are appropriate for planning-level decisions relevant to existing and future systems. However, decisions regarding site-specific technology selection and system design must be based upon locally verified relationships between soil characteristics and on-site system performance. These decisions are possible through use of empirical soils data.

The gathering, assessment, and retention of empirical soils data, principally empirical performance data, are essential to evaluate causes of septic system failure, and more accurately predict future system performance than is possible through use of soil maps and soil ratings. Unlike soil surveys and soil potential ratings, performance data will indicate whether soil characteristics such as seasonal high water tables, slow percolation rates, shallow depth to bedrock, and slope are in fact determinative for efficient operation of on-site systems. Use of empirical soils data will enable local sanitation officials to address why systems are working in soils that are predicted to be unsuitable for on-site sewage disposal and, conversely, why systems are failing in soils predicted to have slight or moderate limitations for on-site disposal. This is not to suggest that there is a solution to every problem. Some soils may never be suitable for on-site systems despite attempts to apply alternative and innovative technologies to overcome site-specific limitations. The suggestion is made, however, that the use of empirical soils data has not been optimized in the past. Opportunity costs incurred by rural unsewered communities which have not made use of available empirical soils data are high in the face of increasingly costly centralized sewerage or other off-site facilities which may be proposed to meet their wastewater management needs.

The Illinois Environmental Protection Agency (1977) has noted the importance of performance data in its "Guidelines for the Preparation of Facilities Plans for Unsewered Communities":

"In general, the actual performance of properly designed and installed septic systems should be regarded as a more meaningful indicator of the soil's ability or lack of ability to support absorption systems than permeability or percolation ratings for the area."

It must not be inferred from this quote that soil permeability data are irrelevant to the wastewater management process. As previously discussed, these and other data including depth to bedrock and seasonal high groundwater help define the scope and resource requirements of efforts to locally verify the relationship between soil characteristics and the performance of septic tank systems. However, soil characteristics data by themselves do not accurately explain the performance of existing systems or predict the performance of future systems. Explanative or predictive accuracy is best achieved through correlation of these data with existing system performance information.

Historically, the costs of securing empirical data have been regarded as prohibitive by small rural communities. Additionally, soils scientists and sanitarians believe that the collection, assessment and retention of empirical soils data for future decision-making are resource intensive efforts better left to the "research and development" community. Officials cite time and personnel restraints and total commitments to "putting out fires" as impediments to actively developing an empirical soils data base.

In actuality, these data are collected continually by sanitarians and soil scientists during site-specific investigations of on-site system feasibility and system performance. Therefore, costs associated with obtaining this information are low. Assessment and tabulation of available soils and performance data for inclusion in an empirical data base could be performed during the winter months when field investigation efforts, and therefore, time and personnel constraints, are minimal.[3] The empirical data base should be developed in a way that will facilitate statistical correlation of soil characteristics, other determinants of performance and on-site system performance itself for use by local, regional, and state governments in Region V.

The economic justification for the collection and use of empirical soils data should be based on 1) a moderate to high level of community interest in on-site systems and 2) the fact that cost savings from optimum operation of existing systems far outweigh the costs associated with data collection. The optimum operation approach is based upon the collection and use of empirical information. It is frequently more cost-effective than construction of new centralized facilities which do not require this information.

The use of empirical soils data in detailed wastewater management planning (i.e. decision-making) in no way precludes the need to conduct on-site investigations prior to system design. The planning application of empirical soils data will determine if apparently limiting soil characteristics indeed inhibit effective on-site sewage treatment. By extension, these data will suggest what technologies may be appropriate to correct observed limitations and improve performance. However, they do not determine system sizing and cost, both which must be ascertained following careful investigation of individual lots.

[3]It is noted, however, that for quick turn-around, community-wide assessments of on-site system performance, as described in Section II.I, additional personnel may be needed.

a. Empirical Approach

The suggested approach to collecting, assessing, and using empirical soils data in small waste flows planning and management involves six stages:

1. Detection of individual malfunctions
2. Collection of data relevant to the causes of malfunctions
3. Assessment of likely causal relationships between data collected and malfunctions
4. Use of causally defined relationships in decision-making for malfunctioning systems
5. Use of causally defined relationships in decision-making for systems similiar to those that are malfunctioning
6. Development of regional or state data bases for increasing knowledge of on-site system capabilities and proven limitations.

Each of these steps is briefly described below.

Stage 1: Detection of Individual Malfunctions

The empirical approach begins with the detection of obvious problems associated with on-site system performance. These problems are manifested by:

- surface malfunctions (effluent rising to the surface)
- groundwater malfunctions (insufficient removal of wastewater constituents by the soil)
- back-up malfunction (sewage backing up in the system).

Methods that may be used to obtain direct evidence of the type and extent of individual malfunctions include (USEPA: "Region V Guidance," 1981):

- Direct observation
- Mailed questionnaires
- Survey of local sanitarians, zoning officials, or septage haulers
- Aerial photography/field investigation (surface malfunctions)
- Well inspection, and sampling and analyis of groundwater for whiteners, chlorides, nitrates, fecal coliform bacteria or other indicators

- Analysis of samples taken from septic tank effluents entering surface water bodies through soil.

During preparation of the Seven Rural Lake EISs, U.S. EPA used the following methods to detect individual on-site system problems:

- Remote sensing by aerial photographs was conducted by U.S. EPA's Environmental Photographic Interpretation Center (EPIC) to detect septic system failures noticeable at the soil surface. True color and color infrared film was used in the aerial survey.

Suspected surface malfunctions detected during inspection of the aerial photographs were subsequently field-checked and recorded. Very heavy tree cover can limit the effectiveness of this method (Krause, et. al. 1979).

Locations of the 42 confirmed[4] and 57 marginal[5] surface malfunctions detected by EPIC personnel in Salem Utility District No. 2, Kenosha County, Wisconsin are shown in Figure III-A-3. The combined number of system failures that were noticeable at the soil surface represented approximately 5% of the total on-site systems in the District.

- Septic leachate surveys were conducted along lake shorelines to detect the emergence of groundwater plumes from nearby septic tank drainfields. Detection of organic and inorganic chemicals associated with domestic wastewater, coliform bacteria, and elevated nutrient concentrations signaled the failure of septic tank systems beneath the soil surface. Such failures normally occur when the soil used in the drainfield has a percolation rate that is too rapid for effective renovation of wastewater and groundwater flows toward a lake shoreline.

- Sanitary survey of a representative number of residences to visually inspect lots and discuss home sewage disposal problems with residents, including sewage back-ups.

Stage 2: Collection of Data Relevant to Causes of Malfunctions

The empirical approach recognizes that several major factors such as permeability and depth to groundwater, in addition to soil characteristics, affect the performance of on-site systems. These factors include:

- System design
- System age
- System usage
- System maintenance

[4]Malfunctions were "confirmed" if standing wastewater or seepage from septic tanks was visible on the land surface during field inspection.
[5]Malfunctions were "marginal" if the accumulation of excess organic matter or the presence of dead vegetation indicated that septic tank systems had confirmed malfunctions in the past.

- Physical condition of structural system components (tanks, distribution boxes, pipes, etc.)
- Biological condition of drainfield (clogging and soil aeration)
- Surface drainage
- Groundwater hydrology

Collection of information relevant to these factors therefore constitutes a logical step in the small waste flows decision-making process during which anomalous system performance should be evaluated on a representative basis. Data on system design, age, usage, maintenance, and condition can be secured during partial sanitary surveys involving interviews with residents and inspection of properties.

In addition, information on local soil characteristics including texture, horizonation, depth, slope, and water table elevation should be obtained in this step for assessment in Stage 3. These data may be available from soil surveys, soil scientists, well drillers and local contractors, or may be obtained during representative sampling of developed properties.

Collection of data relevant to causes of on-site system malfunctions is briefly described in Section IV.C.2 of U.S. EPA's Region V Guidance: Site Specific Needs Determination and Alternative Planning for Unsewered Areas.

Stage 3: Assessing Likely Causal Relationships Between Data Collected and System Malfunctions

This stage involves an assessment of the data gathered in Stages 1 and 2. It specifically seeks to address why existing on-site system malfunctions occur. Results of Stage 3 will determine the nature of repair, replacement, and maintenance techniques required for malfunctioning systems in Stage 4, and for similiar but not yet failing systems in Stage 5. It is emphasized that efforts undertaken during Stage 3 serve to verify relationships between soil characteristics and on-site system performance. These relationships are essential to the small waste flows decision-making process.

An assessment of the likely causal relationships between data relevant to on-site system malfunctions and malfunctions themselves was undertaken by U.S. EPA during its preparation of seven EISs referenced earlier. Salem Utility District No. 2, Kenosha County, Wisconsin is used again as an example of how this assessment could be made.

The location of surface malfunctions in the District detected by aerial photography and subsequent field-checking is illustrated in Figure III-A-3. An initial assessment of the cause of these malfunctions may be performed by correlating information contained in Figure III-A-3 with three limiting soil conditions identified in the District based upon the Soil Survey for Kenosha and Racine Counties (Link and Demo, 1970):

1. Permeability less than 0.63 in/hr (permeability problems)

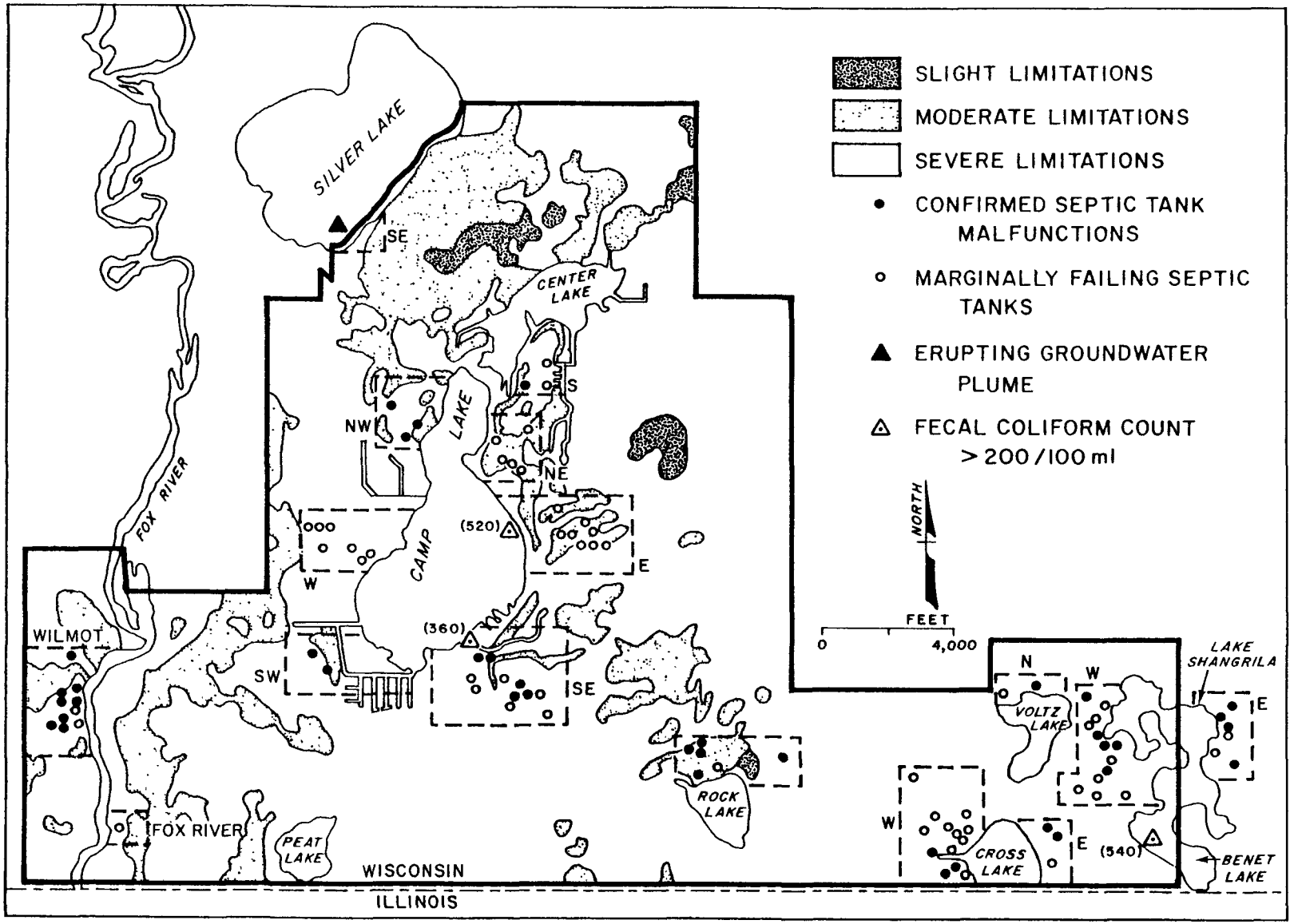

Figure III-A-3. Location of surface malfunctions as correlated with soil survey data in Salem Utility District No. 2, Kenosha County, Wisconsin.

Source: Link and Demo, 1970; U.S. EPA-EPIC, 1979

2. Depth to seasonal high groundwater less than 3 ft. (groundwater problems)
3. 1 and 2 combined

Figure III-A-4 illustrates this correlation, focusing on the most densely populated area in the District. It is an example of how causal relationships between available soils data and identified on-site system malfunctions can begin to be made. Most confirmed malfunctions in the vicinity of Center Lake, Camp Lake and Rock Lake are indicated in Figure III-A-4 to be associated with poorly drained soils, having permeability rates less than 0.63 inches/hour. Marginal malfunctions are indicated to be associated with both high groundwater levels and slow permeability rates. It is emphasized that correlations between available soil and malfunction data as illustrated in Figure III-A-4 are only first order assessments of problems and their causes. Additional information about on-site system design, age, usage, maintenance and condition, obtained during a sanitary survey, and detailed malfunction analysis (See Section II.I) is required by local officials before appropriate small waste flows technologies and management techniques may be selected to overcome existing and future problems.

The correlation between malfunction data (Figure III-A-3) and soil limitation ratings (slight, moderate and severe) illustrated in Figure III-A-4 does not define this causal relationship. In the absence of data suggesting the causes of septic tank system failure, the small waste flows specialist is unable to efficiently provide technological and management responses to wastewater management problems during the decision-making process.

Stage 4: Use of Causally Defined Relationships in Decision-Making for Malfunctioning On-Site Systems.

As previously indicated, locally verified relationships between on-site system performance and limiting soil characteristics are essential to the repair, replacement, and maintenance of existing systems that have failed. These relations, developed during Stages 1 through 3, can be used at varying phases of the small wasteflows decision-making process, including:

1. Estimation of the numbers and types or replace malfunctioning of appropriate technologies to upgrade on-site systems with an entire community (Note: this does not involve technology selection).

2. Design of site analyses conducted to further investigate causal relationships.

3. Selection of site-specific technologies to overcome on-site system problems.

The data presented in Table III-A-4 illustrates how the first decision-making phase was accomplished for the Salem case. Table III-A-4 correlates surface malfunctions (confirmed and marginal) and other on-site system problems with soil series and their attendant characteristics relevant to groundwater conditions and permeability. It also suggests technological responses to apparent, yet not fully defined problems. These "suggested responses" will be modified or verified once Phases 2 and 3 above are completed.

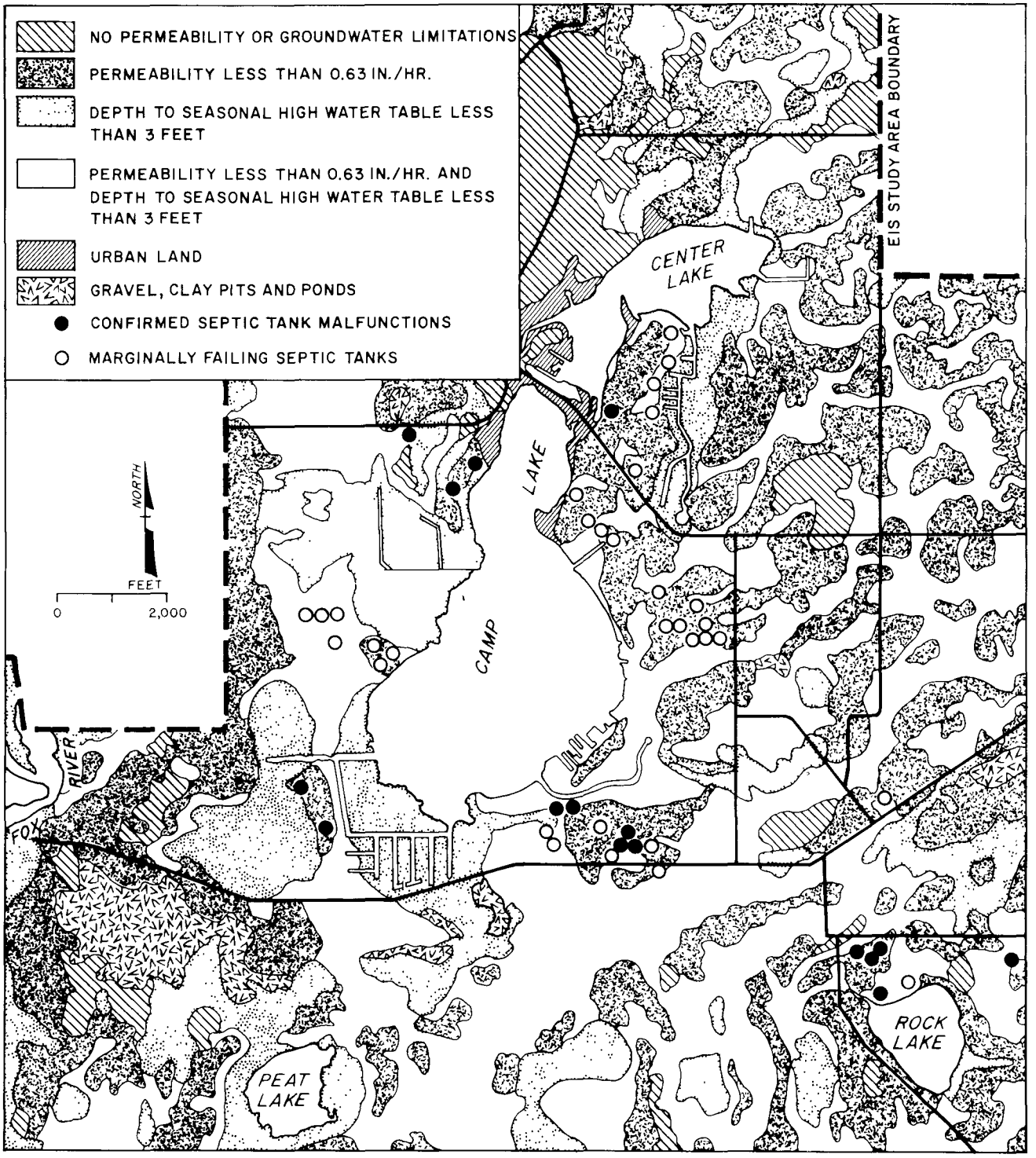

Figure III-A-4. Detailed correlation of available soil and malfunction data for Salem Utility District No. 2, Kenosha County, Wisconsin

Source: Link and Demo, 1970; U.S. EPA-EPIC, 1979

TABLE III-A-4. CORRELATION OF ON-SITE SYSTEM PROBLEMS WITH SOIL SERIES CHARACTERISTICS, IN THE FIRST DECISION MAKING PROCESS, SALEM CASE.

Location	Number of EPIC malfunctions	Other problems	Soil series	Depth to seasonal high groundwater (ft)	Permeability (in/hr)	(depth)	Suggested technologies to overcome identified problems
SILVER LAKE (Southeast)	0	1 groundwater plume	Casuo-Miami loam	3+	>20	(18-60")	New ST/SAS
CENTER LAKE (South	5		Hebron loam	3+	0.2-0.63	(28-60")	New ST/SAS
CAMP LAKE (Northwest)	3	High water table; slow permeability	Kane silt loam	0-1	>20	(30-40")	
			Casco sandy loam	3+	0.63-2.0	(40-60")	Elevated sand mound; new SAS
			Fox loam	3+	20	(18-60")	New ST/SAS
					20	(35-60")	New ST/SAS
CAMP LAKE (West)	7	High water table; slow permeability	Navan silt loam	0-1	0.06-0.2	(35-60")	Elevated sand mound; HT
			Martinton silt loam	1-3	0.06-0.2	(12-60")	Elevated sand mound; HT
			Houghton muck	0-1	2.0-6.3	(0-60")	HT
CAMP LAKE (Southwest)	2	Slow permeability	Saylesville	3+	0.06-0.20	(28-60")	New ST/SAS
CAMP LAKE (Northeast)	5		Navan silt loam	0-1	0.06-0.2	(35-60")	Elevated sand mound; HT
			Saylesville silt loam	3+	0.06-0.2	(28-60")	New ST/SAS
CAMP LAKE (East)	8	High fecal coliform count offshore	Navan silt loam	0-1	0.06-0.2	(35-60")	Elevated sand mound; HT
			Saylesville silt loam	3+	0.06-0.2	(28-60")	New ST/SAS
CAMP LAKE (Southeast)	11	High fecal coliform count offshore	Hebron Loam	3+	0.2-0.63	(28-60")	New ST/SAS
			Aztalan loam	1-3	0.2-0.63	(35-60")	Elevated sand mound; new ST/SAS
			Saylesville silt loam	3+	0.06-0.2	(28-60")	New ST/SAS
ROCK LANE	5		Fox loam	1-3	>20	(30-42")	Elevated sand mound
					<0..06	(40-60")	New ST/SAS
	1		Fox silt loam	3+	>20		New ST/SAS
			Morley silt loam	3+	0.02-0.63	(35-60")	

NOTE: ST/SAS -- Septic Tank - Soil Absorption System
HT -- Holding Tank

SOURCES: Link and Demo 1970
USEPA - EPIC 1979

Stage 5: Use of Causally Defined Relationships in Decision-Making for Similar Systems

Use of locally verified causal relationships between performance and factors influencing performance is not restricted to existing on-site systems that have failed. Their use extends to similar systems that have not yet exhibited any problems. Information developed and standardized during Stage 4 can be extrapolated for use in this stage provided conditions relevant to on-site system performance are similar. These conditions include soil characteristics along with system design condition, and maintenance.

Stage 6: Development of Regional or State Data Bases for Increasing Knowledge of On-Site System Capabilities and proven Limitations.

The final step of the empirical approach involves the synthesis of empirical soils and performance data developed and tabulated by local sanitarians and soil scientists for use by regional or state small waste flows officials. The accumulation of empirical data by counties and municipalities would improve knowledge of on-site system capabilities and proven limitations held by small waste flows planners at higher levels of government who are committed to provide cost-effective wastewater management in their jurisdictions. This commitment encompasses both existing and future on-site systems.

Development of regional or state data bases, over time, could result in modification of existing sanitation regulations which do not reflect actual performance of on-site systems under hypothetically limiting conditions. State regulations presently limit development on marginal but potentially satisfactory soils on the basis of unverified relationships between on-site system performance and soil characteristics. Computerized data bases which correlate soil characteristics and on-site system performance could improve design and permitting decisions. Until a substantial data base is established, however, the present criteria are the best available for permitting decisions relevant to new on-site systems.

Increased knowledge of on-site system capabilities and proven limitations at the regional and state level may also precipitate the preparation of "supplements" to conventional soil surveys addressing soil suitability for on-site sewage disposal. These supplements could be prepared by the USDA Soil Conservation Service in cooperation with State agricultural experiment stations but would incorporate empirical performance data secured by local sanitarians and soil scientists. Supplementary information would not supercede information contained in the soil survey but would help rural area residents install operational and economical on-site systems through inclusion of the following data (by telephone, Lloyd Wright, USDA Soil Conservation Service, Washington, D.C., 22 Dec 80):

- Performance history of systems in use by soil type
- Regulations pertaining to construction and maintenance of systems
- Correlation of soil limitations and performance, addressing why systems are working and why they are failing

- Local techniques which successfully overcome soil limitations.

b. Summary

The empirical approach recognizes that actual on-site system performance is influenced not only by local soil characteristics but also by system design, construction, condition, use and maintenance. It further recognizes that small waste flows decision-making, as opposed to large-scale planning, must be based upon locally verified, not hypothetical relationships between performance and performance "factors." Use of empirical soils data is feasible, conclusive, and cost-effective for small waste flows planning and site-specific technology selection. As with the soil survey and soil potential approaches, the use of empirical soils data does not eliminate the need for on-site investigation prior to site-specific system design and costing. Indeed, its use is based almost entirely on on-site investigation.

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APPENDIX A

The following are definitions of each of the soil potential classes as defined in Rogoff 1979.

Excellent Soil Potential--Soils rated excellent have properties exceptionally favorable for the intended use. Soil limitations or restrictions are minor and can be corrected with usual management techniques or practices to assure high performance for the intended use. The initial installation and management costs to establish the use or maintain it are inexpensive compared to those on all other soils in the county. Continuing limitations are slight after these corrective measures have been applied. Soils rated excellent are the best in the county for the intended use.

Good Soil Potential--Soils rated good have properties favorable for the intended use. Some soil limitations or restrictions exist, but measures necessary to overcome these limitations are available. The measures needed usually increase costs of establishing or maintaining the use, but are not generally prohibitive in relation to design costs for soils rated fair or poor. Limitations continuing after these corrective measures are installed are slight. Performance for the intended use can be expected to be good.

Fair Soil Potential--Soils rated fair have many properties favorable for the intended use. One or more soil limitations exist that can be overcome with corrective measures, but limitations are primarily of a continuing nature requiring practices or designs that need to be maintained, or are more difficult, unusual, and costly. Limitations continuing after these corrective measures have been applied are commonly moderate. Performance for the intended use can be expected to be fair.

Poor Soil Potential--Soils rated poor have properties unfavorable for the intended use. One or more soil limitations exist that are extremely difficult to overcome. The initial installation or maintenance cost of these measures, if available, are prohibitive as compared to those needed on all other soils in the country. Limitations continuing after these corrective measures are installed are severe and seriously detract from environmental quality or economic returns. Performance for the intended use can be expected to be poor. Soils rated poor are the worst in the county for the intended use.

B. PICKEREL LAKE, MICHIGAN, CLUSTER SYSTEM SITE ANALYSIS

Evaluations of needs for improved wastewater facilities were conducted around Pickerel Lake and along the south shore of Crooked Lake in Emmet County, Michigan during 1978. (U.S. EPA, 1980). In addition to scattered individual problems, two housing clusters on the south shore of Pickerel Lake, Ellsworth Point and Botsford Landing, were identified as possibly requiring some form of treatment off-site. Because of the proximity of soils that, based on county soils map information, are potentially suitable for multi-family soil absorption systems, cluster systems were proposed, subject to more thorough investigation of needs and to hydrogeologic testing of potential disposal sites.

Site selection and hydrogeologic testing of the disposal sites were performed to demonstrate appropriate techniques.

1. SITE SELECTION

Preliminary screening of potential cluster system sites was accomplished by evaluating the soils and geologic information previously gathered (Alfred et al., 1973). Additional information was gathered through a site visit and discussions with landowners concerning their knowledge of the soil on their property gained through water well drilling, septic tank installation, and personal investigation.

Sites for the proposed cluster systems were selected based on probability of thickness of unsaturated sands, ease of access (no trees), and cooperation from the landowners. The site locations are shown in Figure III-B-1. For the Botsford Landing area, the landowner whose property had the greatest potential based on all three criteria was Charles Kreeger, owner of Camp Petosega. Soil borings had been conducted on his property previously for an engineering design of a cluster drainfield for the camp. The area of investigation was to the west of where the area of interest currently lies. Two borings to 20 feet had encountered fine to medium sands throughout and no water table. These factors indicated that the Kreeger property would be ideal for a cluster drainfield.

For the Ellsworth Point area, the location that appeared to have the greatest potential for a cluster drainfield was the old beach sands on the terrace above Artesian Lane. Three properties on the terrace were identified as having the greatest potential--from east to west, those of Ray Johnston, Warren Keller, and Henry Hannan. Only the Keller property was cleared; access to the others would have been difficult and would have involved clearing. From the soils survey and information concerning their wells and septic tank drainfields obtained from the respective landowners, it was expected that the clay and gravel glacial till sloped steeply from south to north under the properties and that about 20 feet of unsaturated sands would be found near the escarpment. This information suggested that the Keller property would be suitable for a cluster drainfield.

WAPORA, Inc. subcontracted Geo-Tek, Inc., of Lowell, Michigan, to conduct the field testing program. This included borings and soil samples, installation of piezometers, water sampling, and restoration of the sites. WAPORA, Inc. personnel directed the work, performed the level survey, and conducted

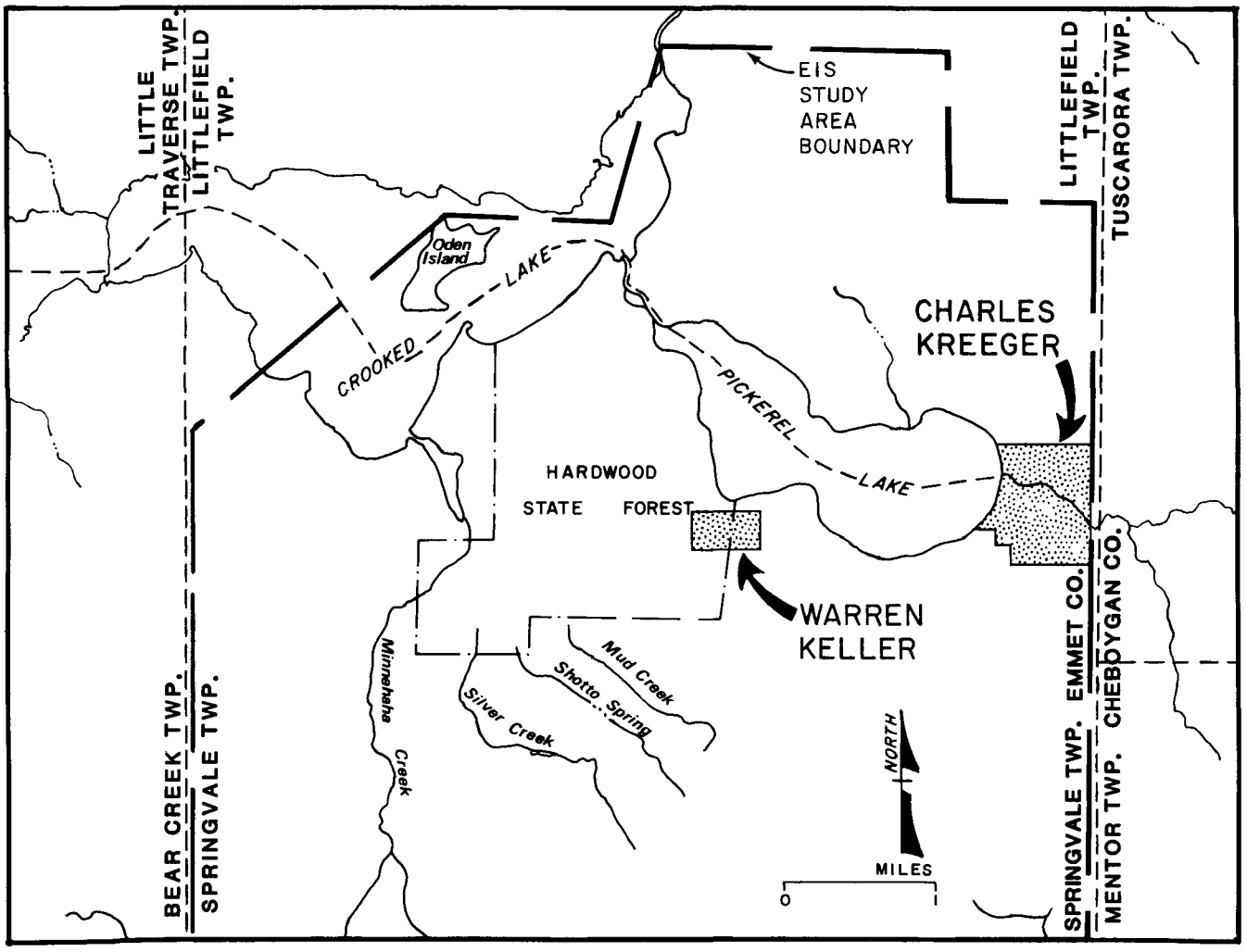


Figure III-B-1. The two properties proposed for cluster drainfields on which soil boring programs were conducted. Emmet County, Michigan.

the water quality tests. The field work commenced on 20 October 1980 and was completed on 22 October 1980.

2. SOIL BORINGS

The soil borings were conducted with a CME-45 drill rig mounted on a four-wheeled trailer. The soil borings were drilled to depths ranging from 19.5 to 50.5 feet., they were drilled by means of continuous flight, hollow-stem auger. At each site, one boring was conducted with continuous split spoon sampling through and below the hollow-stem auger. This allowed the drillers to observe undisturbed cores of soil material for the drilling log. The other borings were conducted with continuous auger boring, with observations made of the material brought to the surface for the drilers log, and by occasional split spoon samples when changes of material were encountered.

Undisturbed samples of soil material for permeability and soil characteristics tests were obtained by means of three-inch diameter Shelby tubes. These were taken from representative soil material at various depths through and below the hollow-stem auger. The Shelby tube ends were sealed with melted wax for transport to the soil testing laboratory.

a. Piezometers and Water Sampling

In borings where enough groundwater was encountered to justify a piezometer, a two-inch diameter well was installed through the hollow-stem auger. The piezometer consisted of a six-foot-long slotted (0.010 inch) plastic well screen and solvent welded Schedule 40 PVC pipe for the casing.

After each piezometer had rested for at least 24 hours, the depth from the ground surface to the water table was measured. Then, an electrically powered jet pump was attached to the exposed casing and the well pumped to clear water where this was considered feasible. A water sample was then taken, its pH and temperature measured and recorded, and the sample bottle marked and packed for shipment to the WAPORA, Inc. water testing laboratory in Cincinnati, Ohio. Where too small amounts of saturated sands were encountered to justify attempts to pump the water clear, a modified bailing procedure was employed to obtain a water sample. The relative elevation of the ground surface at each boring was obtained with a surveyor's level. On the Kreeger property, a temporary bench mark established by a previous survey was utilized. On the Keller property, the ground elevation was estimated based on USGS topographic contours.

After the water samples had been collected, the piezometers were pulled and the holes filled with the excavated material.

b. Laboratory Soil Tests

The laboratory tests on the soils were conducted by Materials Testing Consultants, Inc. of Grand Rapids, Michigan, for Geo-Tek, Inc. The soil samples were tested for original moisture content, original wet density, dry density, specific gravity, void ratio, and undisturbed vertical permeability. These tests were conducted using standard laboratory methods for soils.

The capability of soil material to accept and conduct water determines the maximum rate at which septic tank effluent can be applied to the drainfield. In this study, the most important hydraulic properties were vertical

and horizontal permeabilities. The vertical permeability of the soil samples was determined in the laboratory by the falling-head permeameter method. Where the soil material is nearly homogeneous, the horizontal permeability will approach the value for vertical permeability but will almost always be larger because horizontal layering affects vertical permeability more than horizontal permeability. To illustrate the effect of layering on horizontal and vertical permeabilities, the following example, using values from the laboratory testing, is presented. The first layer is 10-feet thick and has a permeability of 2 inches per hour, the second layer is 0.1-feet thick and has a permeability of 0.02 inch per hour, and the third layer is 10-feet thick and has a permeability of 2 inches per hour. The vertical permeability (harmonic mean) would be given by:

$$Kv = \frac{D}{\frac{d1}{k1} + \frac{d2}{k2} + \frac{d3}{k3}}$$

$$= \frac{20.1 \text{ ft}}{\frac{10 \text{ ft}}{2 \text{ in/hr}} + \frac{0.1 \text{ ft}}{0.02 \text{ in/hr}} + \frac{10 \text{ ft}}{2 \text{ in/hr}}}$$

$$Kv = 1.34 \text{ in/hr}$$

This value is considerably less than that of the majority of the soil material. The horizontal permeability would be given by:

$$Kh = \frac{d1k1 + d2k2 + d3k3}{D}$$

$$= \frac{10 \text{ ft} \times 2 \text{ in/hr} + 0.1 \text{ ft} \times 0.02 \text{ in/hr} + 10 \text{ ft} \times 2 \text{ in/hr}}{20.1 \text{ ft}}$$

$$Kh = 1.98 \text{ in/hr}$$

Thus, overall horizontal permeability is only minimally affected by thin layers of lesser permeability. These formulae must be considered when the laboratory data for vertical permeability are utilized for horizontal permeability estimates.

c. Laboratory Water Tests

The water samples were analyzed in the WAPORA, Inc. laboratory in Cincinnati, Ohio on 22 October 1980. The tests for total phosphorus and nitrates were performed according to the standard procedures given in the Standard Methods for Examination of Water and Wastewater (American Public Health Association, 1975). Two samples could be tesced directly, while two others had to be filtered before the laboratory tests because excessive soil particles were present (these were from the bailer-sampled wells).

3. RESULTS AND DISCUSSION FOR EACH SITE

The field and laboratory program has provided quantitative information about the physical and chemical characteristics of the two sites under consideration as potential cluster drainfield sites. The following sections pre-

sent detailed descriptions and discussions of each site, based on the test results and their interpretations.

a. Charles Kreeger Property

This site (Figure III-B-2) includes the southwest corner of Camp Petosega at the bend in Camp Petosega Road. It is located in T. 35 N., R. 4 W., sec. 36 and is under the political jurisdiction of Springvale Township. The land to the south and west is owned by Eleanor Poquette. The site plan shown was taken from a topographic plan prepared by Abrams Aerial Survey for the owner.

Presently, the area is covered with unmowed grasses and weeds, typical of infertile sands that have a low water-holding capacity. The quality of the grasses improves noticeably toward the south property boundary. A strip of trees, about 80-feet wide, borders the south property boundary. A mowed softball diamond is located in the southwest corner of the property. Also shown on the site plan are the rough boundaries of the airfield that was used until about 1974 by the former owner. (Its use may be restored again.) A high-pressure gas pipeline bisects the property as shown. Northwest of boring 1 is an active borrow area where sand has been removed for construction activities in Camp Petosega.

This site is characterized by nearly level topography and sandy soils. The surface soil material consists of old lake beach overlying morainal material. The topography and soil mapping indicate that the morainal material slopes from south to north in the general area. This is also borne out by the soil borings and well logs from the area. The old lake beach was probably eroded along the bottom of the escarpment that lies to the north of borings 1 and 2. The sands exposed within the borrow area exhibit the variable deposition angles characteristic of beaches and dunes.

The soil boring logs prepared from observations by WAPORA, Inc. personnel are presented in Figures III-B-3, III-B-4, and III-B-5. The driller's logs are included in Appendix A. The two borings on the north, borings 1 and 2, show that medium and fine sands dominate to a depth of about 25 feet, below which some thin lenses of silty sand are present to a depth of about 40 feet (Figure III-B-6). The boring to the south, boring 3, shows that the lenses of silty sands are encountered at a depth of only 7 feet and that clayey material is encountered at a depth of 24 feet. Thus, the borings show that permeable material is much thicker near the escarpment, and that the logical location for the drainfield would be near the escarpment.

Water table measurements are also shown on the soil boring logs (Figures III-B-3, III-B-4, and III-B-5). The water table drops approximately 9 feet between boring 3 and borings 1 and 2 (Figure III-B-6). This strong slope reflects the presence of an aquitard that falls off in the direction of Pickerel Lake. The groundwater flow direction would be toward the lake, in a north-northwesterly direction. The owner reports that below the escarpment the groundwater depth is approximately 3 feet or an elevation of 606 above mean sea level (MSL). The water elevation in Cedar Creek, about 1300 feet north of boring 1, is about 595 MSL. Pickerel Lake, about 1300 feet northwest of boring 1, has an elevation of about 593 MSL. Because the groundwater flow from the south becomes concentrated toward the lake, the slope of the water table surface and the flow of groundwater is probably in the direction of the lake and not north toward Cedar Creek.

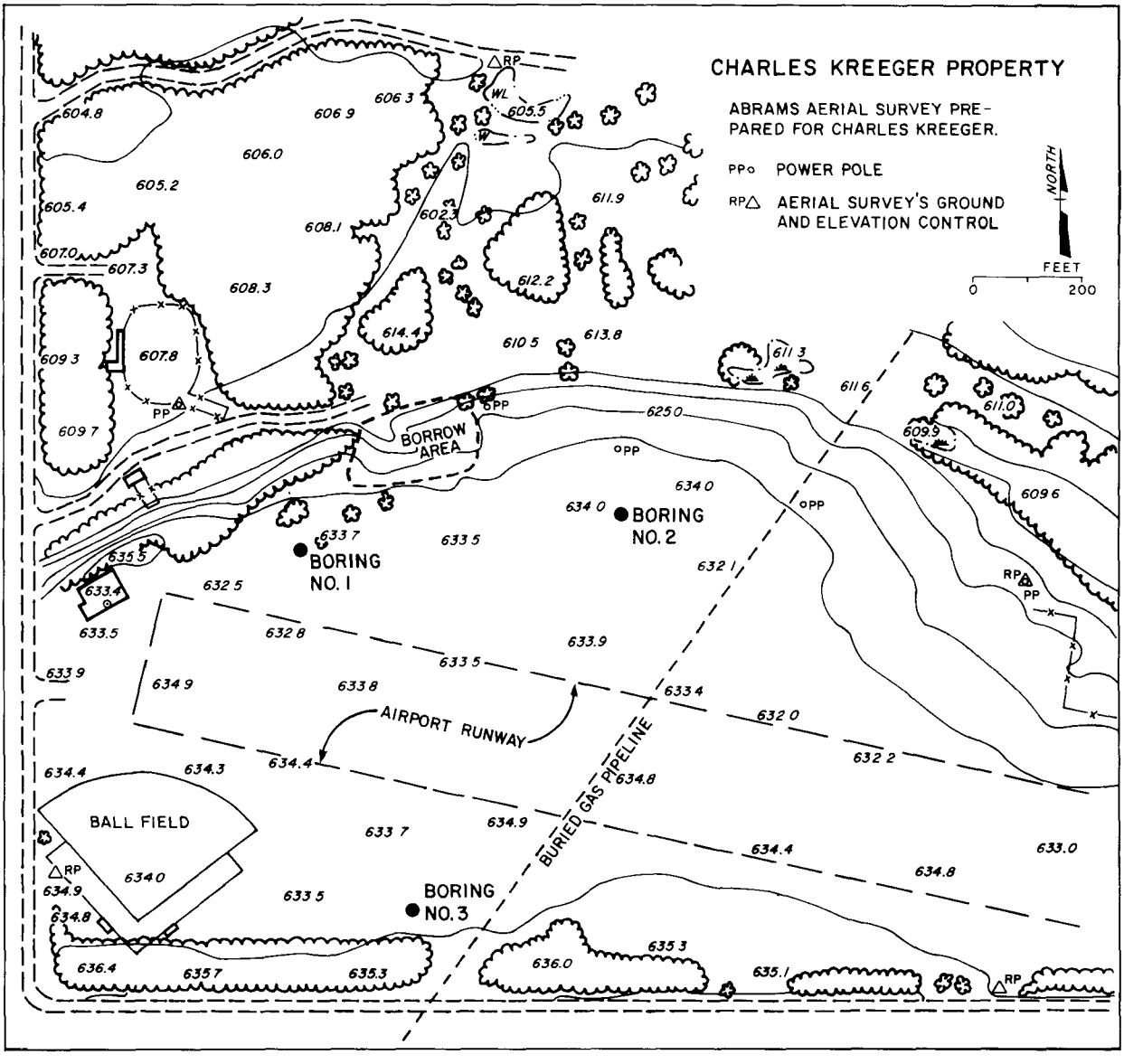


Figure III-B-2. Soil boring locations on the Charles Kreeger property.

Figure III-B-3. Soil boring 1, Charles Kreeger property, performed by Geo-Tek, Inc., on October 20, 1980. Logged by continuous split spoon sampling.

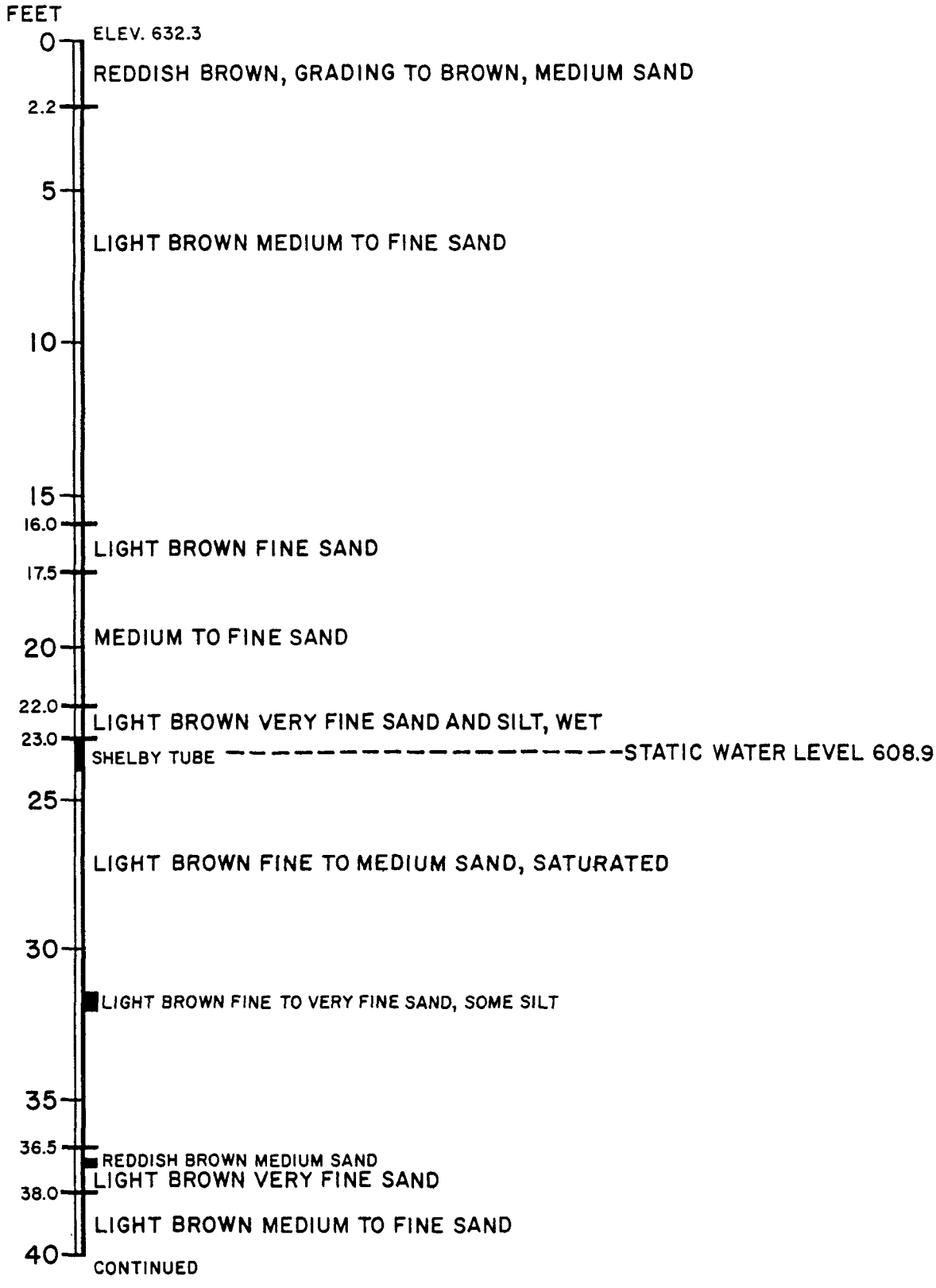


Figure III-B-3. (continued)

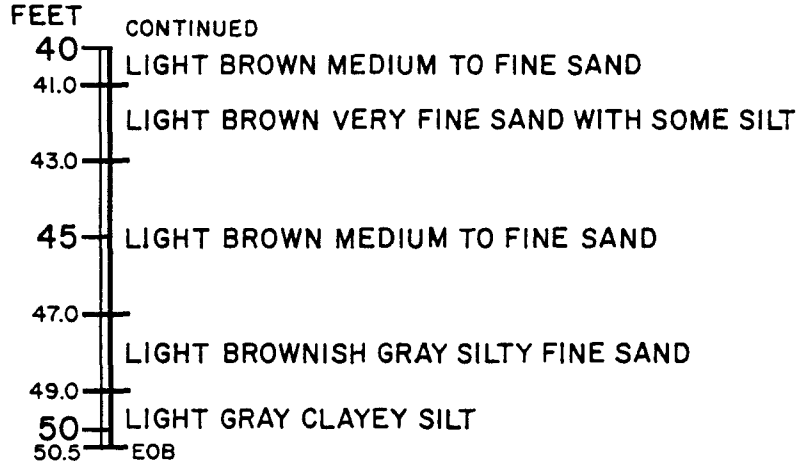


Figure III-B-4. Soil boring 2, Charles Kreeger property, performed by Geo-Tek, Inc., on October 20, 1980. Logged by auger cuttings.

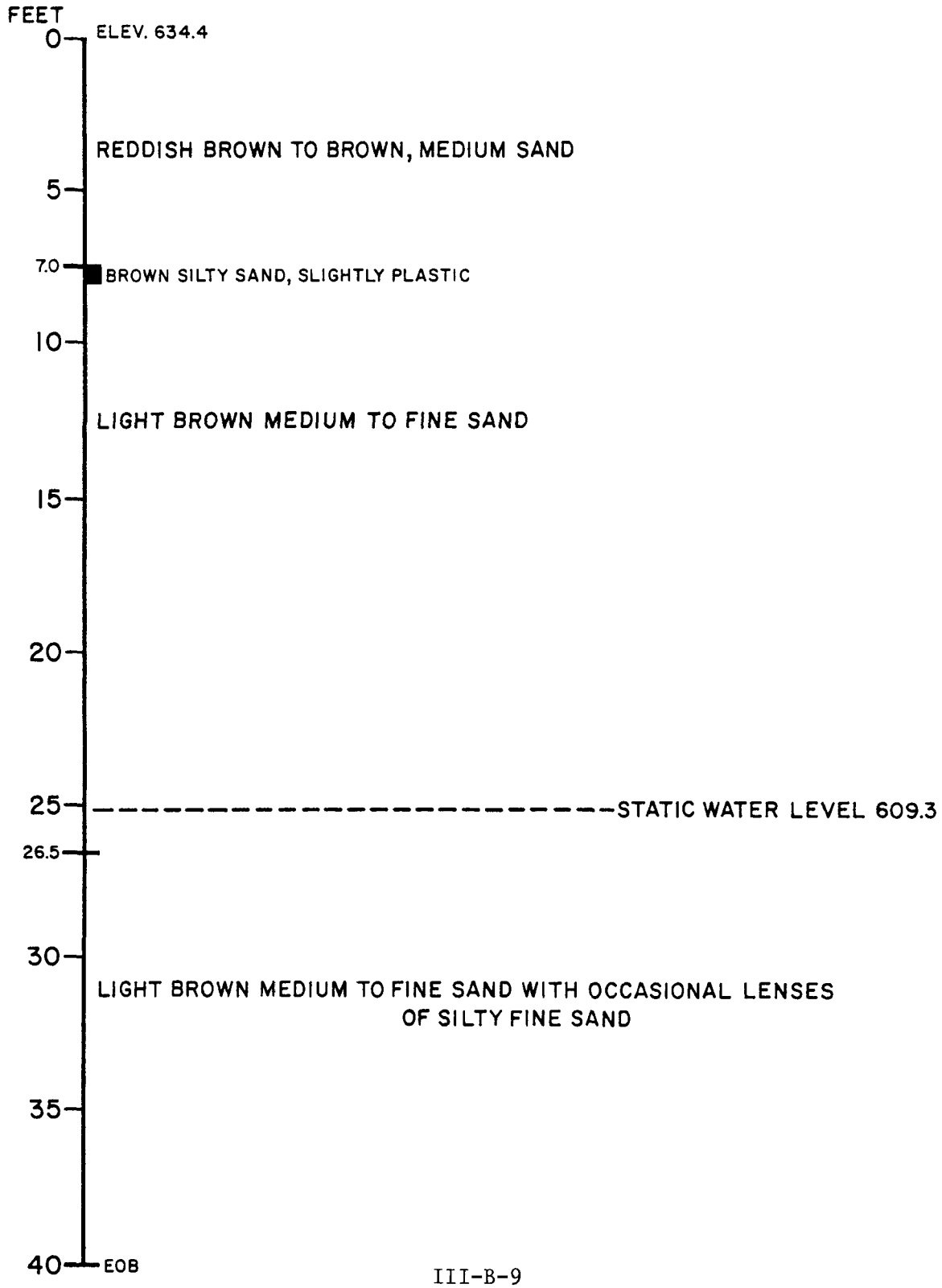
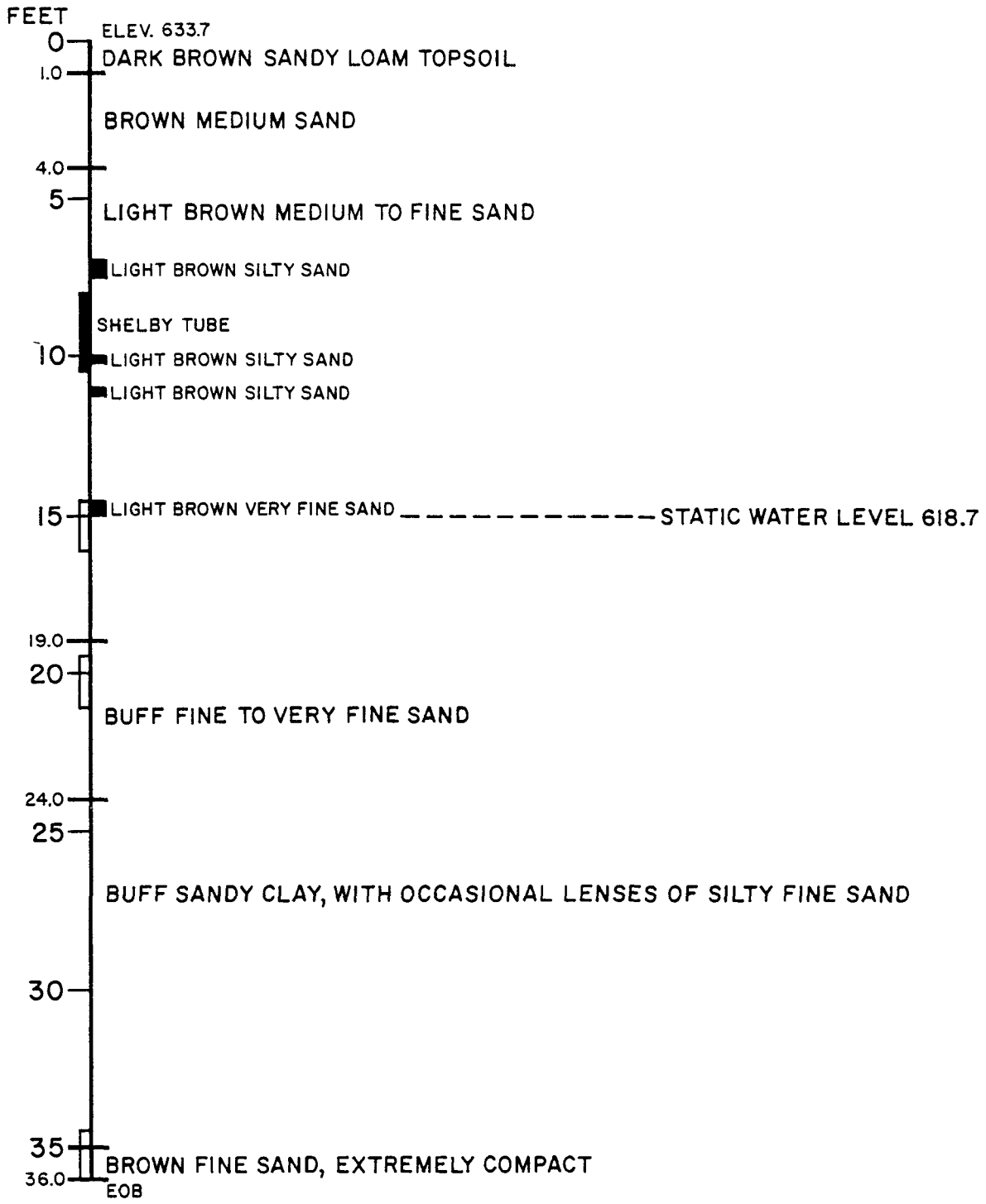


Figure III-B-5. Soil boring 3, Charles Kreeger property, performed by Geo-Tek, Inc., on October 20-21, 1980. Logged by auger cuttings and periodic split spoon samples. (Two adjacent borings logged as one.)



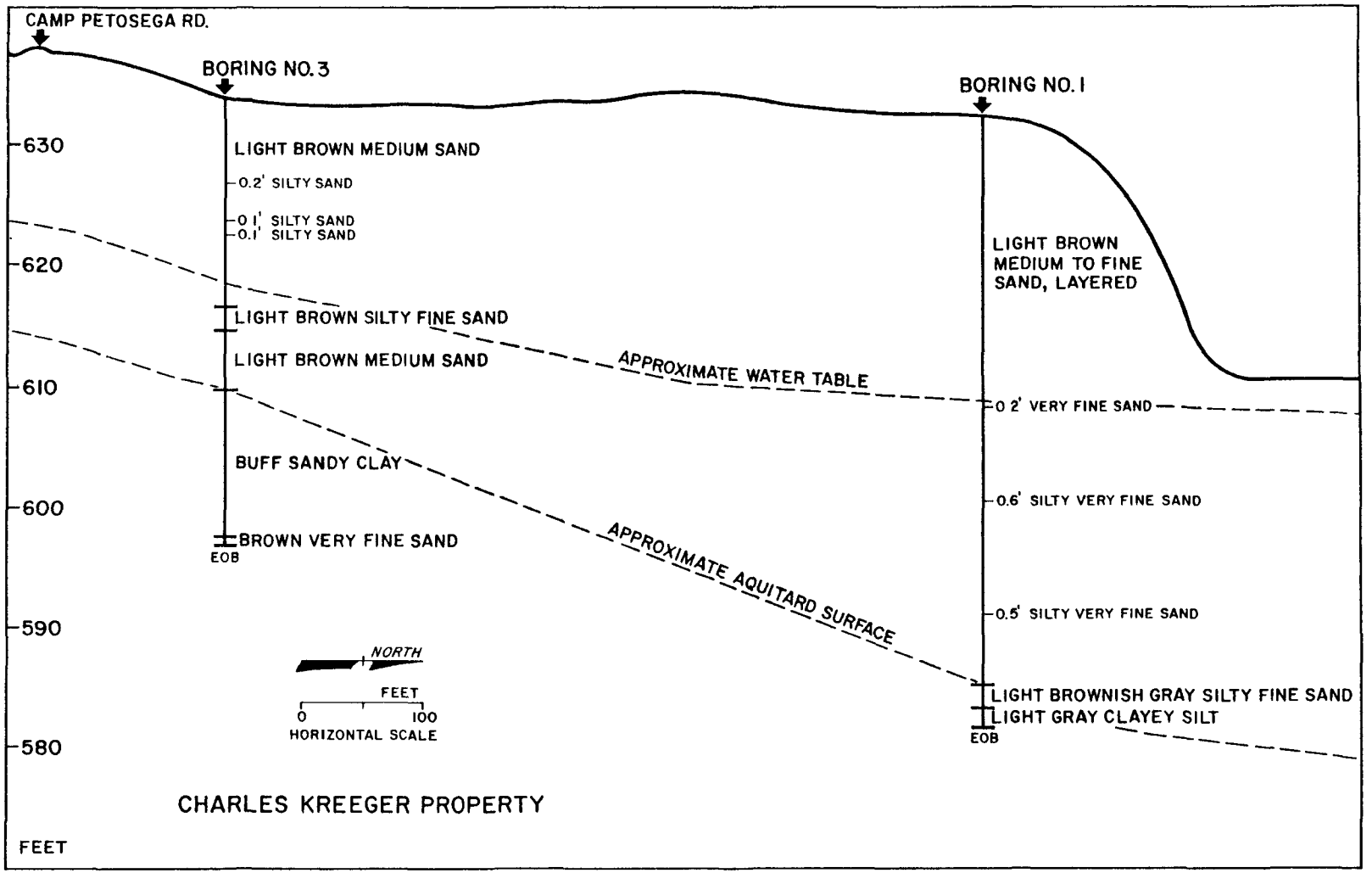


Figure III-B-6. Generalized geologic cross section of the Charles Kreeger property.

The water sample results are shown in Table III-B-1. Generally, the groundwater quality is good, though its slightly acidic nature means that the water may have elevated iron contents. Also, the water pumped from boring 1 had a "swampy" odor and taste. The phosphorus levels are typical for natural groundwater. The nitrate levels are consistent with other reported values for mixed forest and cropland. At the Muskegon, Michigan, Wastewater Management System, background levels of nitrates from wells in similar beach sands have average ranges from 0.1 to 1.2 mg N/l (Demirjian and others, 1980).

TABLE III-B-1. GROUNDWATER SAMPLING RESULTS FOR POTENTIAL CLUSTER DRAINFIELD SITES AT PICKEREL LAKE, MI, SAMPLED ON 22 OCTOBER 1980.

Parameter	Sample location and piezometer: Charles Kreeger 1	Charles Kreeger 2	Charles Kreeger 3	Warren Keller 7
Piezometer depth (feet)	47	40	20	31
Thickness of saturated material (feet)	23.6	14.9	5.0	5.3
Type of sample	Pumped (clear)	Pumped (clear)	Bailer (filtered)	Bailer (filtered)
Temperature (°C)	8	9	9	9
pH (standard units)	6.2	6.2	6.8	6.9
Nitrate (mg N/l)	0.961	1.21	1.37	1.66
Total phosphorus (mg P/l)	0.0024	0.0036	0.0040	0.0018

Soil samples for laboratory analysis (Table III-B-2 and Appendix B) from the Kreeger property were kept to a minimum in order to minimize costs. One Shelby tube each from borings 1 and 3 were obtained, but the tube from boring 1 contained insufficient sample for testing. Nearly all of the soil material encountered in borings 1 and 2 was similar to the soil material in the tube sample from boring 7. The vertical permeability of that sample (boring 7) was 4.1×10^{-3} cm/sec (3.6 in/hr), a value consistent with the particle size of the soil material encountered. A weighted average value of 3 in/hr for horizontal permeability was selected.

In an attempt to model a simplified system that resembles a uniform application of water over a long, rectangular area and a drainage ditch along one side. The Hooghoudt equation for a flat area will adequately predict what will occur. The form of the equation utilized is as follows:

$$L = \frac{K}{q} (y0^2 - y1^2)^{1/2}$$

TABLE III-B-2. SOIL TESTING RESULTS FOR POTENTIAL CLUSTER DRAINFIELD SITES AT PICKEREL LAKE, MI, SAMPLED ON 20-21 OCTOBER 1980

Boring number	Depth(ft)	Soil description	Moisture content(%)	Wet density(pcf)	Dry density(pcg)	Specific gravity	Void ratio	Laboratory vertical permeability[cm/sec(in/hr)]
Kreeger								
1	24.0-26.5	(Insufficient recovery for laboratory testing)						
3 (4)	8.0-10.5	Fine to medium sand	6.4	103.3	97.1	2.644	0.699	1.4 E-3 (1.97)
Keller								
4 (1)	24.0-26.5	Fine to silty fine sand	12.4	128.9	114.7	2.680	0.46	2.1 E-5 (0.03)
5 (2)	9.5-11.0	Fine sand	3.9	116.8	112.4	2.631	0.461	7.8 E-3 (11.0)
6 (3)	14.0-16.5	Fine sand to silty clay	20.2	130.1	108.2	2.695	0.55	8.1 E-7 (0.00115)
7 (4)	8.0-10.5	Fine sand	8.1	106.9	98.9	2.644	0.668	4.1 E-3 (3.6)

Notes:

1. Boring 5 was sampled and tested using brass sleeve split spoon samples.
2. Boring numbers in parentheses are the driller's numbers.
3. Laboratory vertical permeability results are presented in scientific notation. The number to the right of E indicates the order of magnitude. Example: 1.4 E-3 = 1.4 x 10^{-3} = 0.0014.

where: L = distance from the drainage ditch to the far side of the application area, ft

K = horizontal permeability, in/hr

q = drainage or application coefficient, in/hr

y0 = allowable height of saturated material above a specific datum, ft

y1 = height of the bottom of the drainage ditch above that same datum, ft

Values derived from Figure 3 are:

y0 = 30 ft

y1 = 18 ft

A liquid application coefficient, q, of 0.34 gal/ft^2/day, 1.9 x 10^3 in, calculated for trench bottom was used. This coefficient assumes a conservative percolation rate of 15 min/hr; 3 persons/house; an average flow of 65 gal/person/day; and 190 sq ft of trench bottom for every two people (per bedroom). Substituting these values into the equation yields a distance of 390 feet from the drainage ditch (bottom of escarpment) to the far edge of the application area. Considering the proportion of seasonal residences, the water table would actually rise during the summer and fall during the remainder of the year. Thus, this site would perform very well as a drainfield site. Vertical permeabilities in the top 25 feet of material are greater than the application coefficient and would not cause a perched groundwater condition.

The ultimate path of the applied water is difficult to determine without more extensive soils investigations. In general, the water would flow toward the creek and toward the lake. Also, the rise in the groundwater table in the area below the escarpment may be assessed only if considerably more soils information is gained. Because the camp facilities are located on soils with high water tables, protection of these facilities from further rises of the water table may be deemed advisable. A drainage ditch or draintile below the escarpment would provide a great measure of protection.

The pollutants in the septic tank effluent applied to the drainfield would, with the exception of nitrates and chlorides, be nearly completely removed in the soil matrix. Nitrogen in the effluent would be partially taken up by the plant cover, denitrified, and leached to the groundwater flow system. Denitrification and uptake could account for some losses, although the precise percentage cannot be estimated for various soils. Denitrification and uptake are generally greater in fine-textured soils than in sands and gravels. Dilution is recognized as the key mechanism for achieving acceptable levels of nitrates in groundwater below a drainfield. Downgradient from the potential cluster drainfield site are several residences near the lakeshore and the Camp Petosega campsites. It is likely that they all have wells that penetrate to about 50-foot depth into the surficial sands into which the drainfield would discharge. Because stratification of groundwaters usually occurs, it is unlikely that excessive nitrate levels would be encountered in any of the domestic wells. Because the depth to the water table in a large

area below the escarpment is shallow, a considerable portion of the nitrates could be removed by vegetation in that area. If a draintile or ditch would be constructed below the escarpment, much of the nitrate-enriched groundwater would be intercepted and conveyed directly to a surface outlet. The contribution of nitrates to lake eutrophication would be significant.

b. Warren Keller Property

This site (Figure III-B-7) is comprised of the southeast corner of the property of Warren Keller located on Artesian Lane west of Ellsworth Road. It is located in the NW¼ of the NE¼ and the NE¼ of the NW¼ in T. 35 N., R. 4 W., sec. 34 and is under the political jurisdiction of Springvale Township. The land to the east is owned by Ray Johnston; the land to the south is owned by Charles Laubrick along the eastern quarter mile and by Henry Hannan along the western quarter mile. The site plan shown was developed from the USGS quadrangle map, the soil survey, the plat map published by Rockford Map Publishers, and some on-site surveying. The site plan is a schematic only and is not intended to be accurate.

The potential cluster drainfield site is presently unmowed grasses and weeds. Some scattered small trees are present in the northeastern portion of the area. The approximate location of an electrical power line is shown on the site plan. The landowner has proposed construction of a residence in the northeastern portion of the site plan. An access road from Artesian Lane is proposed for the residence. Access to the potential area is limited at present to lanes through the Laubrick and the Hannan properties.

The site is characterized by gently sloping topography. About 10 feet of relief, from northwest to southeast, characterizes the site. The escarpment represents about 35 feet of elevation difference. The soils (Kalkaska sand) are formed in old lake beach material overlying morainal material. The topography and soil mapping indicate that morainal material slopes from south to north in this area. The topographic high to the south of this site is part of this moraine. The old lake beach was probably eroded along the bottom of the escarpment by another lake level higher than the present lake level. The sands exposed along the escarpment exhibit the variable deposition angles characteristic of lake beaches and dunes. The Keller and Kreeger properties both exhibit similar geologic sources, except that the morainal material is nearer the surface and lies at a steeper slope on the Keller property.

The soil boring logs prepared from observations by WAPORA, Inc. personnel are presented in Tables III-B-8, III-B-9, III-B-10, and III-B-11. The driller's logs are included in Appendix A. The three borings on the north, borings 4, 6, and 7, show that medium and fine textured sands are found to depths of 14, 13, and 17 feet, respectively. Thin lenses of silty sand in the medium and fine sands occur below the clean sands. In borings 4, 6, and 7, silty clays were encountered at depths of 21, 18, and 27 feet, respectively. In the boring to the south, boring 5, medium and fine sands were encountered to a depth of 8 feet, below which was encountered very compact, light brown, medium sand that appears to be older and associated with the moraine. At a depth of 25 feet, bluish gray, silty clay was encountered. The borings show that lake beach material is thicker toward the escarpment (Figure III-B-12).

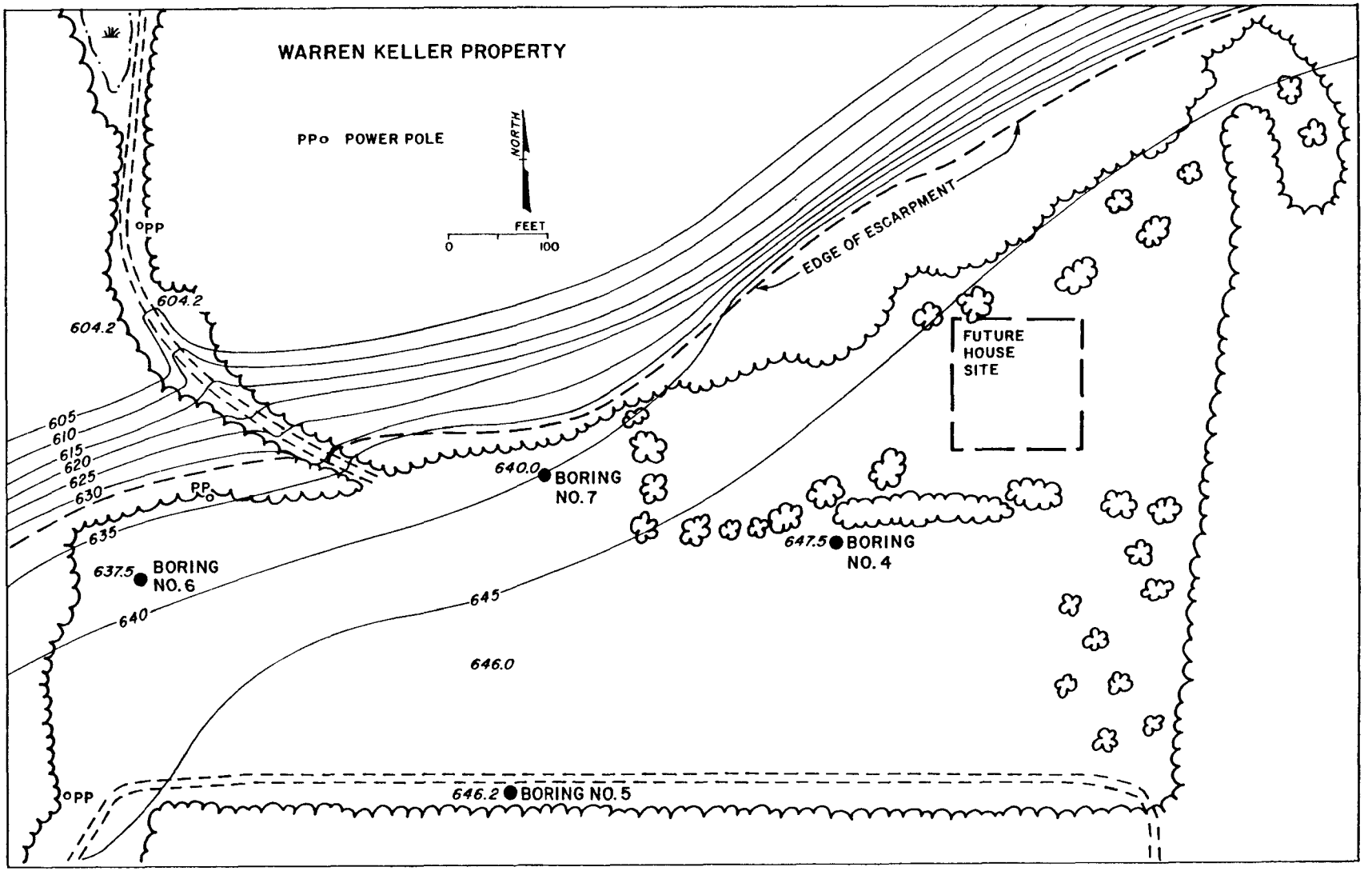


Figure III-B-7. Soil boring locations on the Warren Keller property (contours and site layout are approximate only).

Figure III-B-8. Soil boring 4, Warren Keller property, performed by Geo-Tek, Inc., on October 21, 1980. Logged by continuous split spoon sampling.

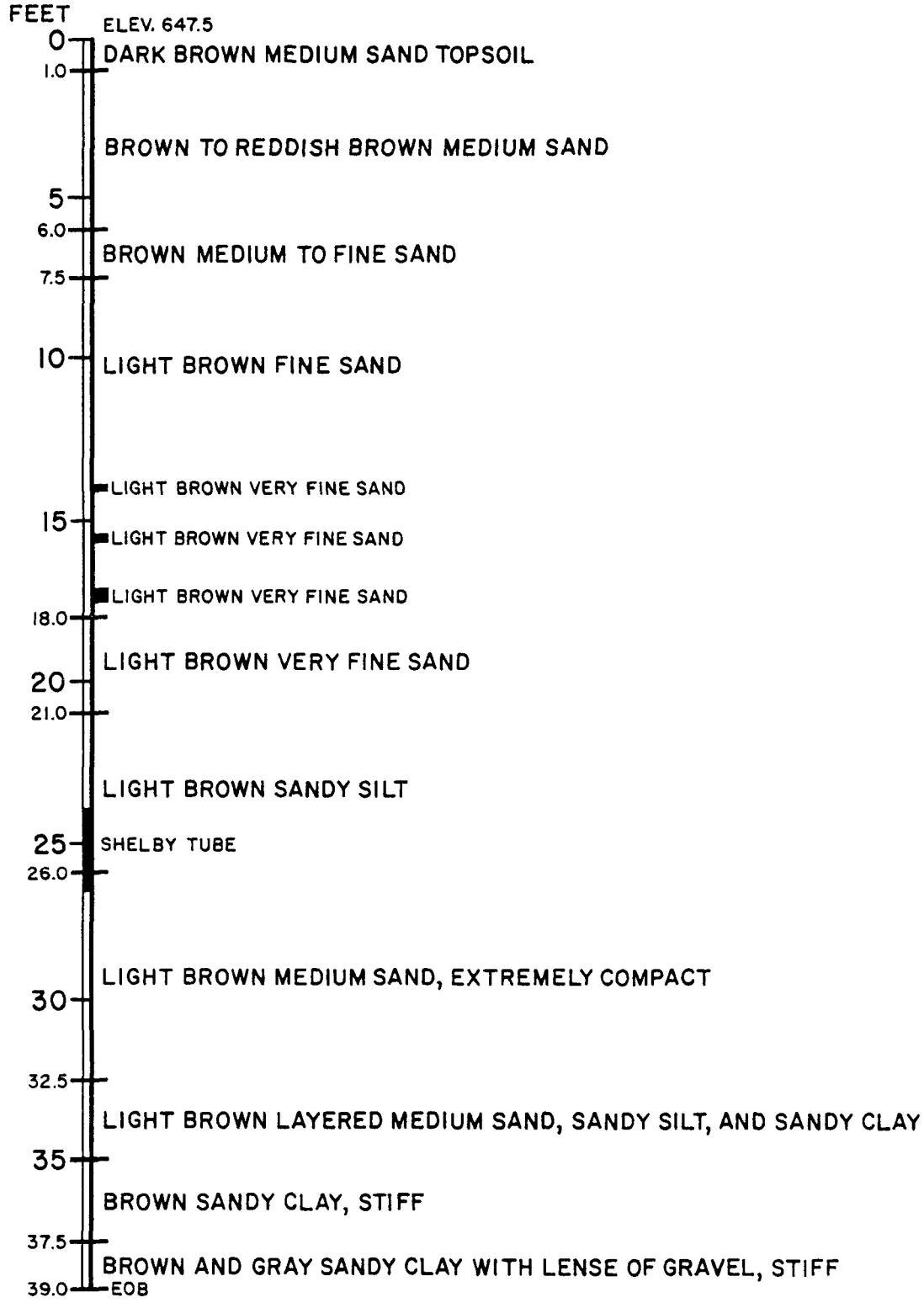


Figure III-B-9. Soil boring 5, Warren Keller property, performed by Geo-Tek, Inc., on October 21, 1980. Logged by auger cutting and occasional split spoon sampling.

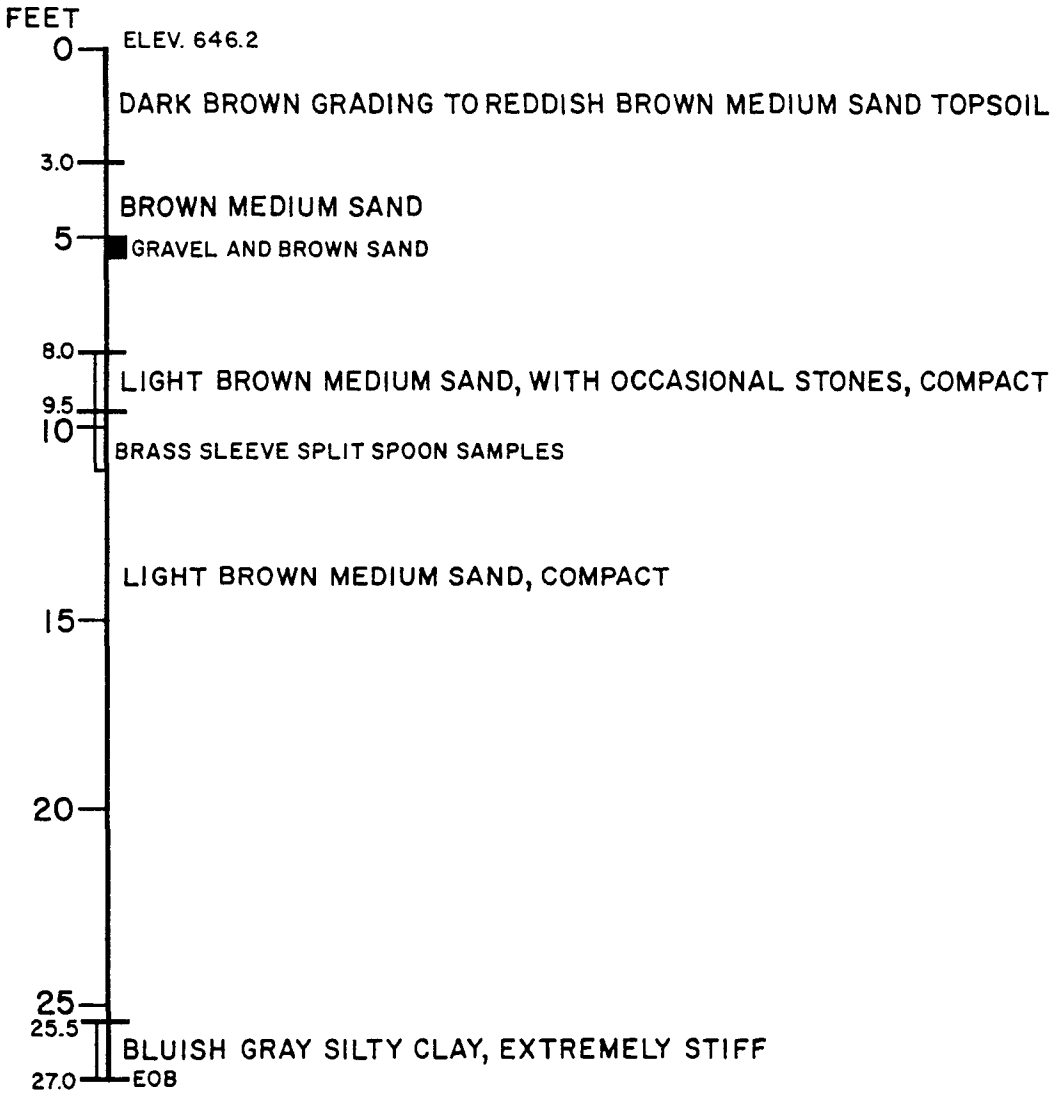


Figure III-B-10. Soil boring 6, Warren Keller property, performed by Geo-Tek, Inc., on October 21, 1980. Logged by auger cuttings and periodic split spoon sampling.

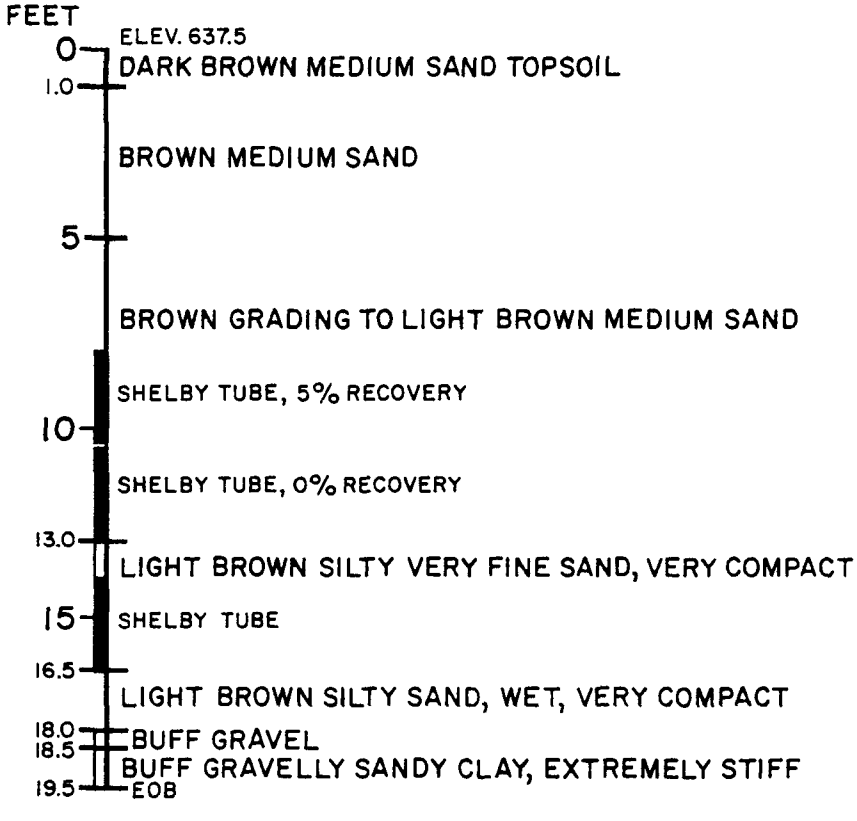
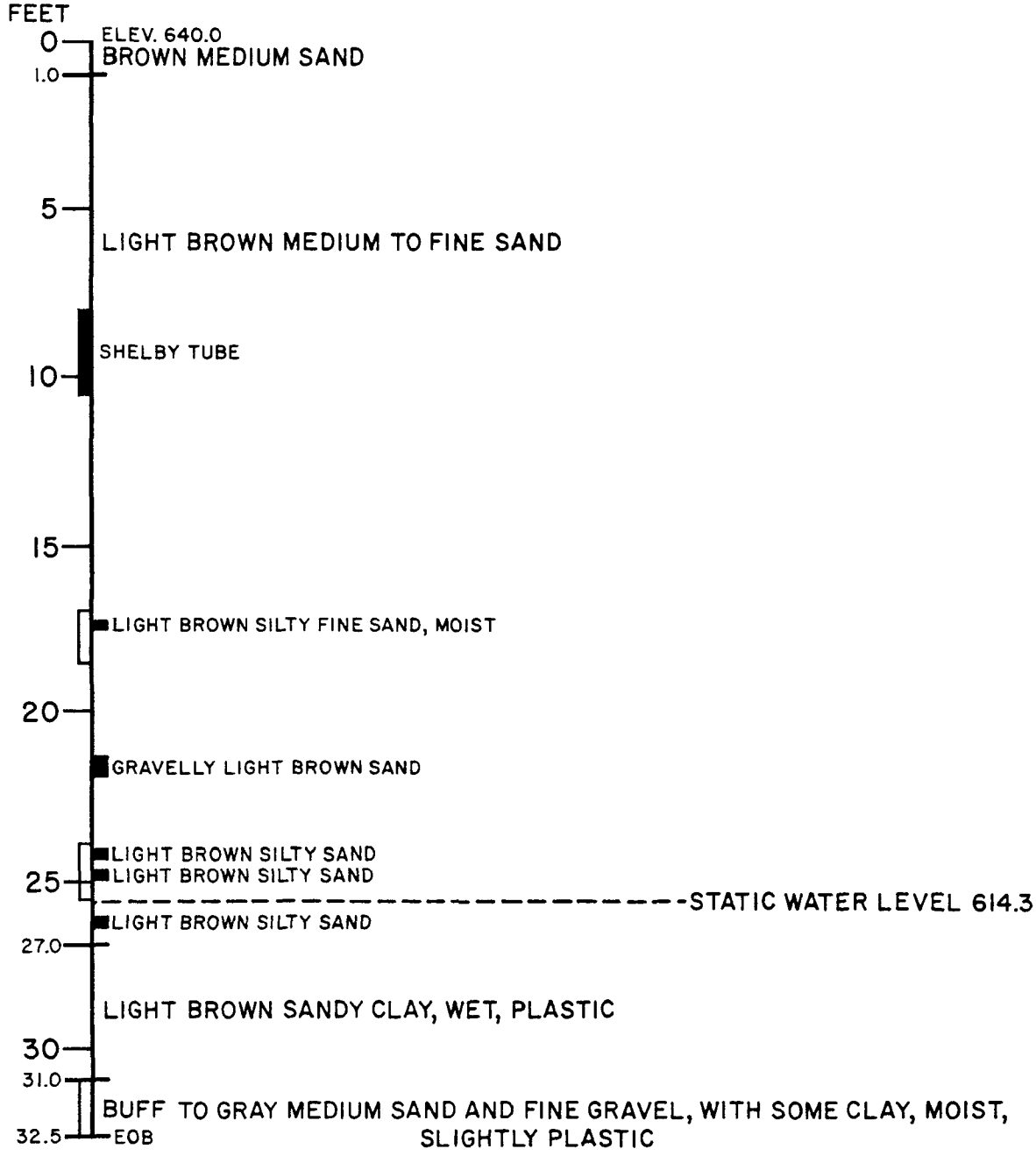


Figure III-B-11. Soil boring 7, Warren Keller property, performed by Geo-Tek, Inc., on October 21, 1980. Logged by auger cuttings and occasional split spoon sampling.



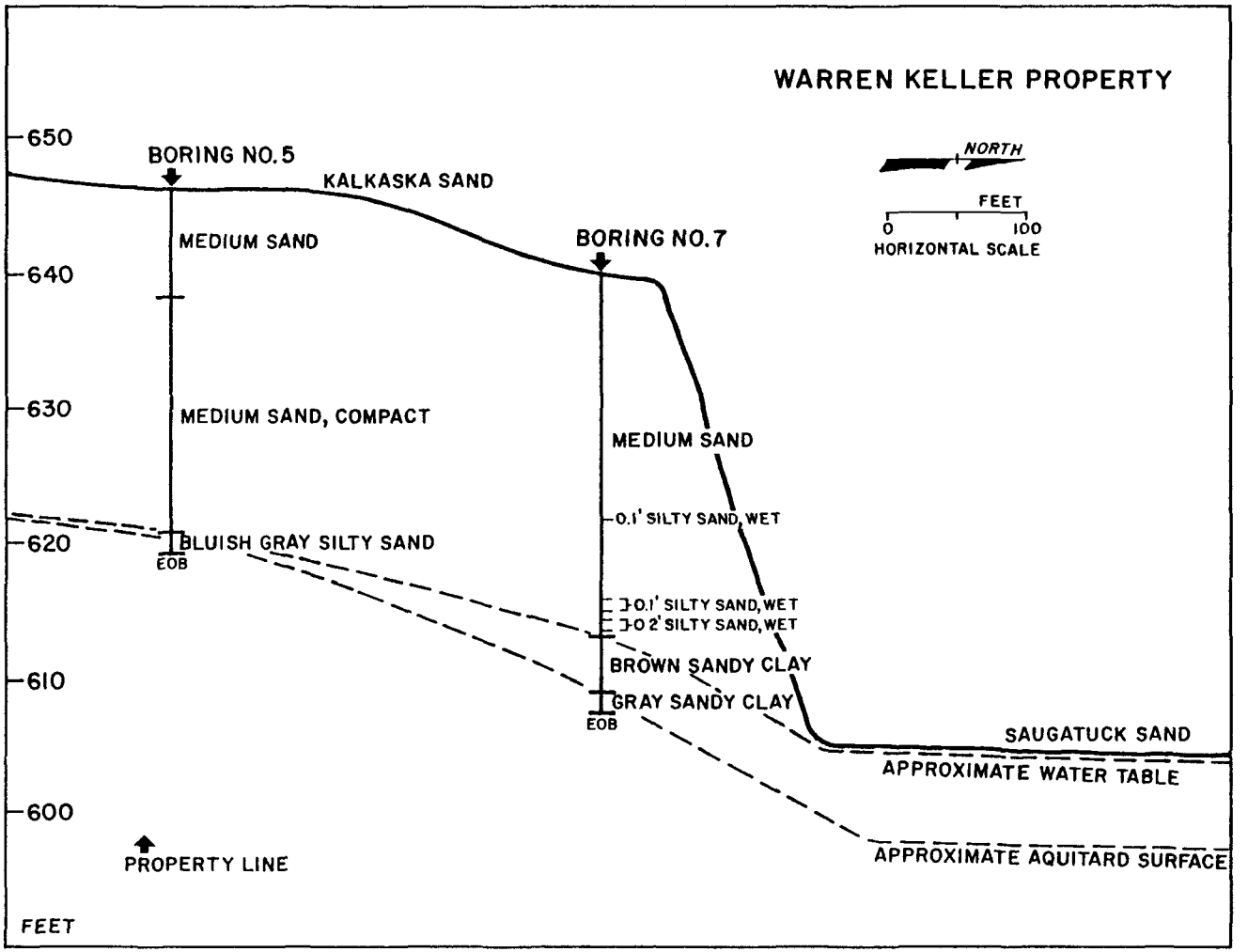


Figure III-B-12. Generalized geologic cross section of the Warren Keller property.

In all of the borings, very little saturated, permeable material was encountered above the aquitard. Even in the boring in which the piezometer was placed, boring 7, only little more than 1 foot was measured. Groundwater movement would proceed downgradient, toward the north. Topography and thickness of saturated material shows that the area contributing groundwater to the potential drainfield site is probably limited in size. The soil survey and observations show that, below the escarpment, the water table is at the soil surface. The soil material is sand and silty clay, similar to the soil material encountered in the borings at somewhat the same elevations.

The results of the water sample testing are shown in Table III-B-1. The lone sample indicates that the groundwater quality is good. The results are comparable to the quality of the groundwater tested from the piezometers installed on the Kreeger property. The pH was slightly higher; nitrate was also slightly higher; and total phosphorus was slightly lower. Recovery of a sample was achieved with a bailer, and the sample was very turbid. The piezometer had to be set down into the sandy clay in order to be able to recover a sample. Thus, the sample may not be representative of groundwater quality.

The results from the laboratory analysis of the soil samples are presented in Table III-B-2 and Appendix B. Shelby tube samples were obtained from borings 4, 6, and 7, and a brass sleeve split spoon sample from boring 5. The vertical permeability results showed a range from 7.8×10^{-3} cm/sec (11.0 in/hr) to 8.1×10^{-7} cm/sec (0.00115 in/hr). The high vertical permeability came from the brass sleeve split spoon sample from boring 5 while the low vertical permeability came from boring 6 of material identified as fine sand to silty clay. The surface soil material in the three borings near the escarpment is characterized best by the soil sample from boring 7. The vertical permeability was measured as 4.1×10^{-3} cm/sec (3.6 in/hr). Thus, similarly to the soil material on the Kreeger property, the composite horizontal permeability was assumed to be 3 in/hr. The soil material encountered in the bottom of each of the borings effectively serves as an aquitard, because its permeability is about 300 times less permeable, assuming that its permeability is similar to the soil material tested from boring 6. The silty fine sand encountered at depths of 21 to 26 feet in boring 4 and tested in the laboratory would contribute little to horizontal flow but can transmit water vertically at 3 times the application coefficient. Thus, the medium sand at depths of 26 to 32.5 feet could contribute to horizontal flow.

The application of water in a drainfield was modeled by means of a simplified analysis. The Hooghoudt ellipse equation modified for a sloping aquitard was used for the analysis. The analysis presumes a long, rectangular application area paralleling a drainage ditch. The form of the equation utilized is as follows:

$$L^2 + 2LS\,\bar{y}\,\frac{K}{q} + y1^2\,\frac{K}{q} - y0^2\,\frac{K}{q} = 0$$

where L, K, and q are as previously defined and

y0 = allowable height of saturated material above the sloping aquitard, ft

y1 = height of the bottom of the drainage ditch above the aquitard, ft

$\bar{y}$ = the difference between y0 and y1, ft

S = the average slope of the aquitard and land surface, ft/ft

Values derived from Figure III-B-12 are as follows:

y0 = 17 ft

y1 = 0.5 ft

$\bar{y}$ = 16.5 ft

S = 0.0345 ft/ft

The application coefficient was assumed the same as for the Kreeger property. The calculated result is a distance of 460 feet from the bottom of the escarpment to the far edge of the application area. This distance is applicable to the area where the depth of permeable material is about 27 feet deep. Thus, the area near borings 5 and 6 would be excluded. The area toward the north, the proposed site of the landowner's residence appears to offer the greatest potential. This site would perform well as a cluster drainfield site.

The applied water would percolate to the aquitard and then move downslope to the face of the escarpment where the present groundwater surfaces. The groundwater table below the escarpment is presently at or near the surface. This water reenters the groundwater system or is collected by a surface drain near Artesian Lane and both flow toward Pickerel Lake. The additional water in this system would not appreciably affect this drainage system.

The conclusions concerning the effects of the potential drainfield on the environment for the Keller property are similar to those for the Kreeger property. Nitrate is the nutrient of concern at this site also. The wells in this area, though, are cased through a thick layer of clay. Potential pollution of these wells is essentially nonexistent. Also, because the water table below the escarpment is at or near the soil surface, uptake and denitrification could be expected to account for greater losses. The surface drains could also be expected to intercept and convey a sizable proportion of the nitrate-enriched water to the lake. The effect of the nutrients from the drainfield on eutrophication in the lake is expected to be negligible.

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Alfred, S.D, A.G. Hyde, and R.L. Larson. 1973. Soil survey of Emmet County, Michigan. U.S. Department of Agriculture, Soil Conservation Service, in cooperation with Michigan Agricultural Experiment Station.

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APPENDIX A

DRILLER'S SOIL BORING LOGS*

*Correct boring numbers for the Keller Property are listed to the right of each incorrect boring number, indicated as incorrect by notation.

GEO-TEK INC.
SUBSURFACE EXPLORATION
P O Box 116 Lowell, Mich. 49331 • Bus Phone: (616) 897-5581

Project Kreeger Property Job No. _____
Location Emmet County Permit No. _____

LOG OF BORING

Well Set At 47'

Boring Number 1	Crew Chief Qualls	Ground Water:	Plugging Record		
Depth Drilled 50.5' ft.	Helper Klitz	Encountered at 25'± ft.	Boring Sealed with: Ex. Soil		
Surface Elev. ____ ft.	Drill Rig 45	After Completion 23.5" ft.	____ between 0 ft. & 50.5 ft.		
Date Started 10/20/80	Boring Method HSA	After 24 hrs: ____ ft.	____ between ____ ft. & ____ ft		
Date Completed 10/20/80		Volume ____	____ between ____ ft. & ____ ft.		
		Seepage at ____ ft.	____ between ____ ft. & ____ ft.		

Depth In Feet	Sample Type	Penetration ASTM D 1586	SOIL DESCRIPTION
6"			Topsoil- Dark-brown, sand loam.
1.0'		1-2-2	Sand- Loose, dark-brown, med.
2.5'		2-2-3	Sand- Compact, dark-brown, med., with a slight binder.
		4-3-4	
		3-4-5	Sand- Compact, brown, fine to med.
		5-4-5	
		5-5-6	
		4-4-5	
		4-5-4	
		3-4-4	
		3-3-3	
		3-3-3	
		3-3-4	
		3-3-4	
		3-3-3	
22.0'			
23.0'		2-3-4	
	3"S.T.		Sand- Compact, brown, fine, silty, with slight binder.
		2-3-4	Sand- Compact, brown, fine to med., with trace of pebbles.
		2-4-5	
		5-5-9	
		5-5-6	
31.5'			
		5-5-6	
		3-3-5	Sand- Compact, brown, fine to med., with occ. lense of silty sand, with slight binder.
		2-3-6	
	SNR		
		6-6-7	
		5-5-7	
		2-3-6	

GEO-TEK INC.
SUBSURFACE EXPLORATION
P O Box 116 Lowell, Mich 49331 • Bus. Phone: (616) 897-5581

Project Kreeger Property Job No. _____
Location Emmet County Permit No. _____

LOG OF BORING

Boring Number 1 | Crew Chief Qualls | Ground Water: Encountered at 25' ± ft. | Plugging Record: Boring Sealed with: Ex. Soil
Depth Drilled 50.5 ft. | Helper Klitz | After Completion 23.5" ft. | ____ between 0 ft. & 50.5 ft.
Surface Elev. ____ ft. | Drill Rig 45 | After 24 hrs: ____ ft. | ____ between ____ ft. & ____ ft.
Date Started 10/20/80 | Boring Method HSA | Volume ____ | ____ between ____ ft. & ____ ft.
Date Completed 10/20/80 | ____ | Seepage at ____ ft. | ____ between ____ ft. & ____ ft.

Depth In Feet	Sample Type	Penetration ASTM D 1586	SOIL DESCRIPTION
41.0'		5-7-12	
		8-10-15	Sand- Very compact, brown, fine, with a trace of pebbles and occ. silty sand lenses.
		7-7-8	
		5-4-8	
48.0'		3-5-10	
		5-5-9	
50.5'		5-3-4	Sand- Compact, brown, very fine, very silty.
	EOB		

GEO-TEK INC.
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Project Kreeger Property Job No. _____
Location Emmet County Permit No. _____

LOG OF BORING

Well Installed At 40'

Boring Number 2	Crew Chief Qualls	Ground Water: Encountered at 26.5'± ft.	Plugging Record Boring Sealed with: Ex. Soil
Depth Drilled 40 ft.	Helper Klitz	After Completion 25'2" ft.	____ between 0 ft. & 40 ft.
Surface Elev. ____ ft.	Drill Rig 45	After 25'2" hrs; 24 ft.	____ between ____ ft. & ____ ft.
Date Started 10/20/80	Boring Method HSA	Volume ____	____ between ____ ft. & ____ ft.
Date Completed 10/20/80	____	Seepage at ____ ft.	____ between ____ ft. & ____ ft.

Depth In Feet	Sample Type	Penetration ASTM D 1586	SOIL DESCRIPTION
			Sand- Damp, dark-brown, med., with a layer of brown very sandy clay.
6.0'			
			Sand- Damp, light-brown, fine, to med.
26.5'			
			Sand- Wet, brown, fine to med., with occ. lenses of fine, silty, sand.
40.0'			EOB

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LOG OF BORING

Boring Number 3 | Crew Chief Qualls | Ground Water: Encountered at 13.5'± ft. | Plugging Record: Boring Sealed with: Ex. Soil
Depth Drilled 36 ft. | Helper Klitz | After Completion 9.3" ft. | ____ between 0 ft. & 36 ft.
Surface Elev. ____ ft. | Drill Rig 45 | After 12 hrs: ____ ft. | ____ between ____ ft. & ____ ft.
Date Started 10/20/80 | Boring Method HSA | Volume ____ | ____ between ____ ft. & ____ ft.
Date Completed 10/20/80 | ____ | Seepage at ____ ft. | ____ between ____ ft. & ____ ft.

Depth In Feet	Sample Type	Penetration ASTM D 1586	SOIL DESCRIPTION
1.0'			Topsoil- Dark-brown, sand, loam.
			Sand- Damp, med., shades of brown.
5.0'			
			Sand- Damp, brown, med., occ. layer of med silty sand.
13.5'			
			Sand- Wet, brown, fine, very silty.
25.5'			
			Clay- Moist, brown, very silty, with layers of fine silty sand.
35.0'			
		5-19- 30	
36.0'			
	EOB		Sand- Ext. compact, brown, fine.

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Project Keller Property — Job No. _____
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LOG OF BORING

Boring Number X4 | Crew Chief Qualls | Ground Water: Encountered at 22'± ft. | Plugging Record: Boring Sealed with: Ex. Soil
Depth Drilled 37 ft. | Helper Klitz | After Completion ____ ft. | ____ between 0 ft. & 37 ft.
Surface Elev. ____ ft. | Drill Rig 45 | After ____ hrs: ____ ft. | ____ between ____ ft. & ____ ft.
Date Started 10/21/80 | Boring Method HSA | Volume ____ | ____ between ____ ft. & ____ ft.
Date Completed 10/21/80 | ____ | Seepage at 22'± ft. | ____ between ____ ft. & ____ ft.

Depth In Feet	Sample Type	Penetration ASTM D 1586	SOIL DESCRIPTION
1.0'		1/2-1/2-2	Topsoil- Black, sand, loam.
		1-1-1	
		2-2-3	Sand- Loose, dark-brown, med., trace of organics.
6.0'		4-4-6	
		5-7-6	Sand- Compact, light-brown, med.
		4-4-5	
		3-2-3	
12.0'		2-2-2	
		2-2-2	Sand- Compact, brown, med to coarse, with layers of fine sand.
		2-2-5	
17.5'		4-5-4	
		4-4-7	
		5-5-7	Sand- Compact, brown, very fine, silty.
		5-7-8	
		6-7-9	
	3"S.T.	6-4-6	
26.5'			
		11-18-22	Sand- Ext. compact, brown, med to coarse.
		11-14-19	
31.0'		12-18-26	
32.5'		26-24-24	Sand- Ext. compact, brown, med to coarse, with some fine to med. gravel.
34.0'		11-11-9	Sand- Silt and clay layers, compact, brown, med sand, silty clay.
		6-12-18	
37.0'		6-14-16	Clay- Ext. stiff, brown, occ. lense of silt and gravel.
	EOB		

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LOG OF BORING

Boring Number 25 | Crew Chief Qualls | Ground Water: Encountered at None ft. | Plugging Record: Boring Sealed with: Ex. Soil
Depth Drilled 27 ft | Helper Klitz | After Completion _____ ft. | _____ between 0 ft. & 27 ft
Surface Elev. _____ ft. | Drill Rig 45 | After _____ hrs: _____ ft. | _____ between _____ ft. & _____ ft
Date Started 10/21/80 | Boring Method HSA | Volume _____ | _____ between _____ ft. & _____ ft
Date Completed 10/21/80 | _____ | Seepage at _____ ft. | _____ between _____ ft. & _____ ft

Depth In Feet	Sample Type	Penetration ASTM D 1586	SOIL DESCRIPTION
			Sand- Damp, dark-brown, med.
6.0'			
3" Tube attempt 8'		10-15-18	Sand- Damp, light-brown, med., occ. stone.
		11-20-22	
25.0'			
27.0'		12-20-26	Clay- Ext. stiff, brown, silty.
	EOB		

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Project Keller Property Job No. _____
Location Emmet County Permit No. _____

LOG OF BORING

Boring Number 3 C
Depth Drilled 19.5' ft.
Surface Elev. _____ ft.
Date Started 10/21/80
Date Completed 10/21/80

Crew Chief Qualls
Helper Klitz
Drill Rig 45
Boring Method HSA

Ground Water:
Encountered at None ft.
After Completion _____ ft.
After _____ hrs; _____ ft.
Volume _____
Seepage at 17' top of clay ft.

Plugging Record
Boring Sealed with: Ex. Soil
_____ between 0 ft. & 19.5 ft.
_____ between _____ ft. & _____ ft.
_____ between _____ ft. & _____ ft.
_____ between _____ ft. & _____ ft.

Depth In Feet	Sample Type	Penetration ASTM D 1586	SOIL DESCRIPTION
6"			Topsoil- Dark-brown, sand loam.
			Sand- Damp, brown, med.
	3"S.T. no recovery		
10.5'			
	3"S.T. no recovery		Sand- Very compact, brown, very fine, with layers of coarse sand.
	3"S.T.	5-8-11	
18.0'			
		12-20-27	Clay- Ext. stiff, gray, very sandy, with pebbles and occ. stones.
19.5'			
	EOB		

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LOG OF BORING

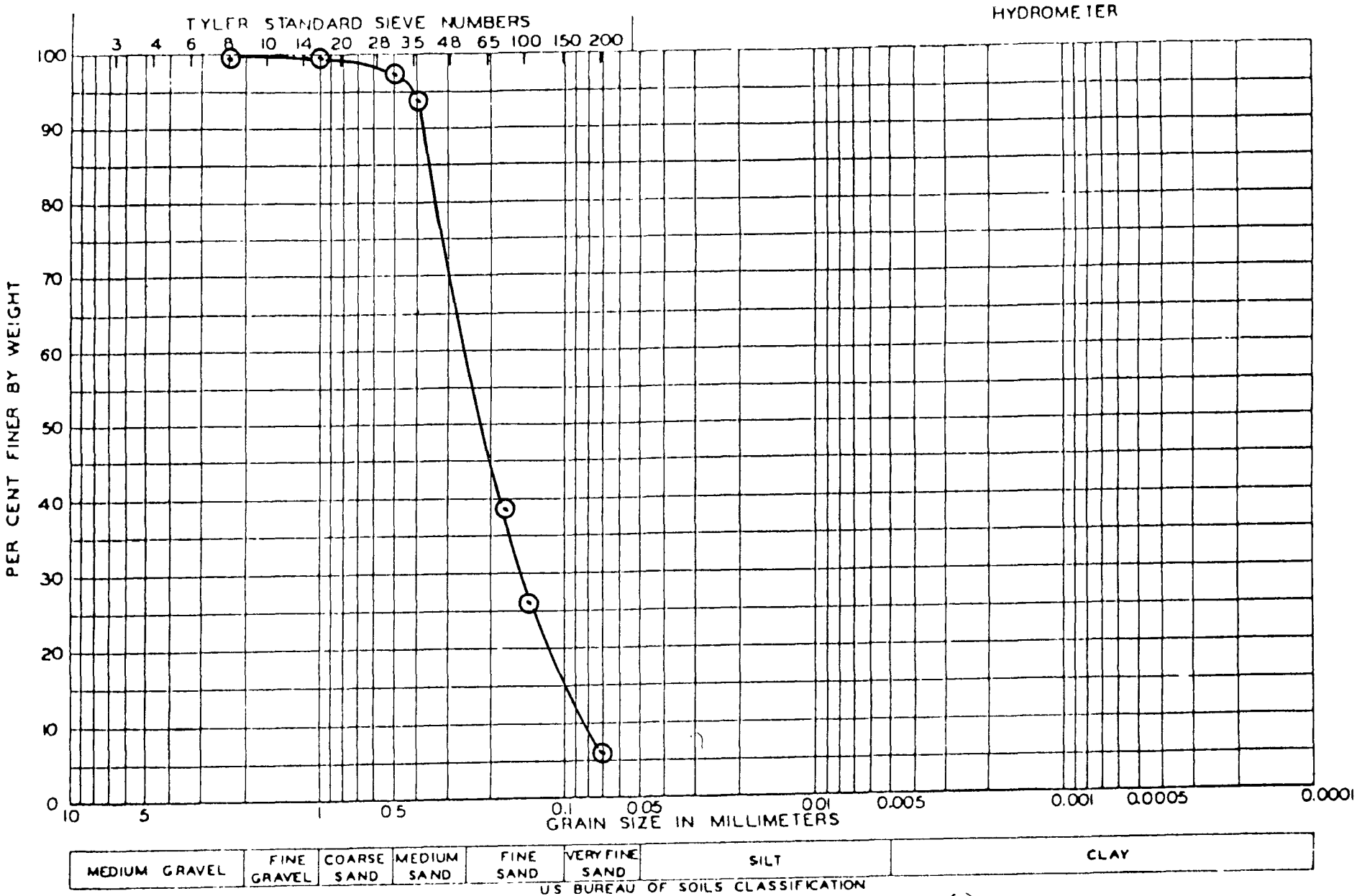
Well Set At 20'

Boring Number 47	Crew Chief Qualls	Ground Water:	Plugging Record		
Depth Drilled 21.5' ft.	Helper Klitz	Encountered at 14'± ft.	Boring Sealed with: Ex. Soil		
Surface Elev. ____ ft	Drill Rig 45	After Completion ____ ft.	____ between 0 ft. & 21.5 ft.		
Date Started 10/21/80	Boring Method HSA	After ____ hrs: ____ ft.	____ between ____ ft. & ____ ft.		
Date Completed 10/21/80		Volume ____	____ between ____ ft. & ____ ft.		
		Seepage at ____ ft.	____ between ____ ft. & ____ ft.		

Depth In Feet	Sample Type	Penetration ASTM D 1586	SOIL DESCRIPTION
			Sand- Damp, brown, med.
	3" S.T.		
13.5'			
		2-3-2	Sand- Wet, brown, med and fine, layers, trace of silt.
		2-2-3	
21.5'			
	EOB		

APPENDIX B

SOILS LABORATORY TESTING RESULTS



PROJECT Geo-Tek Kreeger Property BORING NO 3(4) SAMPLE NO 81616

DEPTH 10' ELEVATION ____ REMARKS fine to med. sand

Orig. Wet Density - 103.3 p.c.f.
Orig. Moisture - 6.4%
Dry Density - 97.1 p.c.f.

GRAIN SIZE DISTRIBUTION DIAGRAM

Sp. Gr. 2.644

Void Ratio - 0.699

Undisturbed Perm. Entire Sample = 1.4×10^{-3} cm/sec

Materials Testing Consultants, Inc.

Nov. 13, 1980

Job No. 235D

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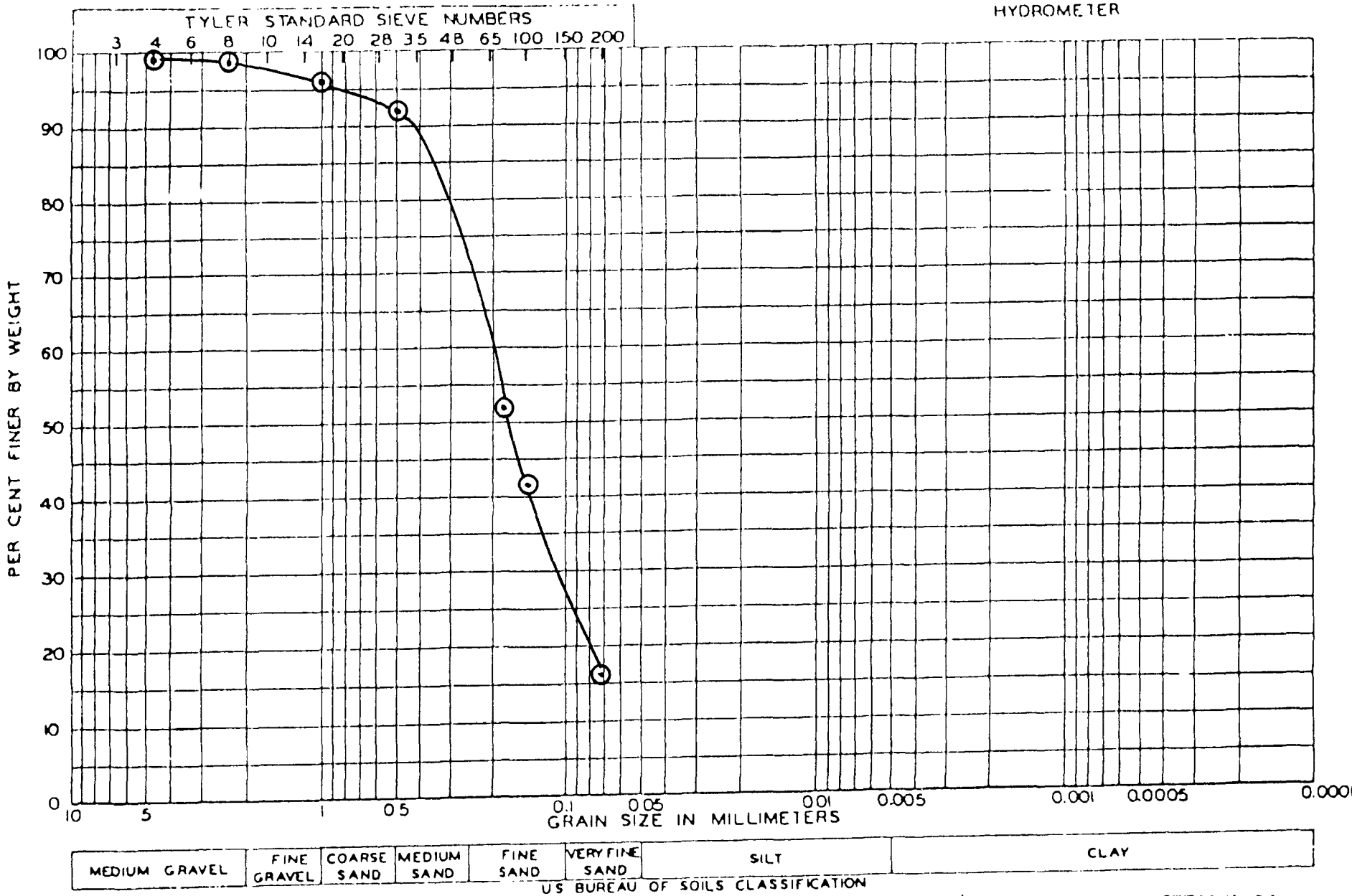

PROJECT Geo-Tek Keller Property BORING NO 4(1) SAMPLE NO [illegible] 14

Orig. Moisture 12.4% DEPTH 25' ELEVATION ____ REMARKS various fine sands to fine sands with some silt

Permeability 2.1×10^{-5} cm/sec

Wet Density - 128.9 p.c.f.
Dry Density - 114.7 p.c.f. Sp. Gr. 2.680

GRAIN SIZE DISTRIBUTION DIAGRAM

Void Ratio 0.46

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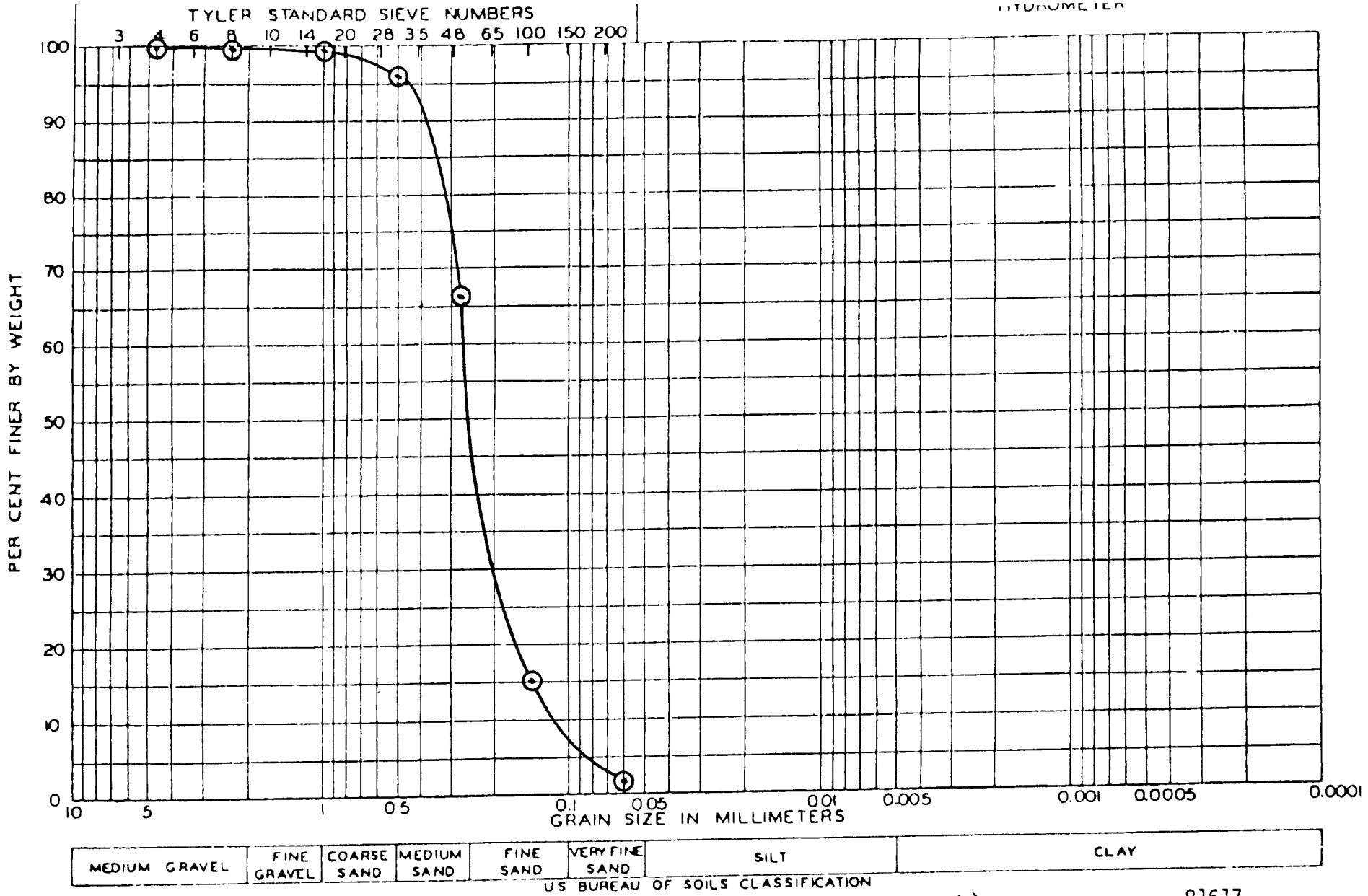


PROJECT Geo-Tek Keller Property BORING NO 7(4) SAMPLE NO 81617

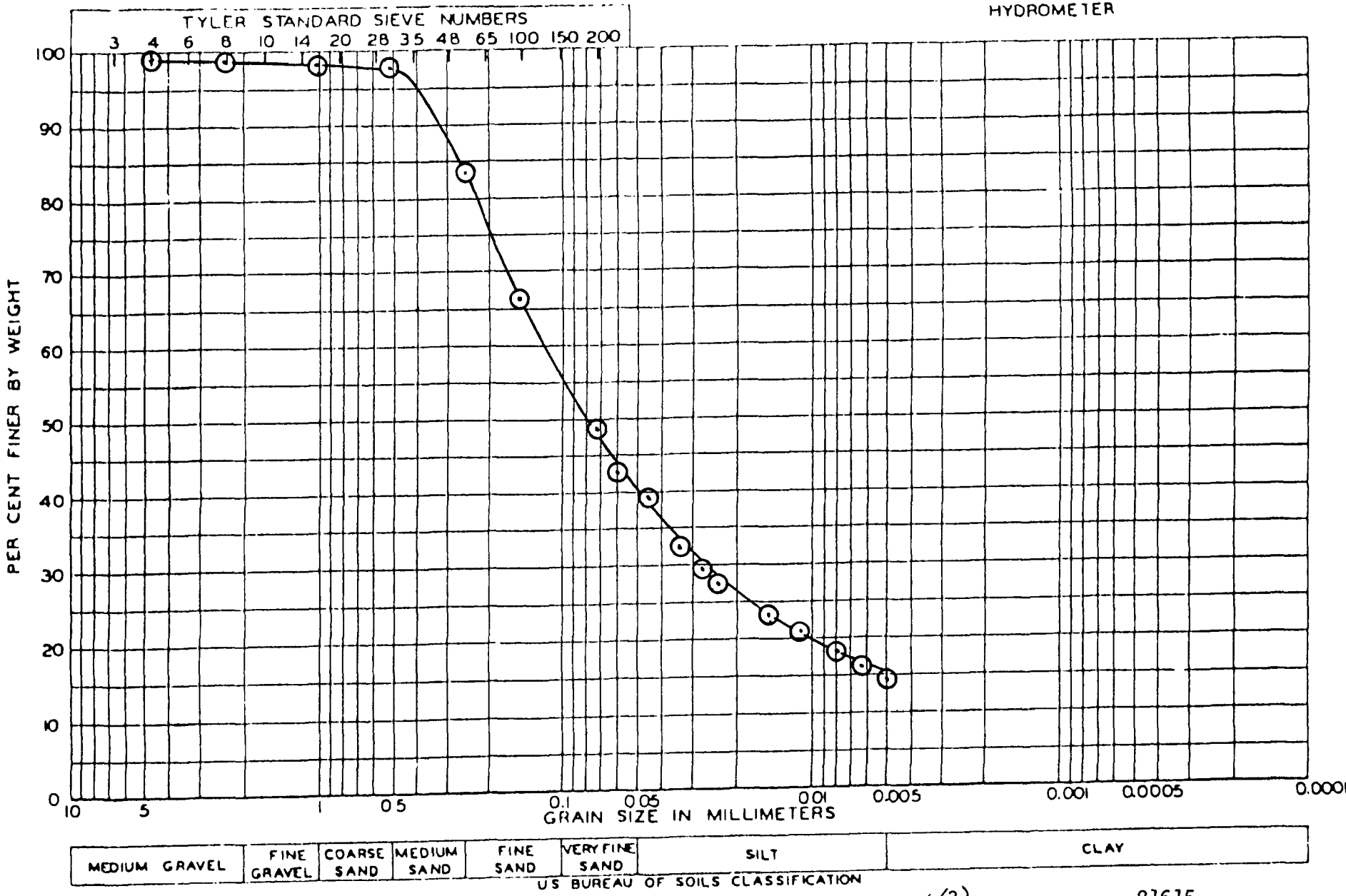
DEPTH ~~5~~ 10 ELEVATION _____ REMARKS Fine Sand

Orig. Wet Density - 106.9 p.c.f.
Orig. Moisture - 8.1% Dry Density - 98.9 p.c.f.

GRAIN SIZE DISTRIBUTION DIAGRAM

Undisturbed Permeability
Entire Sample 4.1×10^{-3} cm/sec
Void Ratio 0.668

Materials Testing Consultants, Inc. Sp. Gr. 2.644 Nov. 13, 1980 Job No. 235D



PROJECT Geo-Tek Keller Property BORING NO 6(3) SAMPLE NO 81615

DEPTH 14' ELEVATION ____ REMARKS Various soils from clean fine sand to silty clay

Orig. Wet Density 130.1 p.c.f.
Orig. Moisture - 20.2%
Dry Density - 108.2 p.c.f. Sp. Gr. - 2.695

Undisturbed Permeability Entire Sample 8.1×10^{-3} cm/sec

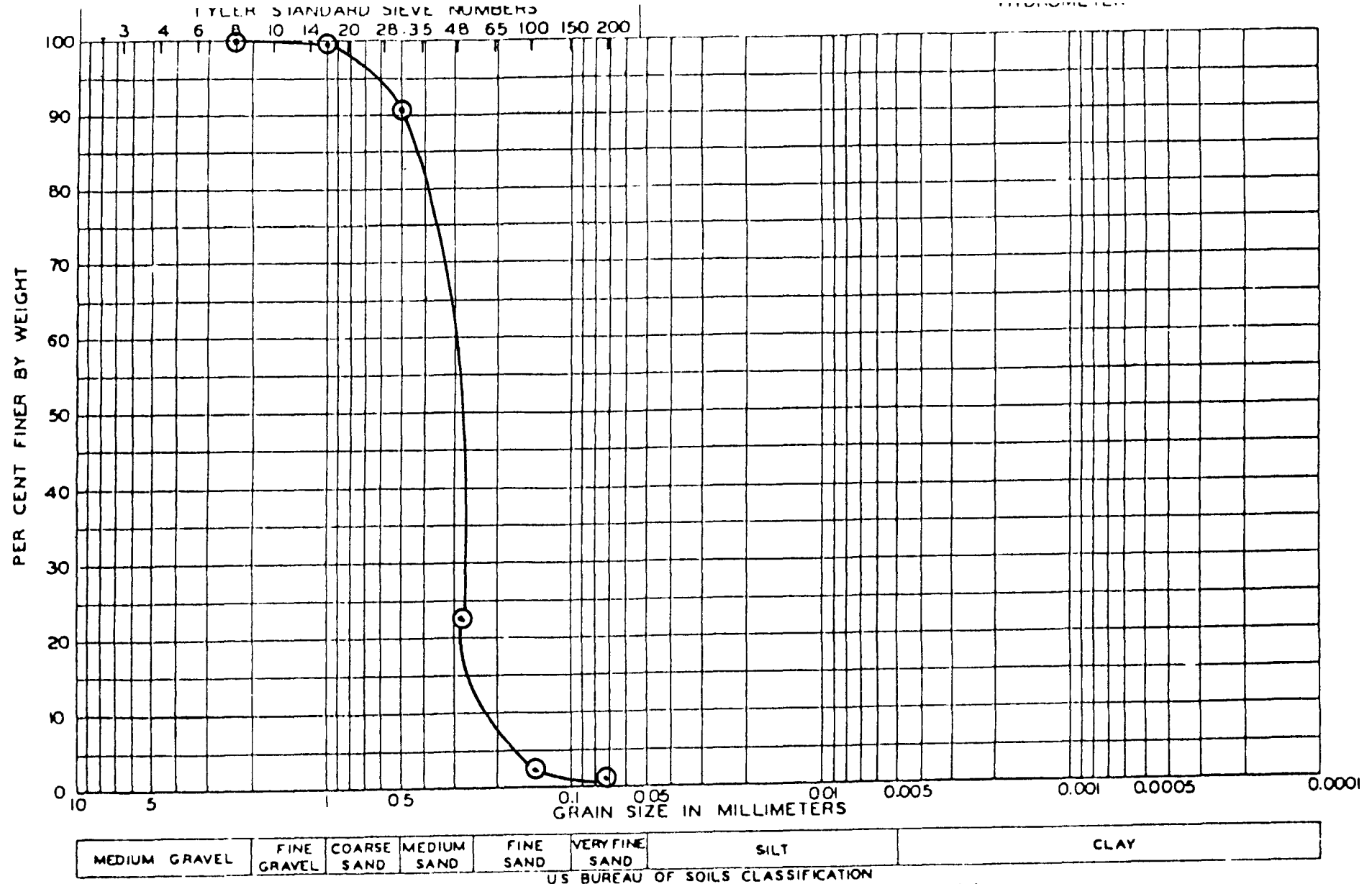
GRAIN SIZE DISTRIBUTION DIAGRAM

Void Ratio - 0.55

Materials Testing Consultants, Inc. Nov. 13, 1980

235D

T. Riordan



Orig. Wet Density - 116.8 p.c.f.
Orig. Moisture - 3.9%
Dry Density - 112.4 p.c.f.

PROJECT Geo-Tek Keller Property
BORING NO 5(2)
SAMPLE NO 81618
DEPTH 11'
ELEVATION
REMARKS Fine sand

GRAIN SIZE DISTRIBUTION DIAGRAM

Sp. Gr. - 2.631
Void Ratio = 0.461

Undisturbed Perm.
Entire Sample - 7.8×10^{-3} cm/sec

Materials Testing Consultants, Inc.
Nov. 13, 1980
Job No. 235D

CHAPTER IV

COST ANALYSIS

A. COST VARIABILITY STUDY

1. INTRODUCTION

The selection, design, and costs of wastewater management technologies in rural areas are significantly affected by local environmental and developmental characteristics. Comparison of alternative wastewater management strategies developed by U.S. EPA during preparation of the Seven Rural Lake EIS's suggested some intuitive relationships between the costs of certain technologies for a particular community, and local factors such as topography, soil and hydrogeological conditions, growth patterns, and density of residential development.

It behooves the sanitary engineer or small waste flows specialist to be cognizant of these relationships as early as possible in the facilities planning process in order to optimize local capital expenditures for new or improved wastewater management technologies and to expedite the facilities planning process. Technology selection that reflects variable environmental and developmental conditions within a community, and is, itself, a direct result of such variability, is preferable and potentially more cost-effective than a selection process that is based solely upon general assumptions regarding these conditions. For example, conventional engineering wisdom suggests that pressure sewers, if feasible, are more cost-effective than gravity sewers for wastewater collection in flat areas. It is also generally known that on-site wastewater management systems are more cost-effective than centralized collection, transmission, and treatment facilities in low density residential areas. But how flat must the topography be in order to optimize the cost-effectiveness of pressure sewers over gravity sewers? How low does the housing density have to be in rural residential areas for on-site wastewater management to be the technology of choice? At what density is it cost-effective to substitute the use of one technology or set of technologies for another? These questions can be addressed through quantification and analysis of the relationships between wastewater management technologies, cost, site conditions and developmental characteristics.

Clearly, under natural conditions, the topography of a particular wastewater management district may not be homogeneously flat and its population may not be uniformly of low density. Environmental and developmental factors work in tandem to produce opportunities for trade-offs between competing technologies to be exploited in the interest of minimizing capital outlays at the local level

a. Objectives

This study of facilities' costs as they are affected by environmental and developmental conditions has three primary objectives:

1) quantify the effects of environmental and developmental variables on the costs of certain wastewater facilities likely to be considered for rural lake communities;

2) compare the ranges of facility costs in order to identify thresholds of cost preference for competing technologies as they are affected by the environmental and developmental variables;

3) identify those environmental and developmental variables that are most critical to the conclusions of cost-effectiveness comparisons between competing technologies.

A secondary objective of this study is to provide data in support of other research tasks described in this Technical Reference Document including:

- Description of economic and environmental justification for variance procedures.
- Effects of variance procedures on SWF agency design, manpower, and costs.
- Review of jurisdictional, environmental, and developmental approaches to defining planning area boundaries.
- Evaluation of segmentation in SWF planning.

b. Major Variables Affecting Wastewater Management Facilities Costs

Variables expected to significantly affect the costs of wastewater management facilities in rural areas include:

- Environmental constraints
  - flat, steep, or rough topography
  - depth to groundwater
  - depth to bedrock
  - soil stability in cuts greater than five feet
  - soil permeability
  - distance to sites suitable for small scale treatment
- Developmental constraints
  - rate of growth
  - housing density
  - house setback from road
- Technology selection
  - large-scale off-site technologies
  - small-scale off-site technologies
  - on-site technologies

c. Uses and Limitations of Cost Variability Study Data

Data developed during the cost variability study, in conjunction with local knowledge of such factors as topography, soil conditions, and housing density, may serve the pre-Step 1 or early Step 1 planning needs of wastewater management officials. Local officials may employ cost variability data to:

- determine the preliminary economic feasibility of sewers,
- define preliminary service area boundaries for various off-site and on-site technologies,

TABLE IV-A-1. FACILITIES FOR WHICH COST RANGES WERE DEVELOPED IN COST VARIABILITY STUDY

Collection facilities	Transmission & centralized treatment facilities	Small scale treatment facilities	On-site systems
• Conventional gravity sewers (≦ 8 inch diameter)	• Facilities developed in Seven Rural Lake EISs*	• Subsurface treatment for average site conditions	• Septic tanks
			• Drainfields for optimal site conditions
• Small diameter (4 inch gravity sewers with septic tanks		• Spray irrigation for average site conditions	• Drainfields for average site conditions
• Low pressure sewers with septic tank effluent pumps (STEP) serving 1, 2, or 3 houses		• Rapid infiltration for average site conditions	• Drainfields for marginal site conditions
			• Dosing units
• Low pressure sewers with grinder pumps serving 1, 2, or 3 houses		• Overland flow for average site conditions	• Alternating drainfields
			• Mounds
			• Holding tanks
			• Maximum flow reduction for bathroom wastewaters
			• Site specific evaluation

* Range in capital and operation and maintenance dollars per design year household obtained from Seven Lake cost analyses.

- generate preliminary alternatives, and

- conduct preliminary cost-effectiveness analyses of pre-Step 1 or early Step 1 alternatives.

The use of cost variability data to assist local facilities planning efforts is optimized through completion of a two-phase process:

Phase I--A first-cut preliminary determination of centralized sewerage facilities' feasibility is made by determining housing density (houses per mile of sewer) within potential service areas of the community. This determination will indicate whether sewers are at all feasible. On the basis of population data alone, high density areas are generally more cost-effectively served by centralized facilities than by small-scale off-site facilities or on-site facilities. The converse is generally true for low density areas, again on the basis of population data alone. The cost-effectiveness criteria used to make preliminary decisions of sewerage facilities' feasibility during this phase are provided by cost tables and cost curves presented here. The end result of Phase I efforts is a breakdown of those segments within a community that could be cost-effectively served by centralized facilities and those that could be more cost-effectively served by other technologies.

Phase II--With additional information about the segmented residential areas delineated in Phase I, efforts during Phase II could indicate the necessary combinations of technologies to cost-effectively satisfy the wastewater management needs of the entire community. Again, tabulated and graphed cost variability data would have to be consulted. Using available information on the number of malfunctions, housing age, lot size, topography and groundwater, and soil conditions on a segment-by-segment basis, it is possible to develop preliminary cost-effective combinations of technologies that are reasonably feasible in light of local environmental and developmental characteristics.

Data and conclusions from this study will be useful in preparation of Plans of Study for Step 1 applications, in delineating centralized and small waste flows service areas on a preliminary basis and in screening preliminary alternatives. However, cost comparisons of final alternatives should be based on locally applicable data for unit costs and necessary technologies.

2. METHODOLOGY

The methodology used to quantify and evaluate the effects of representative environmental and developmental variables on the costs of selected wastewater management facilities (see Table IV-A-1) is described in this section.

This methodology involves completion of the following steps:

Step 1: Select density range
Step 2: Establish growth variables
Step 3: Identify environmental constraints
Step 4: Combine environmental constraints into scenarios

Step 5: Select wastewater management technologies
Step 6: Compile unit costs
Step 7: Estimate present worth per house costs of wastewater technologies
Step 8: Tally wastewater collection, transmission and treatment units necessary for technologies under variable growth rates, densities and scenarios
Step 9: Combine unit present worth costs
Step 10: Prepare per house present worth data in tabular form
Step 11: Prepare per house present worth data in graphic form
Step 12: Analyze data

a. Step 1: Select Density Range

Critical among the developmental variables affecting the per residence cost of wastewater management in rural areas is the density of homes to be served. In low density development areas, conventional sewerage facilities tend to be significantly more expensive than alternative collection systems and on-site systems. The first step in this cost variability study methodology is the selection of a range of housing densities for which cost completion among wastewater technologies may be evaluated. With the focus on wastewater management in rural areas, the extent of housing densities evaluated reflects the range of lot sizes within which on-site systems are considered feasible.

For the sake of analysis, density is defined as the number of houses that could connect directly to one mile of collector sewer. In many rural settings, the collector sewer would be constructed within road right-of-ways so the length of sewer can be approximated by length of road. The range of housing densities evaluated varies depending on the growth rate under consideration (see Section 2b). Under an assumption of no growth (i.e., 0%), housing densities include 25, 50, 75 and 100 houses per mile of sewer or roadway. This range shifts upward to 38, 75, 113, and 150 houses per mile of sewer or roadway under an assumption of future growth (i.e., 50%).

The primary determinant of housing density is lot size. The range of housing densities used in this study is 25-100 houses per mile of sewer or roadway. At the lowest end of the density range, 25 houses per mile, average lot sizes would vary from approximately 1 acre to 8 acres. At or below a density of 25 houses per mile, lot sizes are sufficiently large to accommodate any number of on-site system configurations. At this density, on-site systems are considered quite feasible and competitive with all alternative small- and large-scale technologies. Therefore, an analysis of trade-offs between on-site systems and competing technologies under varied environmental and developmental conditions is essential in order to optimize community-wide expenditures for wastewater management facilities.

At the highest end of the density range, 100 houses per mile, lot sizes are considerably smaller than they would be at a density of 25 houses per mile. Assuming deep lots (i.e., lot depth equals 2 times lot width), on either side of a road, average lot size could be as large as 0.5 acre. At a density of 100 houses per mile, the competitiveness of on-site systems among available wastewater technologies decreases considerably. Higher on-site system implementation costs are attributed to the need for sanitary surveys,

community-wide repairs, and special design provisions for very small lots in response to existing environmental constraints. In spite of these special provisions, on-site systems at a density of 100 houses per mile are within the competitive range in which trade-off analyses are warranted.

b. Step 2: Establish Growth Variables

Another important factor affecting the costs of competing wastewater management technologies in rural areas is the rate at which development is expected to occur during the planning period. Again, parameters have been selected that typify extremes under which on-site systems could be considered competitive. In this case, two growth figures have been used: no growth, a condition under which 0% growth occurs over the 20-year planning period; and future growth, a condition under which 50% growth occurs over the 20-year planning period.

Zero percent was selected as an extreme low growth condition; a growth rate of 50% was selected as a reasonable high end assumption since community growth up to and including 50% is an acceptable criterion when determining eligibility of collector sewers under the U.S. EPA Construction Grants Program.

Any significant changes in trade-offs that might occur between two or more wastewater management technologies under alternative growth conditions (0% and 50%) will be examined. For example, under optimum environmental conditions*, centralized collection, transmission, and treatment facilities may not be competitive with on-site systems under a "no growth" scenario at densities less than 100 houses per mile of sewer or road. Under "future growth" conditions, cost competitiveness is achieved at approximately 90 houses per mile.

c. Step 3: Identify Environmental Contraints

Unit costs for installing sewage collection and conveyance facilities around rural lakes can vary significantly, due to topography, soil material, frost depth, depth to water table, road construction and residential setback distances, and elevation differences. Physical or environmental constraints most likely encountered by facilities planners and contractors, and the extent to which these constraints may affect total unit costs are discussed below.

Topography. Topography in the glaciated rural lake areas of Region V can be expected to vary locally. Wastewater management officials incorporate this variation into the technology selection and design process, providing the appropriate mixture of conventional and alternative sewage collection systems as required by the rise and fall of the land. Undulating or flat terrain generally calls for provision of pump stations, pressure sewers and force mains to convey wastewater to treatment systems. If the terrain is uniformly down sloping, this function can be served by gravity sewers.

Alternatively, steep gravity cuts through hilly terrain would be required to achieve the same objective. Both approaches impart significant construction and operation costs to system users.

---

* Average depth of sewer cut = 8 ft.; no groundwater problems; no rock excavation.

Design responses to variable topography on individual lots are analogous to those deemed appropriate for communities or segments of communities.

Design of Excavation. The sewage collection facilities within the rights-of-way are most affected by depth of excavation. Up to a depth of about 15-feet sewers can be laid by use of a trench box to meet safety requirements and to minimize widths of excavation. When solid sheeting and bracing become necessary, primarily in deep excavations with noncohesive soils, costs increase with the square of the depth. Thus, where sewage flows are small, pumping is often less costly than placing deep sewers. Sewer construction is usually least expensive in stiff clays because the trench walls stand up well with minimal bracing, and groundwater is rarely costly to remove in clay soils. Sands can be very expensive for sewer construction because they tend to flow, which means that full sheeting and extensive bracing would be necessary to restrain the earth pressures.

A factor that would affect the lineal foot cost of pressure sewers is frost penetration depth. One of the assumptions of this study is that a depth of cover of five feet would be maintained, corresponding to maximum frost penetration depth. At this depth, a backhoe must be used for excavation. In cases where the frost penetration depth is less, or where warm sewage will prevent freezing, a trencher may be utilized. Trenchers, though, are cost-effective only where underground obstructions are spaced no closer than 50 feet.

Groundwater. In sandy soils, groundwater can be very extensive. Sand becomes "quicksand" when the groundwater flow is sufficiently extensive to cause bouyancy to the sand particles. Once excavation proceeds into the water table more than ½ to 3 feet, dewatering by means of wellpoints or wells becomes necessary. Sands with fine to very fine textures are difficult to dewater without pumping sand and causing subsidence. In course sands and gravel, extremely large volumes of water need to be pumped, which results in increased costs; however, well spacing may be greater. Dewatering wells are generally less expensive than wellpoints, and the well spacing and depth are the principal determinants of cost. A slowly permeable layer underlying the sewer invert can double dewatering costs because two lines of dewatering wells would be necessary where one would normally suffice. Organic soils and lacustrine clays lack the stability necessary for maintaining grade on the sewer line and for protecting the installed pipe. In these soils the trench must be overexcavated, and special bedding and backfill placed. Depending on the cost of this special material, the lineal foot cost of installing sewers may be increased a minimal amount to greater than $10 per lineal foot.

Rock Excavation. Rock excavation can be another highly variable expense. If the rock can be ripped, the unit cost and the lineal foot cost would be significantly less than the unit cost used in this study. Also, if the rock were to be excavated less than one foot in depth, the unit cost might be significantly higher than that used in this study. Costs based on the number of holes to be drilled, primed, and shot vary more than costs based on the depth of the holes and charge size. Rock can vary significantly in its blast characteristics: some rock is "tough," which means it absorbs the blast and does not fracture cleanly; and some rock is brittle with many existing fracture lines and thus may fracture cleanly and easily.

d. Step 4: Combine Environmental Constraints into Scenarios

In this step, U.S. EPA has systematically incorporated representative physical/environmental constraints (discussed in Section 2c), or combinations thereof, into 8 "scenarios." These scenarios depict those physical constraints that are likely to be encountered and that were expected to significantly affect costs. For pre-Step 1 or early Step 1 facilities planning purposes, these scenarios provide local officials with sufficient framework within which the preliminary economic feasibility of sewers may be assessed, and preliminary alternatives may be developed in light of varying environmental conditions.

The scenarios account for the following potential physical constraints:

- topography,
- average depth to groundwater,
- average depth to bedrock, and
- unstable soil.

Topography. For purposes of this study, topographical constraints are classified as shown in Table IV-A-2.

Average Depth to Bedrock. In two scenarios with steep and rough topography, bedrock is encountered at 2 feet (Scenario 8) and 6 feet (Scenario 6).

TABLE IV-A-2. EFFECT OF TOPOGRAPHY ON SEWERAGE SYSTEM DESIGN

Classification	Technology design response	Scenario
Flat	Minimim grade requirements for conventional gravity sewers may necessitate pump stations dependent on costs of dealing with groundwater conditions. Small diameter (4 inch) effluent sewers may offer a cost advantage over conventional gravity sewers because of comparably shallower grade allowed.	4, 5, and 7
Optimum*	Assumes 8 ft. average depth of cut (a.d.c.), with minimum of 6 ft. cut.	1
Steep	Sewer transects steep hills which requires cuts of up to 30 ft., with 16 ft. a.d.c.	2
Rough	Maximum depth of cut exceeds 30 ft., requiring one pump station and force main per mile of collector sewer.	3, 6, and 8

* Optimum for sewers.

Soil Unstable. In Scenario 4, soils are not sufficiently stable in cuts greater than 5 feet. It is assumed in this instance that imported fill is needed to replace 1,000 feet of peat soil.

A scenario composite of environmental conditions most likely to affect the costs of wastewater management facilities in rural areas is shown in Table IV-A-3.

e. Step 5: Select Wastewater Management Technologies

Wastewater management technologies for which cost ranges have been developed in this study are listed in Table IV-A-1. Collection, transmission and treatment systems included among these technologies represent a full range of feasible large-scale and small-scale (including on-site system) responses to the variable environmental and developmental conditions that exist in Region V.* Since the costs of conventional collection facilities may represent as much as 80 percent of the total capital costs for wastewater management systems in rural areas,** careful consideration has been devoted to selection of several technologies that minimize or eliminate the distance over which sewage must be collected and transmitted via interceptors, force mains and/or pump stations to treatment systems. Large- and small-scale technologies will become increasingly complex as more environmental constraints have to be neutralized.

Included among the systems evaluated in this study are the on-site technologies selected for various scenarios at 10%, 20%, and 50% replacement levels (Table IV-A-4). These replacement levels were developed in accordance with assumed failure rates for existing on-site systems of 10%, 20%, and 50%. Technology mixes shown in Table IV-A-4 are increasingly complex from left to right as higher failure rates indicate relatively greater site limitations.

f. Step 6: Compile Unit Costs

With the definition of environmental and developmental variables and selection of feasible rural wastewater management technologies completed, the next step in the methodology initiates the process by which the effects of these variables upon the costs of competing technologies are determined. It involves the compilation of unit costs for large-scale and small-scale wastewater systems from several sources including local contractors, construction guides, state officials, U.S. EPA cost data, and U.S. EPA's Seven Rural Lake EIS's. Extremely high or extremely low unit costs were not compiled. This effort was coordinated with that required to prepare the Technology "Fact Sheets" in Chapter 1 of the Technical Reference Document.

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* The feasibility of these technologies was determined by U.S. EPA during preparation of the Seven Rural Lake EIS's, on the basis of cost-effectiveness, environmental factors and implementation considerations.

** Based on a recent review of approximately 300 facilities plans for rural communities in the United States (U.S. EPA, 1970).

TABLE IV-A-3. PHYSICAL/ENVIRONMENTAL CONSTRAINT SCENARIOS FOR RURAL AREAS

Scenario	Topographic variable	Average depth to groundwater	Average depth to Bedrock	Soil unstable	Other constraints
1	8 ft. a.d.c.[1]	-	-	-	-
2	16 ft. a.d.c.	-	-	-	-
3	8 ft. a.d.c.	-	-	-	1 pump
4	Flat	6 ft.	-	Peat[2]	-
5	Flat	6 ft.	-	-	-
6	Topography necesitates 1 pump	-	6 ft.	-	-
7	Flat	-	-	-	-
8	Topography necessitates 1 pump	-	2 ft.	-	50% of houses are too low relative to sewer & bedrock. Will require short pumps to gravity and small diameter sewers.

[1] a.d.c. = average depth of cut.
[2] Imported fill needed to replace 1,000' of peat soil.

TABLE IV-A-4. ON-SITE SYSTEM TECHNOLOGIES FOR VARIOUS SCENARIOS AT 10%, 20%, and 50% REPLACEMENT LEVELS.

Scenario	10% Replacement	20% Replacement	50% Replacement
(1)	10% new ST/SAS[1] (45 perc)[2] 5% flow reduction (minimum)	10% new ST/SAS (60 perc) 10% oversize 10% flow reduction (minimum) 10% flow reduction (maximum)	15% new ST/SAS 25% over size (120 perc) 8% oversize * dosed 2% holding tank 20% flow red (minimum) 30% flow red (maximum)
(2) (3)	10% new ST/SAS with gravity dose 10% flow reduction (minimum)	10% new ST/SAS with gravity dose (45 perc) 10% as above with alternating fields also 10% flow red (minimum) 10% flow red (maximum)	20% new ST/SAS with gravity dose (45 perc) 30% as above with alternating fields also 20% flow red (minimum) 30% flow red (maximum)
(4) (5)	10% new shallow (30 perc) placement 10% flow reduction (minimum)	20% new shallow (60 perc) placement ST/SAS 10% flow red (minimum) 10% flow red (maximum)	40% shallow (60 perc placement ST/SAS 10% holding tanks 30% flow red (minimum) 20% flow red (maximum)
(6)	10% shallow (30 perc) placement, gravity dosed ST/SAS 10% flow red (minimum)	20% shallow (60 perc) placement, gravity dosed 10% flow red (minimum) 10% flow red (maximum)	40% shallow (60 perc) placement, gravity dosed 10% holding tanks 20% flow red (minimum) 30% flow red (maximum)
(7)	10% pressure dosed 10% maximum flow reduction	20% dosed mounds 20% maximum flow reduction	30% dosed mounds 20% holding tanks 50% maximum flow reduction
(8)	10% gravity dosed oversize mounds, alternate 10% maximum flow reduction 10% curtain drains reduction	10% gravity dosed oversize mounds, alternate 10% curtain drains 10% holding tanks 20% maximum flow reduction	30% gravity dosed oversize mounds, alternate 30% curtain drains 20% holding tanks 50% maximum flow reduction

[1] ST/SAS = Septic tank/soil absorption system.
[2] 45 perc = percolation rate of 45 minutes per inch.

On-Site Systems and Cluster Systems. A total of 25 certified contractors in Region V, identified through contact with 2 local health agencies in each state, were consulted to obtain unit costs (capital and operation and maintenance) for on-site system and cluster system components including septic tanks, piping, and drainfields. Unit costs of other major system components including dosing pumps were extracted from the literature. The majority of unit costs for flow reduction devices included in the on-site system technologies were compiled from manufacturers. Several of these costs were also extracted from the literature. Land costs for on-site and cluster systems were assumed to be $500 per acre, identical to those used for costing of land application systems. Costs for detailed site analyses, which were included among the capital and operation and maintenance costs for on-site systems, were derived from costs for similar analyses recommended by U.S. EPA in the Seven Rural Lake EIS's. All unit costs for on-site and cluster systems were upgraded to July 1980 dollars using the ENR construction cost index of 3260.

Land Application Systems. Unit costs for land application systems, which include collection, transmission, and treatment systems, were compiled from the literature.

Land treatment costs for three different systems (slow rate, rapid infiltration, and overland flow) were estimated by using cost curves presented in "Cost of Land Treatment Systems," U.S. EPA 430/9-75-003, revised September 1979.

The cost data were updated to July 1980 using the following indexes and unit costs.

EPA sewage treatment plant index	$\frac{362.1}{177.5} = 2.04$
EPA sewer index	$\frac{387.4}{192.2} = 1.99$
Industrial material wholesale price index	$\frac{275.6}{120} = 2.3$

Labor cost $15/hr including fringe benefits (U.S. EPA, 1979)

Power cost $0.04/KWH (U.S. EPA, 1979)

Land cost $500/acre

Unit cost for piping, excavation, fittings and sewage collection and transport were obtained from Means, 1979.

Centralized Transmission and Treatment Systems. In order to develop estimates for transmission and treatment facilities in new centralized wastewater systems, cost data were extracted from six of the seven alternatives reports prepared by Arthur Beard Engineers, Inc., for the Seven Lake EIS's. These engineering reports provided separate cost data for treatment and collection facilities. Collection facilities (collector sewers, interceptors, pump or lift stations, force mains, pressure sewers and pressure units) were costed for both present and future construction in the reports.

In order to distinguish between local collector sewers and facilities that serve transmission functions, the collection facilities were divided into three groups and their respective capital, operation and maintenance, and salvage values tabulated as follows:

- Eight-inch gravity sewers, pressure units, house sewers, small pumping stations and pressure sewers located in developed areas were considered to be wholly for collection. Their costs were not included as non-collection costs.

- Facilities located in developed areas but sized larger than the minimum were identified. The incremental capital and operation and maintenance costs above similar but minimum-sized equipment were included as noncollection costs.

- Costs for facilities located outside of developed areas or for those that have transmission as their primary purpose were included as noncollection costs.

Facilities projected for construction after the initial year of operation were not included as noncollection costs since they were all collection sewers or house sewers.

Salvage values were figured by straight line depreciation with structures (all sewers, force mains, and 50% of pumping stations) assumed to have a 50-year useful life, and mechanical equipment (50% of pumping stations) assumed to have a 20-year useful life.

Present worths for treatment and transmission facilities were calculated for a 20-year period at an interest rate of 7-1/8%.

Present worths were inflated to a 1 July 1980 ENR Index of 3,260.

Collection Facilities. Unit costs for the following wastewater collection technologies were extracted from Means, 1979:

- conventional gravity sewers,

- small diameter gravity sewers with septic tanks,

- pressure sewers with septic tank effluent pumps,

- pressure sewers with grinder pumps.

g. Step 7: Estimate Present Worth Per House Costs of Wastewater Technologies

The estimation of present worth per house costs in Step 7 facilitates the identification of trade-off points between large-scale, small-scale and on-site system approaches to rural wastewater management under varying growth, density and site conditions. The procedures followed in the estimation of these costs for selected groups of technologies are described below.

(1) On-Site Systems, Cluster Systems and Land Application Systems

Capital and operation and maintenance unit costs compiled and updated in Step 6 were converted to present worth (pw) unit costs (capital dollars + pw O & M dollars - pw salvage value). The present worth costs for an entire one-mile segment were then converted to a per house present worth by dividing by the number of houses present at the end of the 20-year design period.

Future Growth. In order to calculate costs for future growth, a 50% growth rate was assumed to occur uniformly over the 20-year project life. The costs for future growth are therefore related to density and not failure rates. Two future growth costs were calculated. The first assumes all future systems to be standard 60 minutes per inch (MPI)perc ST/SAS. The second method assumes that a mix of technologies similar to those used in developing costs for individual segments will be installed for future systems. Future costs were calculated by multiplying the present worth of one system constructed per year (for 20 years) by the number of systems added per year. This incremental cost for future growth was then added to the total present worth (TPW) for each one-mile segment to obtain a TPW per segment with 50% growth. Future growth-mixed technology means that the cost of a system assumed to be constructed annually was developed from the costs of the various technologies used for specific segments rather than a standard system (ST/SAS - 60 MPI perc rate). The TPW costs with future growth can be compared to similar costs for sewered alternatives to determine trade-off points for sewering vs. non-sewering at various densities and environmental conditions.

For land application systems, the present worth computations were based on the following:

Planning period	20 years
Discount rate	7.125%
Service life	
Structural components including roads, fencing, and piping	50 years
Mechanical components	20 years
Spray irrigation equipment	30 years

The calculation of present worth unit costs in this step initially served to facilitate subsequent estimation of present worth costs for combinations of collection, transmission and treatment technologies (Step 9).

(2) Centralized Transmission and Treatment Technologies

The estimation of present worth costs per house for centralized wastewater transport and treatment facilities involved the analysis of data describing the relationship between house densities and present worth facilities costs per house. Design and cost data developed during the Seven Rural Lake EIS provided the basis for this analysis from which July 1980 per house present worth values will be determined.

In the analysis, the present worths of transmission/treatment non-collection facilities were divided by the design year (2000) dwelling unit equivalents for each of six communities studied in the Seven Rural Lake EIS.* This present worth per dwelling unit is plotted in Figure IV-A-1 against number of dwelling units per mile of collector sewer (including non-interceptor force mains, pressure sewers and gravity sewers in presently developed segments).

While the treatment and transport elements of this non-collector present worth vary widely as reflected in the entries of Table IV-A-5, and do not correlate with either total community dwellings or with density (dwelling units per mile of sewer in developed areas), the total non-collection present worths do correlate with density as shown in Figure IV-A-1. Because of economies of scale, the curved line passing through these points probably best reflects the actual relationship between housing density and cost per house. The linear regression drawn through the points, however, provides a reasonable approximation of the relationship within the range of densities involved, 30 to 130 units per mile.

Based on this linear regression, the following present worth values were used in the cost variability study for treatment and transport elements in centralized collection/transmission/treatment alternatives:

Density	Cost
Dwelling Units/Mile of Collector Sewer	July 1980 Present Worth Per Design Year Dwelling Unit
25	3,712
38	3,392
50	3,097
75	2,482
100	1,866
113	1,546
150	636

h. Step 8: Tally Wastewater Collection, Transmission and Treatment Units Necessary for Technologies Under Variable Growth Rates, Densities and Scenarios

Step 8 involved the assemblage of discrete collection, transmission and treatment units which comprise various wastewater technologies or systems suitable under different growth conditions, densities and environmental constraints. Technology units serving one-mile segments at densities of 25, 50, 75, and 100 (0% growth) and of 38, 75, 113, and 150 houses/mile were tallied. The degree of complexity in technology was directly proportional to the degree of environmental constraint under evaluation. An example of this relationship between environmental limitations and technological responses for on-site systems is shown in Table IV-A-4.

* Six communities include: Crooked/Pickerel Lakes, MI; Salem Utility District No. 2, WI; Steuben Lakes, IN; Green Lake, MN; Nettle Lake, OH; and Otter Tail Lake, MN.

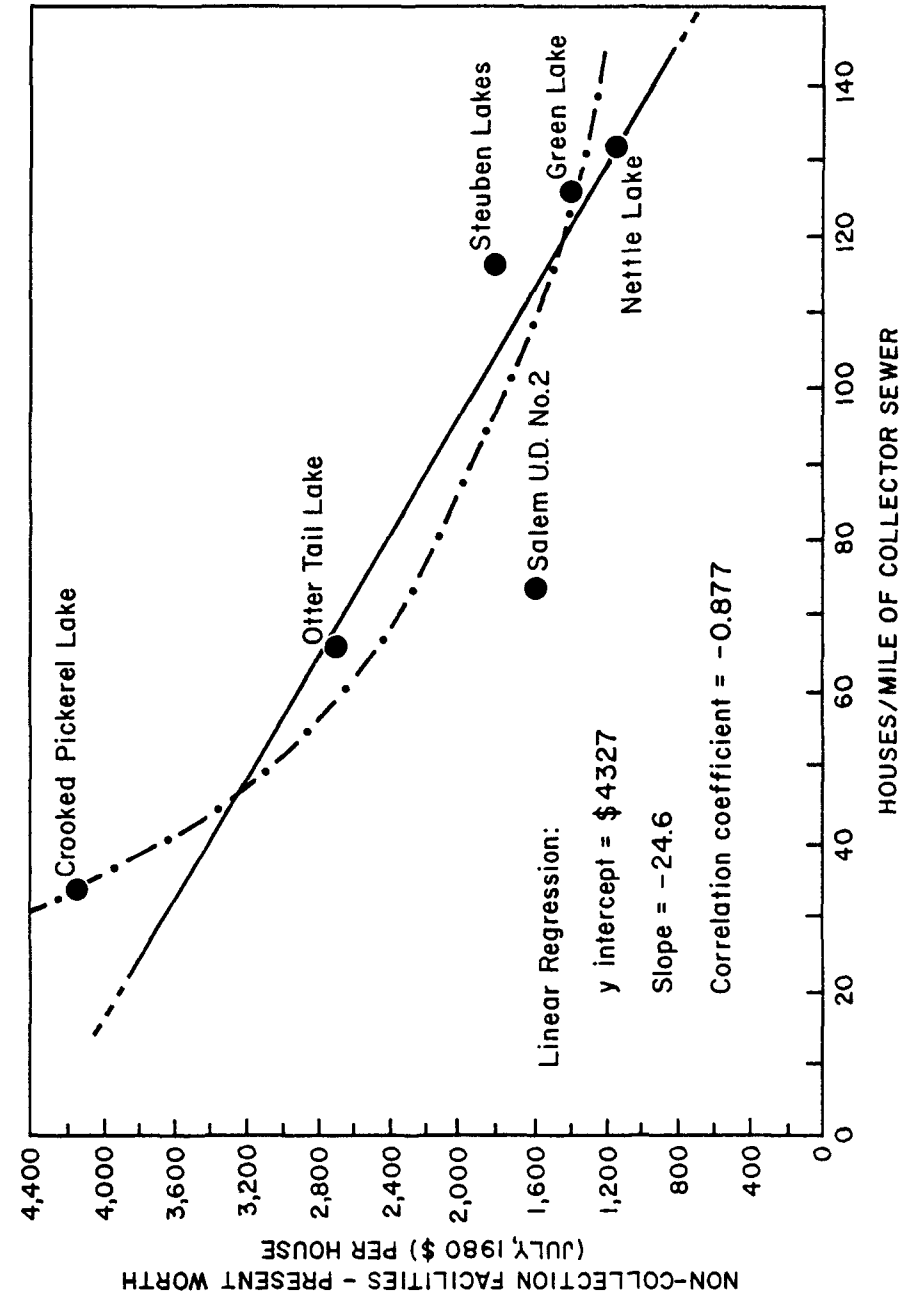

Figure IV-A-1. Present worth dollars per dwelling unit of non-collection facilities. (Source: Seven rural lakes EIS's, USEPA, 1979-1980.)

TABLE IV-A-5. PRESENT WORTH AND DWELLING UNIT STATISTICS FOR FACILITIES

Community	Design year dwelling units	Present Worth per Dwelling Unit of: Treatment and transmission ($)	Present Worth per Dwelling Unit of: Treatment ($)	Present Worth per Dwelling Unit of: Transmission ($)	Dwelling Units per Mile of: All sewers	Dwelling Units per Mile of: Collector sewers	Dwelling Units per Mile of: Transmission sewers
Crooked/Pickerel Lakes, Michigan	366	4,048	273	3,775	20.7	34.5	51.9
Salem Utility District No. 2, Wisconsin	2,102	2,614	1,389	1,225	64.5	66.3	1,110.0
Steuben Lakes, Indiana	3,189	1,541	625	916	63.1	74.0	431.1
Green Lake, Minnesota	6,196	1,778	855	923	93.7	116.8	473.4
Nettle Lake, Ohio	2,109	1,369	967	402	104.0	125.7	605.2
Otter Tail, Minnesota	495	1,098	646	452	100.9	131.9	429.4

SOURCE: U.S. EPA, 1979-80. Seven Rural Lake EIS's.

i. Step 9: Add Present Worth Unit Costs

All per house present worth costs for wastewater collection, transmission and treatment units in large-scale, small-scale and on-site systems, assembled in Step 8, were added in this step. With development of per house present worth cost figures completed in Step 9, the first objective of the cost variability study is achieved. The first objective is to quantify the effects of environmental and developmental variables on the costs of selected rural wastewater facilities. Remaining efforts (Steps 10-12) involve presentation and analysis of cost variability data.

j. Step 10: Prepare Per House Present Worth Data in Tabular Form

Per house present worth dollars for various technologies and combinations thereof, by housing density, by growth rate, and by scenario (which were added in Step 9), were tabulated for presentation. Tabulated data were presented for these technologies:

- collection only,
- centralized collection, transmission and treatment,
- collection, transmission and land application,
- collection, transmission and cluster systems, and
- on-site technologies.

The format used to present this information is shown in Table IV-A-6.

k. Step 11: Prepare Per House Present Worth Data in Graphic Form

A major effort in the cost variability study involved the plotting of cost data that were calculated and tabulated through Step 10. In Step 11, 138 graphs were prepared illustrating the effects of environmental and developmental variables on the present worth costs of several rural wastewater management technologies. These graphs are classified and quantified in Table IV-A-7.

An extensive amount of graphic data has been prepared to:

1) Enhance interpretation of data developed during the cost variability study. Curves contained in the graphs concurrently depict cost relationships between technologies, growth rates, housing densities and scenarios.

2) Facilitate achievement of the second objective of the cost variability study (i.e., compare the ranges of facility costs in order to identify thresholds of cost preference for competing technologies as they are affected by environmental and developmental variables).

Table IV-A-6. COST VARIABILITY STUDY RESULTS

TECHNOLOGY:

TOTAL PRESENT WORTH DOLLARS PER HOUSE

	DENSITY (houses/mile)	SCENARIO 1	2	3	4	5	6	7	8
0% Growth	25								
	50								
	75								
	100								
50% Growth	38								
	75								
	113								
	150								

TABLE IV-A-7. GRAPHIC DATA PREPARED IN COST VARIABILITY STUDY

Type of graph	Number of Graphics @ 0% growth	@ 50% growth	Total
Technology graphs	18	18	36
Scenario graphs	40	40	80
Cost-effective option graphs	8	8	16
Trade-off analysis curves	N/A*	N/A	6
			138

* N/A = Not applicable.

3) Facilitate use of cost variability study data by wastewater management officials in pre-Step 1 or early Step 1 decision-making.

Graphic material prepared in Step 11 is briefly described below.

Technology graphs. "Technology graphs" illustrate the relationships between housing densities and per house present worth costs of selected technologies by scenario. Each technology graph depicts this relationship for one type of technology and eight scenarios. Each curve on the technology graph represents one scenario. Technology graphs would be most useful to wastewater management planners who are interested in determining the effects of environmental constraints (8) on one (1) type of technology under 0% or 50% (or both) growth conditions. The eight scenarios are listed in Table IV-A-3. A "technology" here includes a component or combination of components of a wastewater management system. A "system" is composed of a complete range of wastewater management facilities including wastewater collection, transmission, or transport and treatment. Cost curves were developed for these technologies:

- collection technologies (no transmission or treatment);
  - conventional gravity sewers,
  - small diameter gravity sewers/septic tanks,
  - pressure sewers w/septic tank effluent pumps,
  - pressure sewers w/grinder pumps;
- centralized collection, transmission, and treatment systems;
- collection, transmission and land application by rapid infiltration, slow rate or overland flow methods;
- collection, transmission and cluster systems; and
- on-site technologies for 10%, 20%, and 50% replacement levels.

By definition, "collection technologies" do not constitute a wastewater management system. However, since collection components may comprise as much as 80% of total capital costs for centralized wastewater management systems, conventional and alternative collection technologies are included in the cost

variability study. The collection technology curves can also be used where transmission and treatment facilities are already constructed so that centralized alternatives require only the construction of collector and house sewers. Examples of technology curves describing relationships between housing density (houses/mile of collector sewer) and per house present worth costs under 0% and 50% growth conditions are shown in Figures IV-A-2 and -3, respectively.

Scenario Graphs. Examples of "scenario graphs" prepared during the cost variability study are illustrated in Figures IV-A-4 and -5. These curves present the same information as that contained in the technology curves except that they are organized by scenario rather than by technology (i.e., one scenario and several technologies). Scenario graphs may prove their usefulness in the wastewater management planning process by indicating cost advantages of one or more technologies over others under an assumed set of environmental constraints (i.e., a scenario). This information is presented for both 0% and 50% growth conditions.

Cost-Effective Options. Cost-effective option graphs represent a distribution of information presented in the scenario graphs. Each cost-effective option graph depicts the most cost-effective large-scale and small-scale off-site and on-site technologies for a given scenario. There are 16 graphs in all: 8 graphs for the 8 scenarios under 0% growth conditions, and 8 graphs for the 8 scenarios under 50% growth conditions.

1. Step 12: Analyze Data

The last step in the methodology is an analysis of all tabular and graphic data developed to achieve the third objective of the cost variability study (i.e., to identify those environmental and developmental variables that are most critical to the conclusions of cost-effectiveness comparisons between competing technologies).

3. ASSUMPTIONS

The development of cost ranges for large- and small-scale wastewater management facilities that are evaluated in the cost variability study has been based on an assortment of design and economic assumptions. Assumptions for each of the following technologies are discussed in this section.

- o On-site systems
- o Cluster systems
- o Land application systems
- o Centralized collection, transmission and treatment facilities

General assumptions universally supporting all cost-effectiveness analyses conducted in this study are cited initially.

a. General Assumptions

1. The cost variability study is based on the following economic parameters:

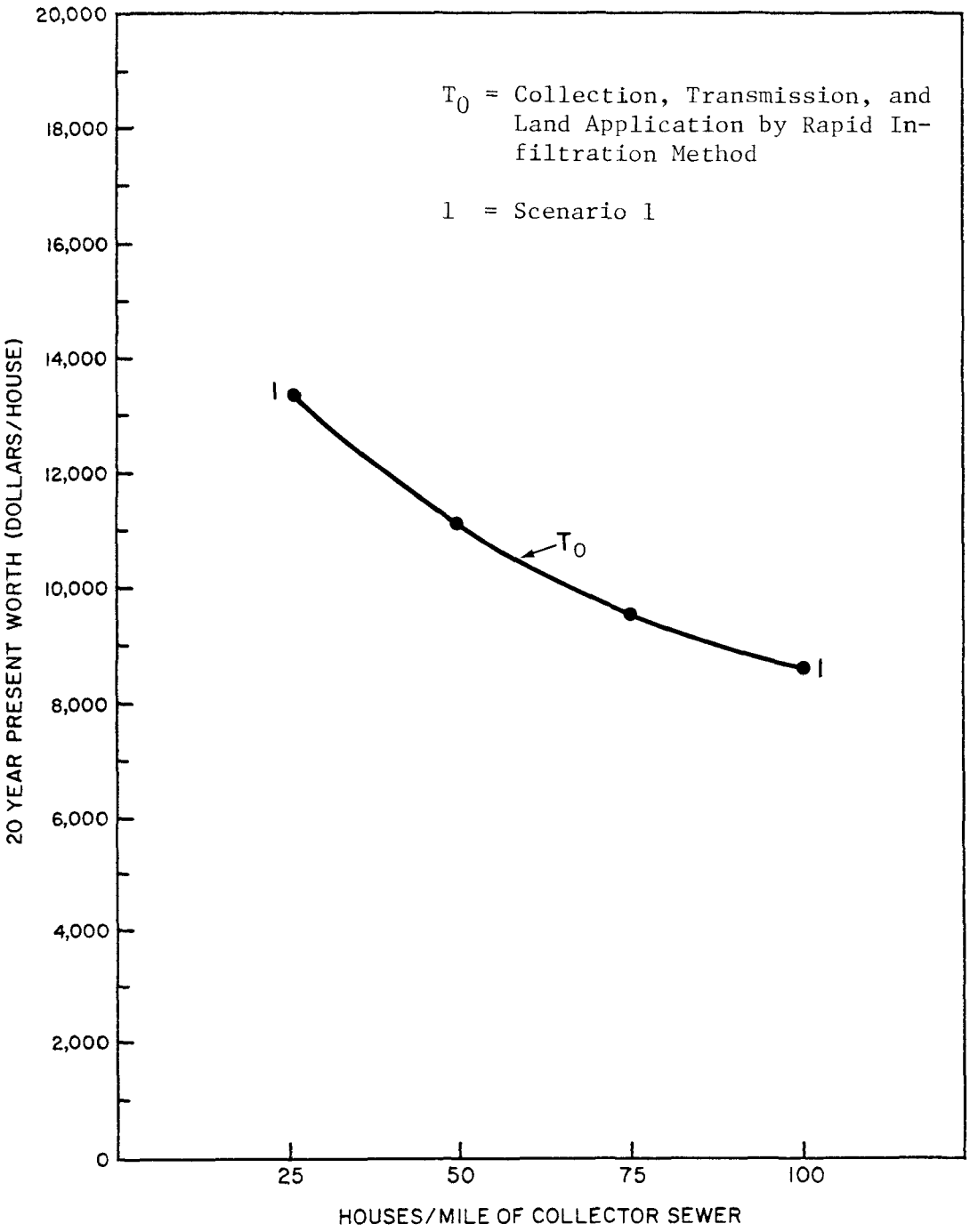

Figure IV-A-2. Graph used to illustrate 20 year present worth cost (per house) vs. housing density for selected technologies for 8 scenarios; growth rate over 20 year period = 0%.

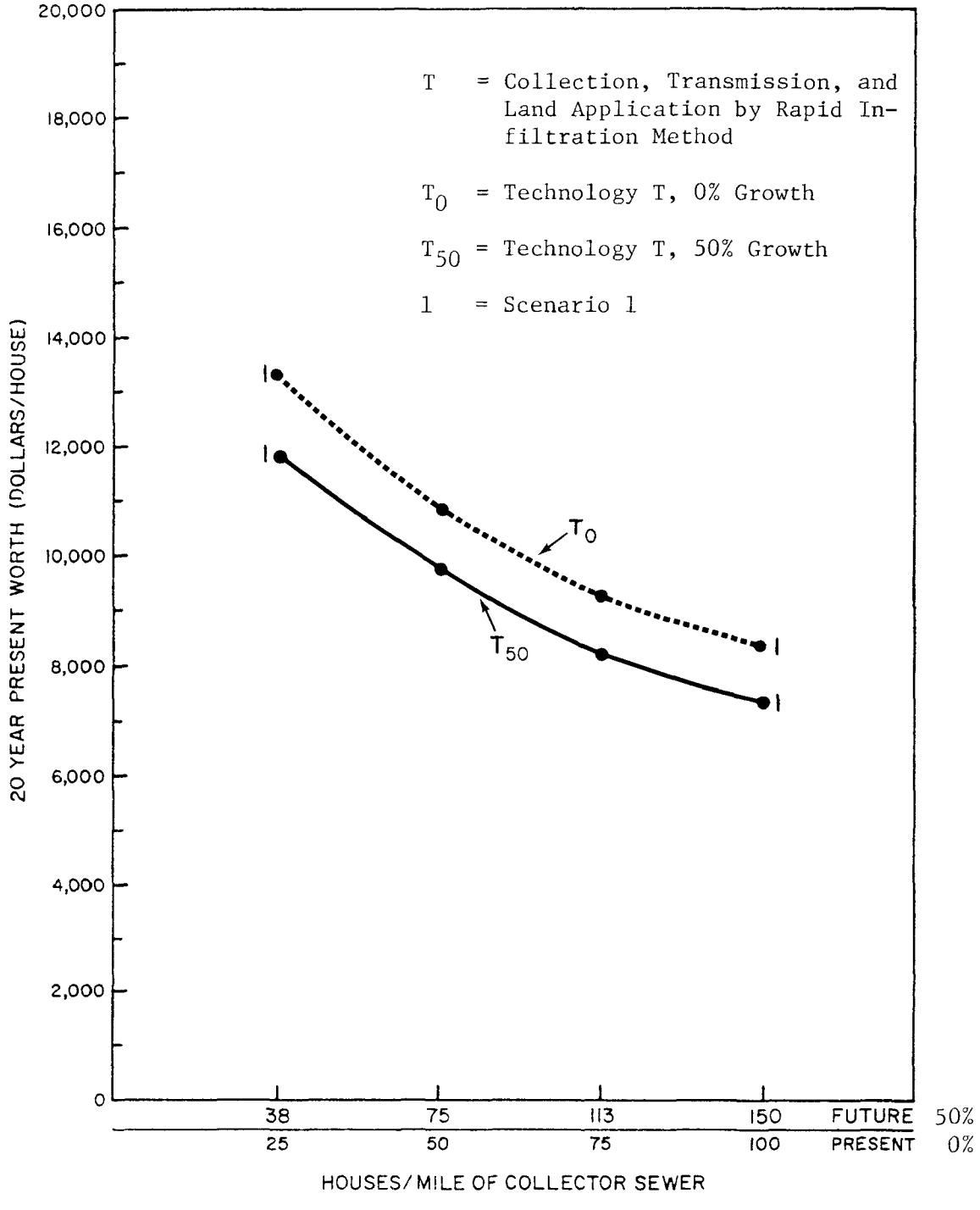

Figure IV-A-3. Graph used to illustrate 20 year present worth cost (per house) vs. housing density for selected technologies for 8 scenarios; growth rate over 20 year period = 50%.

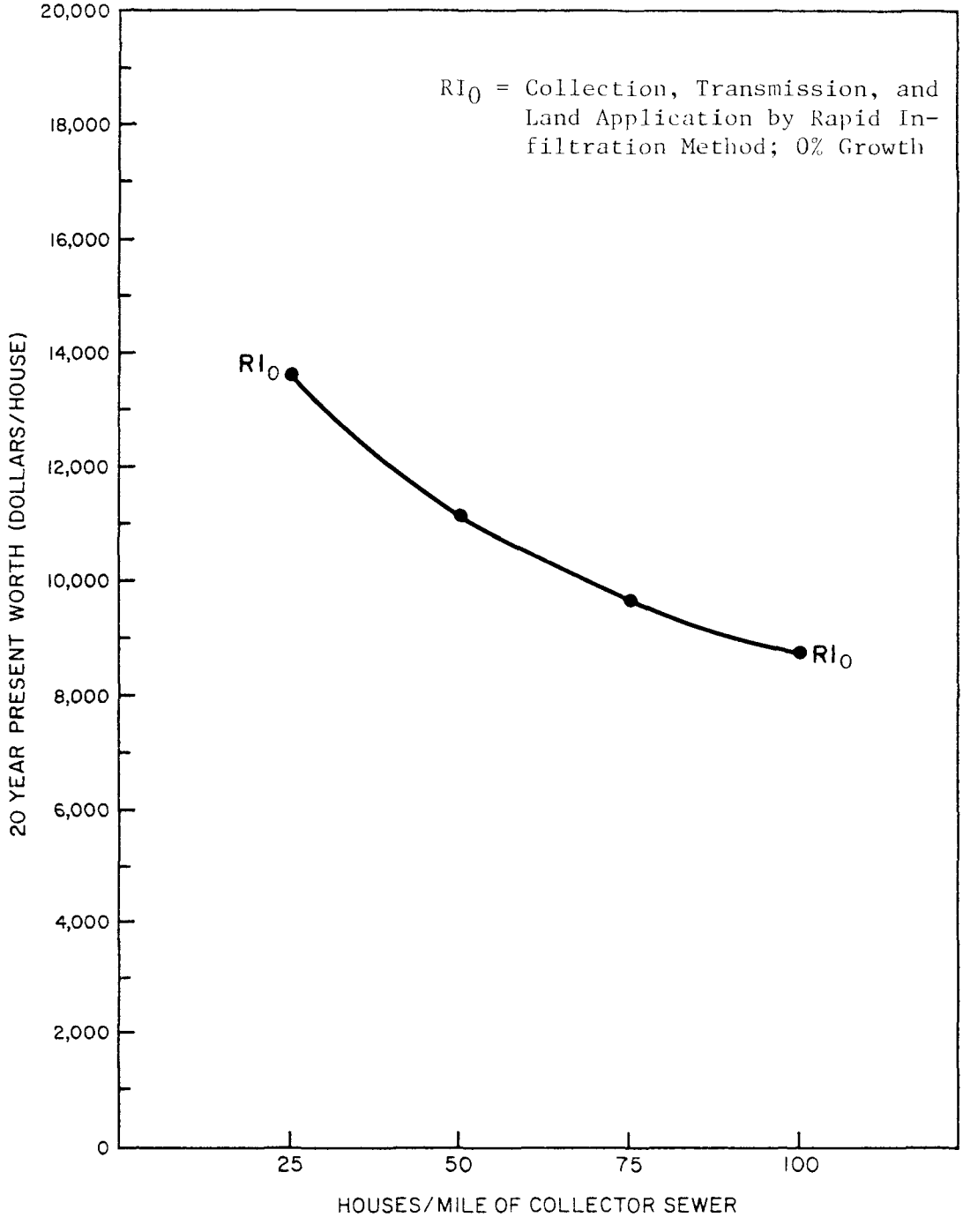

Figure IV-A-4. Graph used to illustrate 20 year present worth cost (per house) vs. housing density for selected technologies for 8 scenarios; growth rate over 20 year period = 0%.

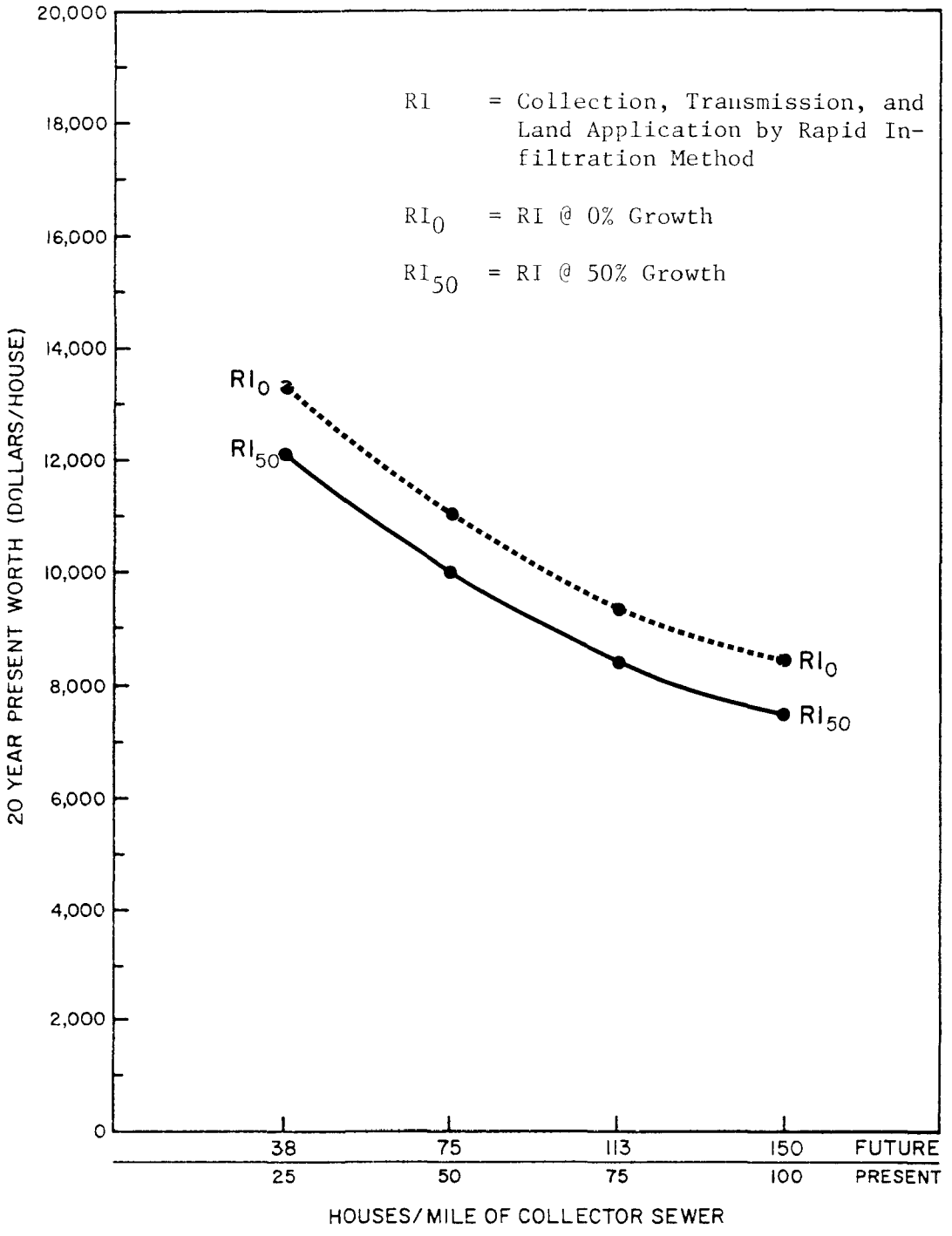

Figure IV-A-5. Graph used to illustrate 20 year present worth cost (per house) vs. housing density for selected technologies for 8 scenarios; growth rate over 20 year period = 50%.

- Planning period = 20 years
- Discount rate = 7.125%
- Service life - structures and sewers = 50 years - mechanical equipment = 20 years
- Construction cost index = 3260 ENR, 1 July 1980
- All final cost results expressed as July 1980 present worth
- Capital costs are derived by multiplying construction costs by 1.2. The additional 20% includes costs for administration of the project, legal fees, and project contingencies.

2. Segments are one mile in length. For centralized and small-scale, off-site alternatives, the point of flow concentration is the end of the segment.

3. Two rates of growth are assumed for all technologies in this study:

- No growth = 0%
- Future growth = 50%.

4. Housing density @ no growth =
   25, 50, 75, and 100 houses/mile.

   Housing density @ future growth =
   38, 75, 113, and 150 houses/mile.

5. Population, housing and flow assumptions for a range of housing densities are listed in Table IV-A-8. The length of house sewers assumed with the various densities is also listed.

b. On-Site Systems

In addition to the general assumptions described at the beginning of Section 2, the following parameters were used in the development of on-site system costs.

1. Unit costs for on-site systems are adopted from available literature and Region V contractors' estimates.

2. Unit present worth costs are calculated after updating unit operation and maintenance construction costs to July 1980 dollars.

3. Straight line depreciation is assumed for calculating salvage values.

4. Unit operation and maintenance costs include only those costs associated with the upkeep of the system. No costs for the small waste flows (SWF) district or monitoring are included.

TABLE IV-A-8. POPULATION, HOUSING AND FLOW ASSUMPTIONS BY HOUSING DENSITIES

1980 Houses/mile	10	25	50	75	100	150
1980 Population/mile[a]	35	88	175	263	350	525
1980 Base Flow[b]	2,100	5,250	10,500	15,750	21,000	31,500
I/I[L] 4"	800	800	9,800	800	800	800
8"	1,600	1,600	1,600	1,600	1,600	1,600
1980 Flow w/ I/I 4"	2,900	6,050	11,300	16,550	21,800	32,300
8"	3,700	6,850	12,100	18,150	22,600	33,100
2000 No Growth	10	25	50	75	100	150
2000 Houses/mile[d]	15	38	75	113	150	225
2000 Population/mile[d]	3,180	7,980	15,750	23,730	31,500	47,250
I/I[c]	Same as above					
2000 Flow w/ I/I 4"	3,980	8,790	16,550	24,530	32,300	48,050
8"	4,780	9,580	17,350	25,330	33,100	48,850
Average House Sewer Length	175'	175'	125'	75'	50'	35'

[a] 3.5 persons/house

[b] 60 gpcd

[c] 200 gpd gravity and small diameter only

[d] 50% increase over 1980

5. Administrative and monitoring costs associated with SWF management are excluded from the analysis altogether, as similar costs are not included for centralized alternatives.

6. Unit costs for flow reduction include the following: minimum flow reduction - toilet tank dams (2 sets), faucet aerators (3), low-flow shower heads (2); maximum flow reduction - air assisted toilet (1), air assisted shower (1), and faucet aerators (3).

7. The detailed site analysis costs were developed for three replacement levels: 10%, 20%, and 50%.

8. Detailed site analysis costs per house are based on a one-mile segment with 100 homes per mile.

9. When estimation of certain items in the detailed site analysis is not possible on a per mile basis, average estimated costs from the Seven Rural Lake EIS's are used. These costs are calculated on a per house basis, averaged, and applied to the 100 homes in the segments. Costs that were calculated in this manner include: fluorescence and groundwater meters, field sampling equipment, paper supplies, cameras and film, report production, communication, graphics, and report preparation.

10. Future growth is 50% (spread uniformly over the 20-year design period).

11. Future growth cost calculations assume that new systems will include septic tank/soil absorption systems (ST/SAS) (60 perc), and alternative systems. These assumptions allow two present worth costs to be used independently of one another.

12. Future growth costs are calculated by multiplying the present worth cost of constructing one system per year for 20 years by the number of systems to be replaced per year.

c. Cluster Systems

1. Cluster system unit costs are derived from the literature and contractors' bid sheets.

2. A cluster system includes drainfield, gravity siphons and dosing chamber, monitoring wells, and land.

3. Costs are developed for 50,000, 30,000, 20,000, 10,000, and 2,000 gpd.

4. Drainfield design is based on using a loam soil with a percolation rate of 45 MPI which yields an application rate of 0.45 GPD/SF.

5. The application area includes 50% more drainfield than required. This provides for the construction of 3 fields: 2 fields are in operation while a third is resting and acting as an emergency backup. Land area includes a 25 foot buffer around the drainfield.

6. Cost of land is assumed to be $500/acre to maintain comparability to other off-site technologies.

7. Drainfield trenches are assumed to be 3 feet wide with 7 feet between trench centers.

8. Monitoring wells are included in each estimate as follows:

Sewage flow (gpd)	Number of monitoring wells
50,000	3
30,000	3
20,000	2
10,000	2
2,000	1

9. Operation and maintenance costs are estimated at $100 per monitoring well per year. Other operation and maintenance costs, such as pumping individual septic tanks, are not included.

10. Hydrogeologic surveys are included in the estimated costs.

11. Land values are assumed to increase 3% per year in order to calculate salvage values in accordance with the cost-effectiveness guidelines (Appendix A 40 CFR 35).

d. Land Application Systems

The basic design assumptions and unit processes for land application systems* involving treatment by rapid infiltration, slow rate, and overland flow are described for each system as follows:

- *Rapid Infiltration*

Design application rate	10 in/week
Period of application	52 weeks
Storage	30 days

The rapid infiltration system includes a stabilization or facultative pond as preapplication treatment followed by storage for 30 days. The pretreated wastewater is pumped out of the pond and transmitted to the infiltration basins by 250 feet of force main. The treated wastewater is chlorinated prior to application on land. The present worth per dwelling unit for three different design flows is as follows:

Design Flow (GPD)	No. of Dwelling	PW/Dwelling ($)
200,000	114	2,340
300,000	150	2,120
500,000	225	1,700

* Land application systems in this study include wastewater collection, transmission and treatment components.

- Slow Rate with Solid Set Sprinklers

Design application rate	1 in/week
Period of application	26 weeks
Storage	180 days

The slow rate system includes a stabilization of faculative pond as preapplication treatment, followed by storage for 180 days. The pretreated wastewater is designed to be pumped out of the pond and transmitted to solid set sprinkers. The transmission is by 1,000 feet of force main. The treated wastewater is chlorinated prior to application on land. The present worth per dwelling unit for three different design flows is as follows:

Design Flow (GPD)	No. of Dwelling	PW/Dwelling ($)
200,000	113	4,190
300,000	150	3,850
500,000	225	3,330

- Overland Flow

Design application rate	4 in/week
Period of application	26 weeks
Storage	180 days

The overland flow system includes a stabilization or facultative pond as preapplication treatment followed by storage for 180 days. The pretreated wastewater is designed to be pumped out of the pond and transmitted to gated pipe for distribution on overland flow terrace. The transmission to the application site is by 1,000 feet of force main. The treated wastewater is chlorinated prior to application on land. The overland runoff is collected in ditches along the bottom of terrace. Cost of disposing of the collected runoff is not included in the estimate. The present worth per dwelling unit for three different design flows is as follows:

Design Flow (GPD)	No. of Dwelling	PW/Dwelling ($)
200,000	113	2,860
300,000	150	2,580
500,000	225	2,150

e. Centralized Collection, Transmission and Treatment Facilities

Assumptions used in the development of costs for centralized (large-scale) collection, transmission, and treatment facilities during this task are based upon general design assumptions relevant to those facilities that were formulated during U.S. EPA's preparation of the Seven Rural Lake EIS's. Key design assumptions, upon which costs for wastewater collection facilities examined in this study are based, are presented in Table IV-A-9. These facilities include:

- conventional gravity sewers,
- small diameter gravity sewers w/septic tanks,
- pressure sewers with septic tank effluent pumps (STEP), and
- pressure sewers with grinder pumps.

TABLE IV-A-9. DESIGN ASSUMPTIONS

	Peaking Factor	Minimum Velocity	Assumed Pipe Material	Friction Factor, C[b]
1. Conventional gravity sewers[c]	4	2 fps	Common	100
2. Small diameter gravity sewers	4	1 fps	PVC	150
3. Pressure sewers with STEP	a	1 fps	PVC SDR 26	130
4. Pressure sewers with grinder pumps	a	2 fps	PVC SDR 26	150

[a] Flows are based on probability of pumps operating simultaneously, as given in Flanigan, L.J. and C.A. Cadmik, 1979, PRESSURE SEWER SYSTEM DESIGN WATER AND SEWAGE WORKS, APRIL 1979, pp. 25, 26, 28-30, 32, 34, 87.

[b] The friction factor is for the Hazen-Williams formula.

[c] Force mains for the gravity sewer system are designed similar to the GP + PS system.

[d] Force mains for the septic tank effluent sewer system are designed similar to the STEP + PS systems.

1. The determination of the percent shoring for gravity collection lines is performed on a segment basis. Ten percent less shoring is required for force mains and low pressure sewers due to their shallower average depth.

2. All pressure sewer lines and force mains 8 inches or less in diameter will be PVC SDR26, with a pressure rating of 160 psi. Those force mains larger than 8 inches in diameter are constructed of ductile iron with mechanical joints.

3. Cleanouts in the pressure sewer system will be placed at the beginning of each line, and thereafter, one every 500 feet of pipe in line. Cleanout valve boxes will contain shut-off valves to provide for isolation of various sections of line for maintenance and/or repairs.

4. The pumping units investigated for the pressure sewer system utilized effluent and grinder pumps. Both units include a 2 by 10 foot basin with discharge at 8 feet, control panel, visual alarm, mercury float level controls, valves, rail system for removal of pump, antiflotation device, and the pump itself. The grinder pump is a 2 hp pump with a total dynamic head of 90 feet. The effluent pump is manufactured in a 1, 1½ or 2 hp pump.

4. RESULTS

Cost variability study results, developed according to the methodology described in Section 2 and based upon the assumptions identified in Section 3, are presented and discussed here. The results indicate, via cost tables and graphs, the effects of housing density, environmental constraints and growth rate on the per house total present worth costs of selected off-site and on-site and wastewater management technologies.

a. Technology Graphs

Each technology graph prepared during the cost variability study shows the total present worth cost per household of one rural wastewater management technology as affected by eight different environmental constraints, two growth rates and housing densities which range from 25 to 150 houses per mile of roadway or collector sewer. Each curve on a technology graph represents one scenario or set of limiting environmental conditions.

Large-scale off-site, small-scale off-site, and on-site technologies examined in the technology graphs were described in detail in Table IV-A-1. Collectively, general technology categories that have been assessed include:

- Collection only (no transmission or treatment)
- Collection, transmission, and centralized treatment
- Collection, transmission, and land application
- Collection, transmission, and cluster systems

- On-site technologies.

The effect of four (4) different collection-technologies on the costs of large-scale and small-scale off-site wastewater management systems will be examined carefully.

The eight scenarios addressed in the cost variability study and depicted on the technology graphs include the following environmental constraints (and responses to constraints) which are most determinative in affecting technology costs:

Scenario 1 - 8-foot average depth of cut

Scenario 2 - 16-foot average depth of cut

Scenario 3 - 8-foot average depth of cut (with one pump station and force main)

Scenario 4 - Flat topography, 6-foot average depth to groundwater (and imported fill needed to replace 1,000 of peat soil

Scenario 5 - Flat topography and 6-foot average depth to groundwater

Scenario 6 - Topography necessitates 1 pump station and force main; 6-foot average depth to bedrock

Scenario 7 - Flat topography

Scenario 8 - Topography necessitates/pump station and force main; 2-foot average depth to bedrock (50% of houses are low relative to sewer and require septic tank effluent pumps).

Two growth rates, 0% and 50%, are examined for their effect on per house present worth costs of wastewater technologies. These figures represent two hypothetical rates of growth for the 20-year planning period.

The 36 technology graphs (Figures IV-A-4-1 through IV-A-4-36) presented in this section are indexed in Table IV-A-4-1. Tabulated background data for each graph are also indicated in Table IV-A-4-1.

The following discussion of technology graph results is organized by major technology category.

b. Collection Technologies (No Transmission or Treatment)

It was stated earlier that the cost of a conventional collection system generally represents more than 80% of the total capital cost of wastewater management facilities in rural areas. Lower housing densities in rural areas, with attendant greater lengths of sewer per system user, and unfavorable soil and topographic conditions, can result in excessive costs per household for sewerage collection facilities. Alternative collection systems are less sensitive to some of the constraints that contribute to excessive cost of conventional sewers. For these reasons, U.S. EPA has undertaken an evaluation

of collection systems (with no sewage transmission and treatment) as part of its cost variability study. These systems include the following alternatives to conventional gravity sewers:

- small-diameter or 4-inch, gravity sewers
- pressure sewers with septic tank effluent pumps
- pressure sewers with grinder pumps.

The per house total present worth costs for conventional gravity sewers, small-diameter gravity sewers, pressure sewers with septic tank pumps, pressure sewers with grinder pumps, and under 0% and 50% growth rates for the 20-year planning period are shown in Figures IV-A-4-1 through 4-8. Backup data for these graphs are included in Table IV-A-4-2 through 4-5. The cost data in Figures IV-A-4-1 and 4-2 for conventional gravity collection sewers indicate that the total present worth costs per house are highest for Scenarios 2, 4, and 6. At lower housing densities (25 and 38 houses per mile), the costs of small-diameter gravity sewers (Figures IV-A-4-3 and 4-4) are slightly (i.e., 10%) less than those of conventional gravity sewers for all scenarios, and under both 0 and 50% growth conditions. At high densities (100 and 150 houses per mile), the reverse is true for all scenarios except 4, 5, and 7. In these three scenarios, representing flat topographical conditions, the costs of conventional gravity sewers are approximately 15% more than small-diameter gravity sewers. This cost difference is due to the different depths of cut required in flat topography for conventional and small-diameter gravity sewers. For conventional gravity sewers, the depth of cut would be between 8 and 24 feet, to provide adequate grade to maintain a minimum velocity of 2 fps. For a small-diameter sewer transporting effluent from septic tanks, the minimum velocity can be 1.5 fps which reduces the maximum depth of cut to 16 feet. The costs of pressure sewers with septic tank effluent pumps shown in Figures IV-A-4-5 and 4-6 are fairly close to each other for Scenarios 1, 2, 3, 4, 5, 6, and 7; only Scenario 8 is significantly higher. In comparison to conventional gravity and small-diameter gravity sewers, the pressure sewers with septic tank pumps are highly competitive under Scenarios 2, 4, and 5 for all housing densities and for both 0 and 50% rates of growth. By using septic tanks' pump pressure sewers, cost savings are in the range of $1,300 to $17,000 per house. The higher cost differences occur in the lower density range (i.e., 25 to 50 houses/mile), whereas lower cost differences occur in the higher density range (i.e., 75 to 100 houses/mile). However, pressure sewers with STEP pumps are expensive when compared to conventional gravity sewers for Scenarios 1 and 3. The use of conventional gravity sewers could result in cost savings in the range of $1,000 to $3,000 per house. The cost of pressure sewers with grinder pumps shown in Figures IV-A-4-7 and 4-8 are comparable to the cost of pressure sewers with septic tank pumps. The cost of Scenarios 1, 2, 3, 4, 5, 6, and 7 are fairly close to each other and only Scenario 8 is significantly higher. The costs of grinder pump pressure sewers are higher than septic tank effluent pump pressure sewers by $250 to $350 per house for scenarios without growth, (i.e., with 0% growth).

Per house present worth costs of coventional and alternative collection sewers decrease as growth rate changes from 0% to 50% for the 20-year planning period. For conventional and small-diameter gravity sewers, this reduction in

household costs is largest in Scenarios 2, 4, 5, and 7, with estimated decreases of 31-33% for conventional gravity sewers and 27-33% for small-diameter sewers. Per house present worth costs of septic tank pump and grinder pump pressure sewers are relatively less sensitive to a change in growth rate (0% to 50%), with household cost reductions of 20-28% and 10-23%, respectively. Futhermore, the reductions in per house present worth costs of pressure sewers are almost universally equal for each scenario. The one exception is Scenario 8, which registers a 20-40% greater reduction in household present worth costs than reductions associated with Scenarios 1 through 7.

The general conclusions of collection systems in this cost variability study are as follows:

- In areas where the average depth of cut for conventional gravity sewers is 8 feet or the depth of cut is 8 feet and one pump station is required, then conventional gravity sewers are less expensive than alternative collection systems.

- In areas where the average depth of cut of conventional gravity sewers is 16 feet, or where topography is flat, or where the groundwater table is at a depth of about 6 feet and dewatering would be required to install conventional gravity sewers, then pressure sewers with septic tank pumps or grinder pumps are less expensive than conventional gravity sewers.

- A change in the growth rate from 0 to 50% effectively reduces the estimated household present worth costs of collection systems by 10% to 33%. The low end of this range includes pressure sewers; the high end includes conventional and small-diameter gravity sewers.

c. Collection Transmission and Centralized Treatment

The household total present worth costs for centralized off-site wastewater management systems, under 0 and 50% growth rates for the 20-year planning period, are illustrated in Figures IV-A-4-9 through 4-16. These graphs depict the household costs of collection/transmission/centralized treatment systems as affected by environmental conditions, developmental conditions, and various modes of collection. Collection methods in this analysis are the same as those examined earlier: conventional gravity sewers, small-diameter gravity sewers, pressure sewers with septic tank effluent pumps, and pressure sewers with grinder pumps. Tabulated data supporting these technology graphs are presented in Tables IV-A-4-6 through 4-9.

The hierarchy of technology curves for collection/transmission/centralized treatment systems is identical to that for collection (only) systems under both 0 and 50% rates of growth.

For centralized systems using conventional gravity collection sewers, per house present worth costs are highest for Scenarios 2, 4, and 5 (see Figures IV-A-4-9 and 4-10). Moderately high costs are associated with Scenarios 7 and 8. Household present worth costs are lowest for Scenarios 1, 3, and 6. Under 50 percent growth conditions, costs for the eight scenarios become slightly more competitive yet remain in the same order of highest to lowest cost.

The substitution of small-diameter gravity sewers for conventional gravity sewers has no effect upon the overall ranking of collection/transmission/centralized treatment system costs by scenario under both 0% and 50% growth rates (see Figures IV-A-4-11 and 4-12). At low housing densities (25 and 30 houses per mile), household present worth costs of systems using small-diameter sewer collection components are slightly less than costs of systems with conventional gravity components for all scenarios and under 0 and 50% growth conditions. At high housing densities, conventional gravity system costs are slightly less than those of small-diameter gravity systems for all scenarios except 4, 5, and 7. As stated in the discussion of collection system results, these exceptions are attributed to different depths of cut required in flat topography by the two gravity systems.

The array of technology curves for centralized systems using pressure sewers with septic tank effluent pumps (Figures IV-A-4-15 and 4-16) is identical to the array of collection technology curves in Figures IV-A-4-5 and 4-6) and Figures IV-A-4-7 and 4-8. Centralized off-site systems utilizing pressure sewers with septic tank pumps are less expensive than systems utilizing conventional gravity and small-diameter sewers for Scenarios 2, 4, and 5 for all housing densities under both 0% and 50% growth rates. Costs of centralized systems utilizing pressure sewers with grinder pumps are competitive with systems using pressure sewers with septic tank pumps for all scenarios under both growth rates.

The decrease in household present worth costs of centralized wastewater collection/transmission/treatment systems, estimated to result from a change in the growth rate (from 0% to 50%), ranges from 12% (pressure sewer systems) to 44% (gravity sewer systems). Cost variability study conclusions for centralized collection/transmission/treatment systems are similar to those for collection (only) technologies discussed earlier. The analysis of centralized collection/transmission/treatment systems does conclude, however, that a change in the growth rate from 0 to 50% reduces the estimated household present worth costs by 12 to 44%. This is slightly larger than the decrease noted for collection (only) technologies in a similar assessment.

d. Collection, Transmission and Land Application

Technology curves that illustrate the household present worth costs of centralized off-site systems utilizing three different methods of land application, as affected by density, environmental conditions and growth rate are included in Figures IV-A-4-17 through 4-22. Backup cost data for these curves are listed in Tables IV-A-4-10 through 4-12, which identify the different collection technologies utilized by density in each centralized land application system. Collection methods selected, including conventional gravity sewers, small-diameter gravity sewers, septic tank pump pressure sewers, or grinder pump pressure sewers, were those found to be most cost-effective based upon consideration of environmental conditions, density and growth rate. The three methods of treatment by land application that were evaluated in this exercise include rapid infiltration, slow rate and overland flow.

The technology curves indicate that, under 0 and 50% growth rates, most per house present worth costs of all land application systems are highest for Scenario 8. In this scenario, bedrock depth is severely limiting with an average of two feet. Site preparation costs of wastewater management facili-

ties under these conditions are extremely high. Present worth costs of wastewater management systems involving land treatment under Scenario 8 become competitive with system costs under Scenarios 2, 4, and 5 at a density of approximately 100 houses per mile. Household present worth costs for all land application systems are lowest for Scenario 1 under both 0 and 50% rates of growth. Scenario 1 represents optimum sewer construction conditions with minimum depth of cut requirements and problems associated with groundwater, bedrock or soil stability. Examination of the land application technology curves for both growth rates also indicates that per house land application system costs under Scenario 1 are generally competitive with Scenario 3 at a density of approximately 75 houses per mile. A change in collection technologies from conventional gravity to small-diameter gravity sewers, at this density accounts for the trade-off. An exception is noted for wastewater management systems involving rapid infiltration, where costs under these scenarios are competitive at a density of 75 houses per mile. Costs of land application systems under Scenarios 2, 4, 5, and 7 are all quite competitive up to densities of approximately 50 houses per mile, where systems costs under Scenario 7 become significantly lower through the high ends of the density range. This reflects the fact that small-diameter gravity sewers (in the collection component of the system) under Scenario 7 are more cost-effective than pressure sewers with septic tank pumps, under Scenarios 2, 4, and 5 at densities of 75 houses per mile or greater. Per house savings in present worth costs, realized by use of small-diameter gravity sewers instead of septic tank effluent pump pressure sewers, range from $500 to $600 under a 0% growth rate, and $80 to $150 under a 50% growth rate.

As with other off-site wastewater management facilities, household present worth costs of land application systems decrease as the rate of residential growth changes from 0 to 50% for the 20-year planning period. The range of decrease in these costs is from 9 to 23%. Decreases in per house costs of systems using the slow rate method are slightly less than those of systems using either the rapid infiltration or overland flow treatment methods.

In conclusion,

- Household present worth costs for centralized off-site systems with land treatment are highest under environmental conditions which include an average depth to bedrock of 2 feet and which involve densities less than 100 houses per mile. These costs are lowest under conditions where optimum sewer construction conditions exist.

- In most cases, conventional gravity collection technologies are cost-effective components of land application systems only in segments with moderate to high housing densities (75 to 150 houses per mile). Small-diameter gravity sewers are usually competitive with septic tank effluent pump pressure sewers at low to moderate densities (25 to 75 houses per mile) for all land application systems under both rates of growth. In no instance were grinder pump pressure sewers cost-competitive collection components.

e. Collection, Transmission and Cluster Systems

Per house present worth costs of off-site facilities utilizing cluster systems as the method of treatment are illustrated in Figures IV-A-4-23 through 4-30. Cost data prepared in support of these technology curves are presented in Tables IV-A-4-13 through 4-16. As with previously assessed off-site systems, U.S. EPA has evaluated the effect of conventional and alternative collection technologies on the household present worth costs of these small-scale wastewater management systems. It was determined in the cost variability study that, under both 0 and 50% growth rates, small-scale systems utilizing small-diameter gravity sewers and septic tank pump pressure sewers were more cost-effective than systems using other modes of wastewater collection.

Present worth costs of cluster systems employing conventional and small-diameter gravity sewers were highest for Scenarios 2, 4, 5, and 8. These conditions involve excessive depths of sewer cut, dewatering along sewer trenches, and extensive rock excavation. Lowest costs for approaches utilizing gravity sewers were associated with Scenario 1. For cluster system schemes including septic tank pump and grinder pump pressure sewers, highest household present worth costs were associated with Scenario 8, in which depth to bedrock is severely limiting (2 feet). Per house present worth costs of cluster system approaches using septic tank effluent pump pressure sewers under the remaining seven scenarios are extremely competitive, being within $80-$300 of one another for all densities under 0% growth conditions and within $50-$200 for all densities under 50% growth conditions. Costs associated with grinder pump pressure sewers under the seven remaining scenarios (0% growth conditions) are competitive within each of two groups: Scenarios 4, 5, and 7 (flat topography) and Scenarios 1, 2, 3, and 6 (hilly topography). Under a 50% growth, the remaining seven scenarios costs for these pressure sewers become competitive within a range of $600.

Per house present worth costs for small-scale approaches involving the use of cluster systems can be expected to decrease by approximately 10 to 30% as the residential growth rate increases from 0 to 50%. Larger reductions in household costs are observed in cluster systems utilizing conventional and small-diameter gravity sewers under Scenarios 2, 4, and 5. Costs of systems using pressure sewer technologies are less sensitive to a change in the growth rate.

The cost variability study concludes that the hierarchy of costs of off-site approaches involving the use of cluster systems is very similar to the hierarchy of costs for collection technologies alone. Where shallow bedrock conditions exist, costs of systems involving pressure sewers are prohibitive under all housing densities. In flat topography, household present worth costs of systems using gravity sewers are prohibitive.

f. On-Site Technologies

On-site technology present worth costs per house are illustrated in Figures IV-A-4-31 through 4-36. Corresponding backup data are presented in Tables IV-A-4-17 through 4-19. The on-site technology curves are linear instead of curvilinear for all scenarios. This is attributed to the fact that economies of scale, which are realized for off-site facilities as density

increases, are not realized for on-site systems. On-site costs per segment are not density dependent. Per house present worth costs will increase, not decrease, as the rate of residential growth increases. Again no economies of scale are realized as segment densities increase over the 20-year planning period. These economies are apparent, indeed are quite significant, for most off-site technologies. However, from the technology graphs, on-site technologies involving 10%, 20%, and 50% replacement levels for existing on-lot systems are significantly more cost-effective than most of the off-site systems.

The graphs indicate that present worth costs per house increase as the rate of on-site system replacement increases. The on-site technology curves suggest that highest household costs are associated with Scenario 8. Again, shallow depth to bedrock is the factor imparting high costs to on-site system users. However, compared to off-site technologies considered in the same scenario (8), on-site technologies are most cost-effective except at high densities where replacement rates exceed 20%.

Lowest household present worth costs of on-site technologies are associated with scenarios involving flat topography and an average depth to groundwater of six feet (Scenarios 4 and 5). Efficient operation of on-site systems may work, independent of these environmental conditions.

g. Scenario Graphs

The 80 scenario graphs show the same information as the technology graphs. However, each scenario graph represents one set of conditions as listed in Table IV-A-3 (p. IV-A-10) and depicts the per household costs of various technologies as determined by the environmental condition(s). These depictions, Figures IV-A-4-37 through IV-A-4-52, are referred to as scenario curves. Table IV-A-4-1 (Index of Tech Scenario, etc.) lists the scenario graphs included in this study and identifies cost tables supporting the scenario graphs.

The summary objective of preparing scenario graphs in addition to technology graphs was to enhance the reader's ability to identify trade-offs between technologies for any given environmental condition or set of conditions. This information may be used by local wastewater management planning officials in their early development of alternatives to be ultimately (late Step 1) evaluated in a detailed cost-effectiveness analysis. Technology graphs cannot serve this purpose adequately, as they only depict trade-offs between scenarios.

The following discussion of scenario graph results is again organized by wastewater management technology categories. Detailed review of the scenario graphs affords definition of subtle relationships between technology costs for the same scenario. The discussion identifies only the major trade-offs between competing technologies and points of interest.

h. Collection Technologies (No Transmission or Treatment)

An inspection of scenario graphs for these technologies indicates that optimum or near optimum sewer construction conditions (Scenarios 1 and 3) favor conventional gravity and small-diameter gravity sewers for all housing

densities. Gravity sewers that are cost-effective under these optimum conditions at 0% growth become even more cost-effective as the growth rate increases to 50% over the 20-year planning period. When local officials consider bedrock constraints (Scenarios 6 and 8), gravity sewers are cost-effective only at densities above 25 houses per mile. Under both growth rates, pressure sewers with septic tank effluent pumps and grinder pumps are significantly more cost-effective than gravity sewers in circumstances involving flat or rough (16 feet average depth of cut) topography.

i. Collection Transmission and Centralized Treatment

Scenario graphs for technologies in this category are presented in Figures IV-A-4-53 through 4-68. Observations made about scenario graphs for collection technologies described above hold true for collection/transmission/centralized treatment systems. The only difference between these two sets of graphs is the incremental cost for wastewater transport and treatment. As described in Section 2.g. of this study, these add-on costs range from $3,712 at the lowest end of the housing density range to $636 at the highest end of that range.

j. Collection, Transmission and Land Application Systems

The scenario graphs for land application systems (Figures IV-A-4-69 through 4-84) indicate that per house present worth costs for land application by all three methods (rapid infiltration, slow rate, and overland flow) are lowest under Scenario 1 and highest under Scenario 8. The difference between costs of systems constructed under the remaining scenarios is insignificant. The graphs also indicate that land application systems using treatment by rapid infiltration are more cost-effective than systems using either slow rate or overland flow treatment, all housing densities considered. Slow rate systems are the least cost-effective. At no housing density do three scenario curves intersect, suggesting that trade-offs between land application systems should not be expected. This assumes that users of these curves will make the same density-by-density determination of optimum (i.e., cost-effective) collection technologies as was made in this study (see Tables IV-A-4-10 through 4-12). If this determination is not made, there could be trade-offs.

k. Collection, Transmission and Cluster Systems

Observations based on scenario graphs for collection/transmission/cluster system treatment (Figures IV-A-4-85 through 4-100) are identical to those made of graphs for collection and collection/transmission/centralized treatment systems. In many scenarios, cost increments attributed to wastewater transport and treatment via cluster systems have neutralized or eliminated trade-offs between technologies that were observed in the scenario graphs for collection.

l. On-Site Systems

Scenario graphs for on-site technologies, which include 10%, 20%, and 50% replacement levels for existing on-site systems, are presented in Figures IV-A-4-101 through 4-116. With the exception of mixed on-site technologies based on a replacement rate of 50% under Scenario 8, these graphs indicate that topographic, groundwater and bedrock conditions have an insignificant

effect on the household present worth costs of on-site technologies (10%, 20%, and 50% replacement). Costs for on-site technologies with a replacement level of 10% range from $588 to $1,523 per house. These figures represent the lowest on-site technology costs considered in this study. This range increases from $1,205 to $2,328 per house under 20% growth conditions. Highest on-site technology costs are represented by approaches involving 50% replacement level. These costs range from $3,641 to $5,923 under no growth conditions, and from $4,546 to $7,154 under 50% growth conditions. The increase in per household costs between 0% and 50% growth is due to the use of the design year's number of dwellings for calculating per household costs. 100% of the cost of future on-site systems are included in the present worth estimate, whereas only 10% to 50% of existing systems need replacement.

m. Cost-Effective Options

This set of graphics synthesizes the most cost-effective technology options that were apparent during analysis of the scenario graphs. The most cost-effective technology category is highlighted as a "cost-effective option" by scenario. In all scenarios, on-site technologies at 10% and 20% replacement levels are the most cost-effective options available at densities ranging from 25 to 150 houses per mile. Occasionally off-site facilities are competitive with on-site technologies at 50% replacement levels. The following brief discussion will identify those technologies that are competitive with on-site systems.

Scenario 1. Communities with sufficient wastewater transport and treatment capacity and who need only to construct new conventional or small-diameter gravity collection lines may find their project to be competitive with on-site systems @ 50% replacement at housing densities of approximately 50 homes per mile (under 0% growth condition). As the growth rate increases to 50%, off-site facilities become more competitive vis-a-vis on-site systems. Centralized collection/transmission/treatment systems are competitive with 50% replacement on-site technologies in the vicinity of 90 houses/mile. With growth conditions in effect, collection (only) technologies (gravity sewers) become competitive with 20% replacement on-site systems. At high housing densities, all centralized off-site facilities are competitive under Scenario 1 (see Figures IV-A-4-116 through 4-125).

Scenario 2. Under Scenario 2, average depth of cut requirements put off-site systems out of competitive range with on-site technologies (see Figures IV-A-4-117 and 4-126). Cost-effective collection modes include both septic tank pump and grinder pump pressure sewers. A collection (only) system, utilizing pressure sewers with septic tank pumps, is competitive with 50% replacement on-site technologies at approximately 110-130 houses/mile at 50% growth.

Scenario 3. Cost-effective options under this scenario (see Figures IV-A-4-118 and 4-127) feature collection by gravity sewers. As in the previous two scenarios, off-site systems utilizing treatment by rapid infiltration and by cluster systems are not competitive. Collection (only) technologies achieve competition with 50% replacement on-site systems at densities of approximately 70 houses/mile (no growth) and 50 house/mile (50% growth). Competition with 20% replacement on-site technologies is achieved by collection (only) in the vicinity of 100 houses/mile.

Scenario 4. On-site technologies (all replacement levels) are clearly the most cost-effective options in this scenario (see Figures IV-A-4-119 and 4-128). There are no developmental conditions under which off-site facilities are competitive. Environmental factors mitigating against cost competition here include flat topography and groundwater depths averaging six feet.

Scenario 5. Cost-effective options for Scenario 5 (see Figures IV-A-4-120 and 4-129) are similar to those for Scenario 4 due to similar physical conditions.

Scenario 6. Under no growth conditions, on-site technologies (all replacement levels) are the most cost-effective options. At 50% growth, centralized collection (gravity)/transmission/treatment is competitive with 50% replacement on-site technologies at around 115 houses per mile (see Figures IV-A-4-121 and 4-130).

Scenario 7. Cost-effective options in this scenario include on-site systems (at all densities) and collection only technologies in the vicinity of 80 houses/mile (no growth) and 70 houses/mile (with growth). See Figures IV-A-4-122 and 4-131.

Scenario 8. Off-site facilities costs are significantly higher than those for on-site facilities under no growth conditions. Gravity sewer collection (only) facilities become competitive with 50% replacement on-site technologies at approximately 80 houses/mile, when rock blasting is economical. Under conditions of 50% growth, off-site system competitiveness improves with competition achieved at densities of approximately 70-85 houses/mile for collectors only, 90-110 houses/mile for centralized treatment and 115-150 houses/mile for cluster systems.

TABLE IV-A-4-1. INDEX OF TECHNOLOGY, SCENARIO AND COST-EFFECTIVE OPTION GRAPHS PREPARED DURING COST VARIABILITY STUDY

Figure number and title		Page number	Table reference
Figure IV-A-4-1.	Cost of conventional gravity sewers (no transmission/treatment) for 8 scenarios; 0% growth	A-1	IV-A-4-2, page B-1
4-2.	Cost of conventional gravity sewers (no transmission/treatment) for 8 scenarios; 50% growth	A-2	IV-A-4-2, page B-1
4-3.	Costs of small diameter gravity sewers w/septic tanks (no transmission/ treatment) for 8 scenarios; 0% growth	A-3	IV-A-4-3, page B-2
4-4.	Costs of small diameter gravity sewers w/septic tanks (no transmission/ treatment) for 8 scenarios; 50% growth	A-4	IV-A-4-3, page B-2
4-5.	Costs of pressure sewers w/septic tank effluent pumps (no transmission/ treatment) for 8 scenarios; 0% growth	A-5	IV-A-4-4, page B-3
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APPENDIX A

TECHNOLOGY CURVES
Conventional Gravity Sewers
(No Transmission/Treatment)
0% Growth

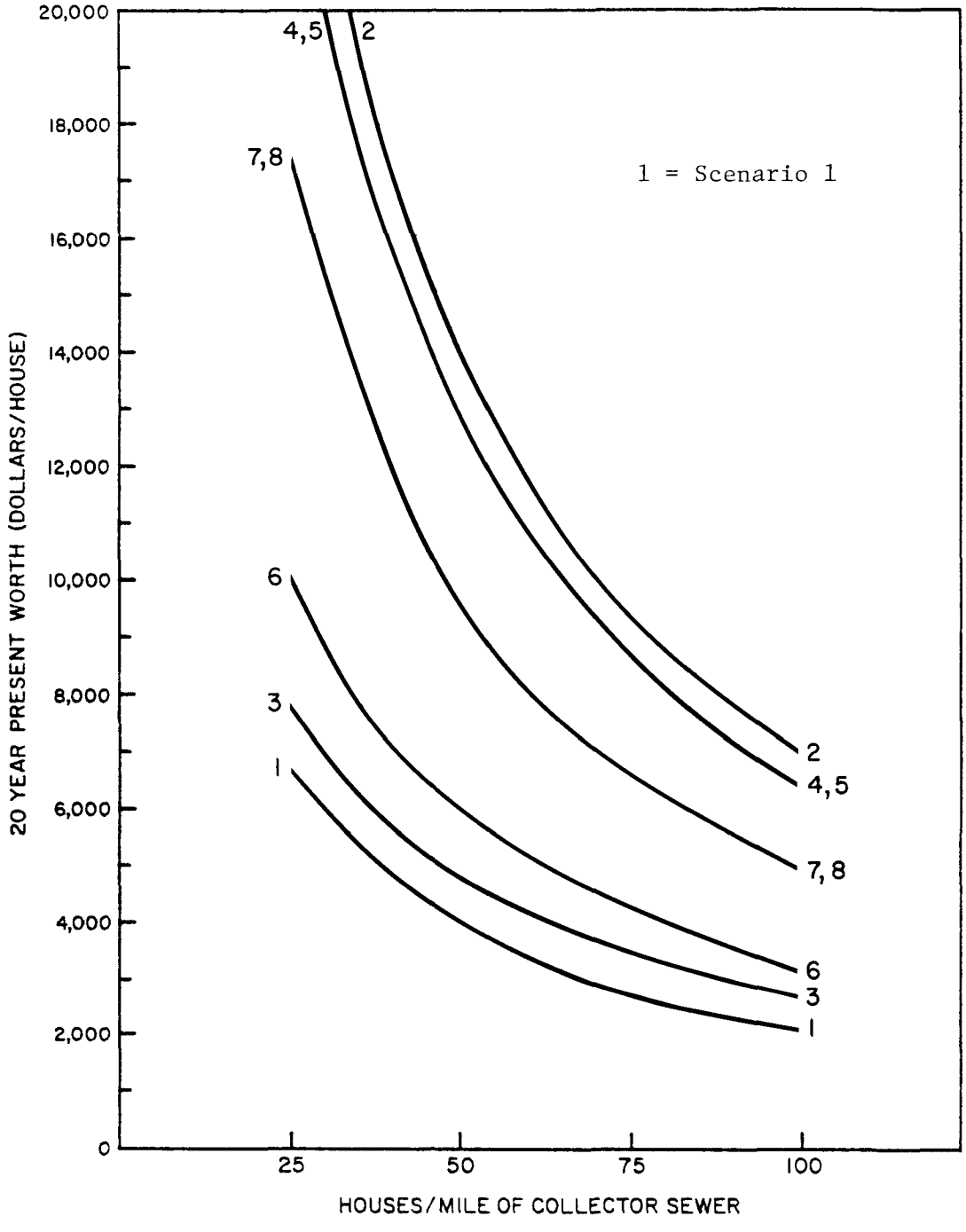

Figure IV-A-4-1. Costs of conventional gravity sewers (no transmission/treatment) for 8 scenarios; 0% growth.

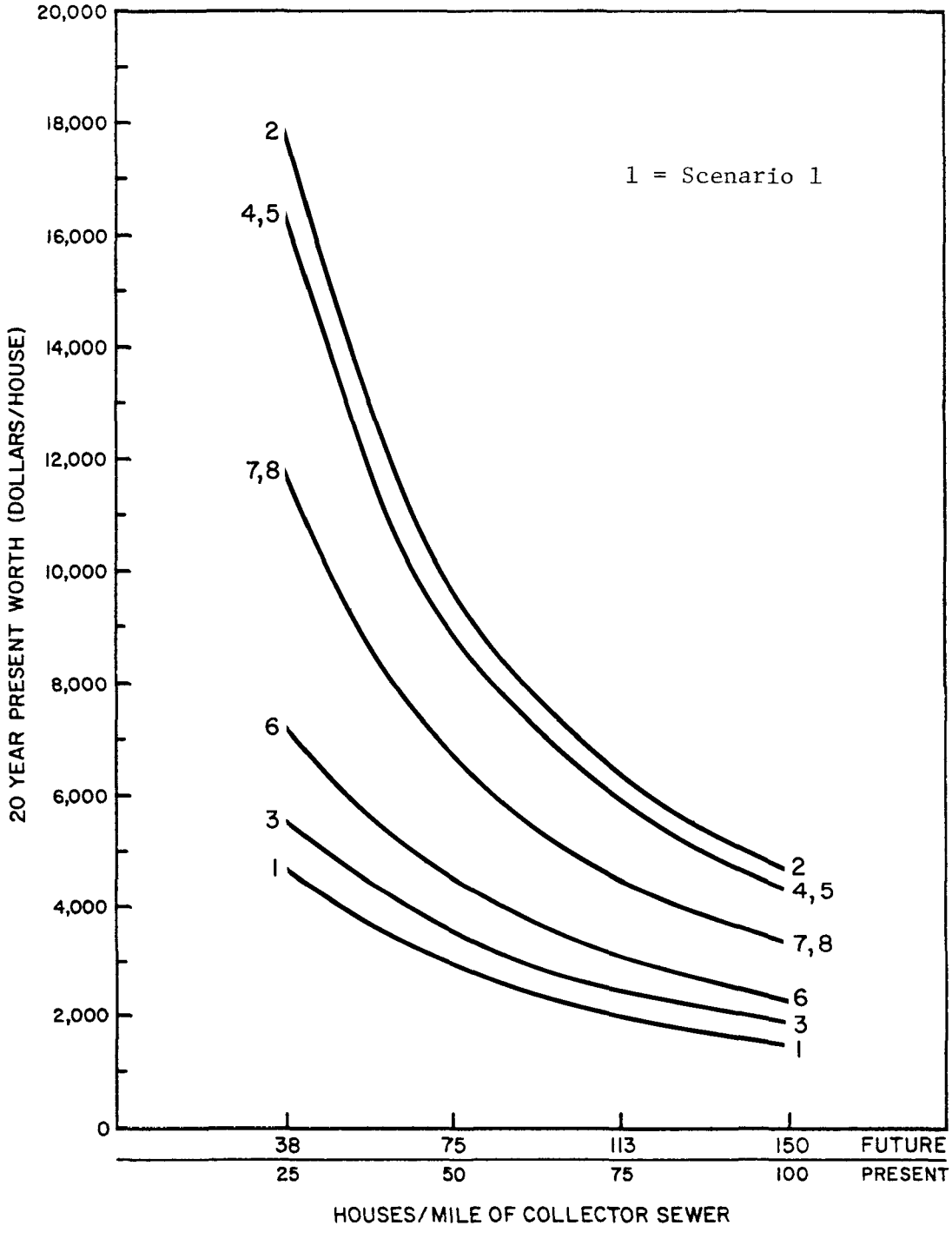

Figure IV-A-4-2. Costs of conventional gravity sewers (no transmission/treatment) for 8 scenarios; 50% growth.

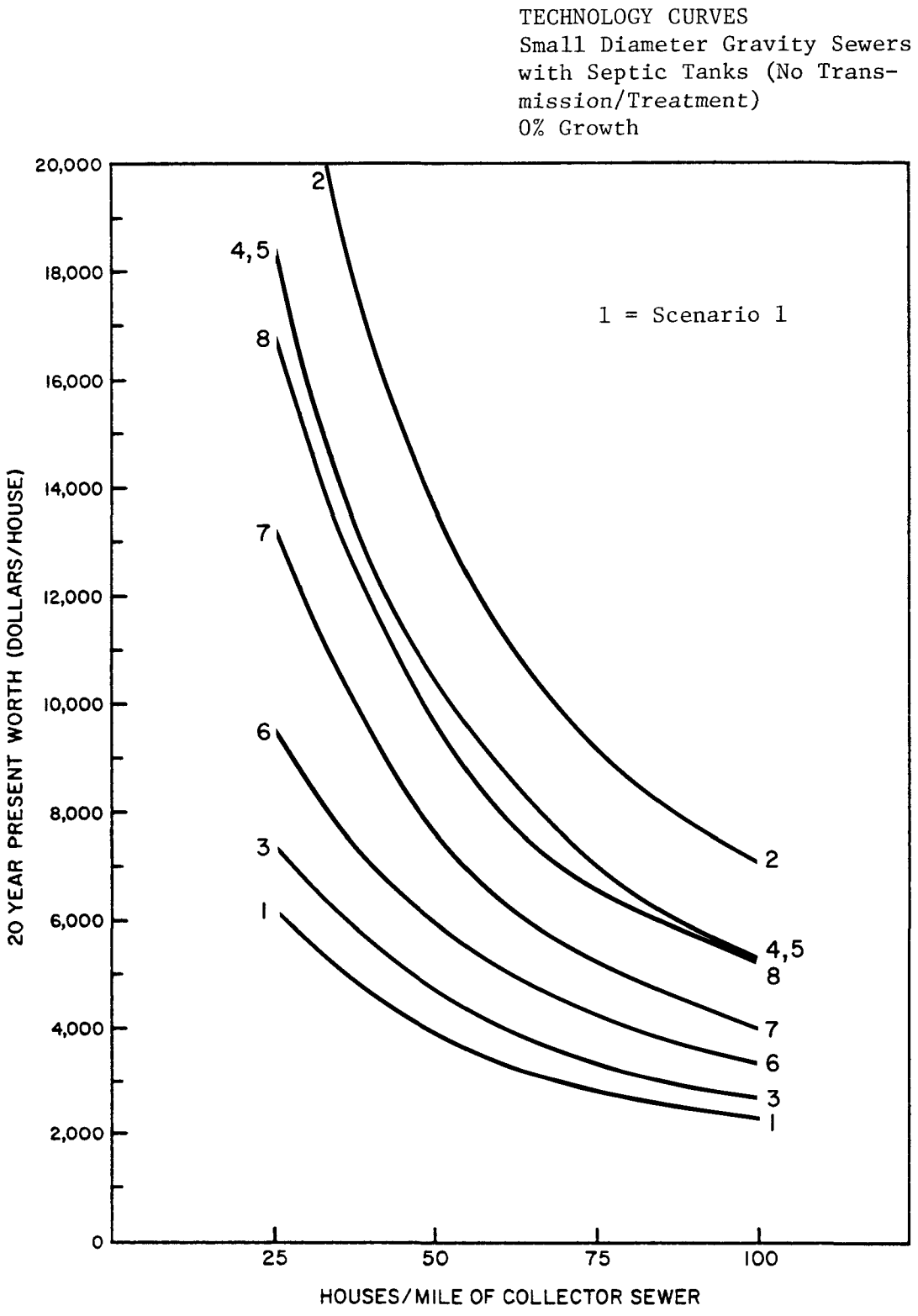

Figure IV-A-4-3. Costs of small diameter gravity sewers with septic tanks (no transmission/ treatment for 8 scenarios; 0% growth.

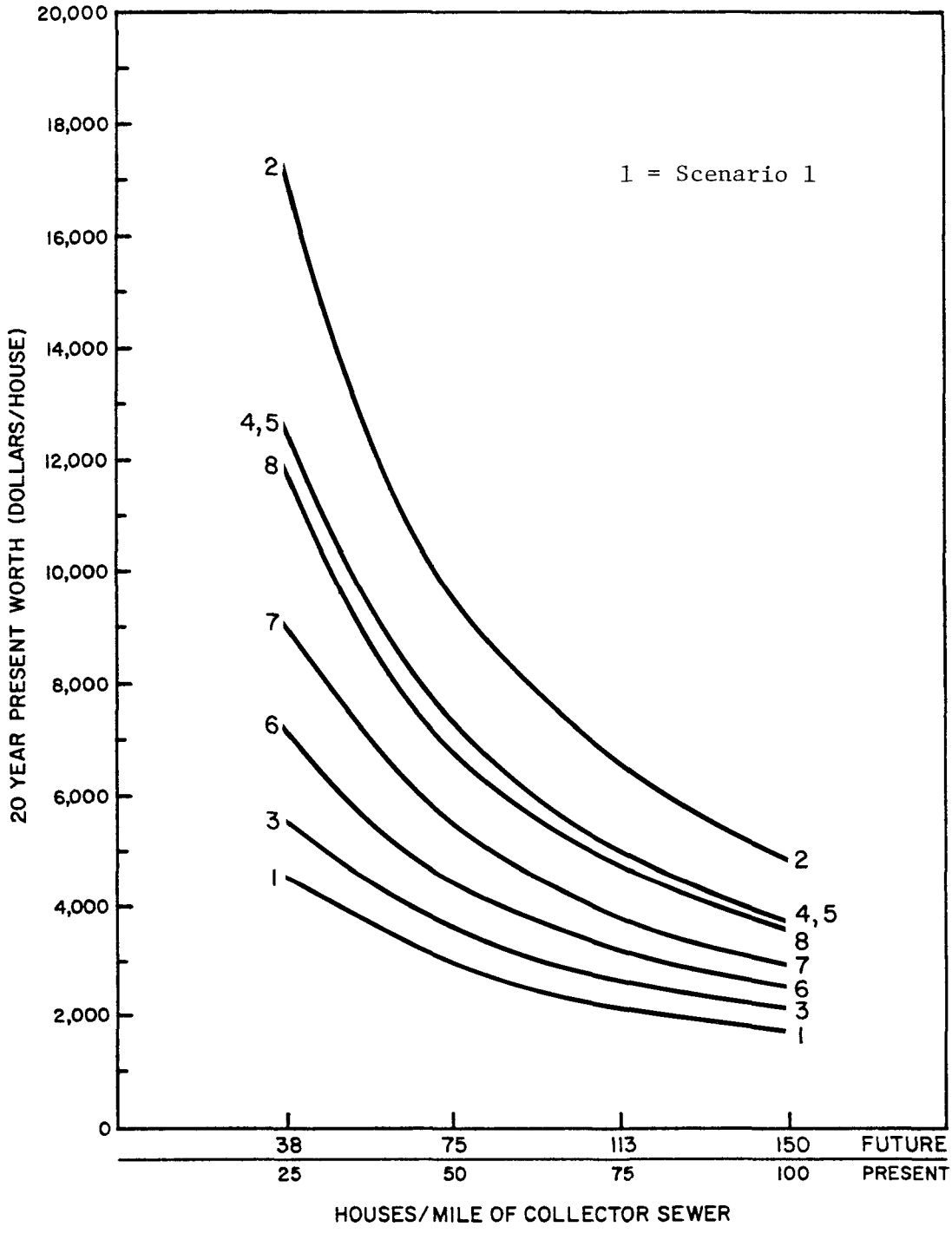

Figure IV-A-4-4. Costs of small diameter gravity sewers with septic tanks (no transmission/ treatment) for 8 scenarios; 50% growth.

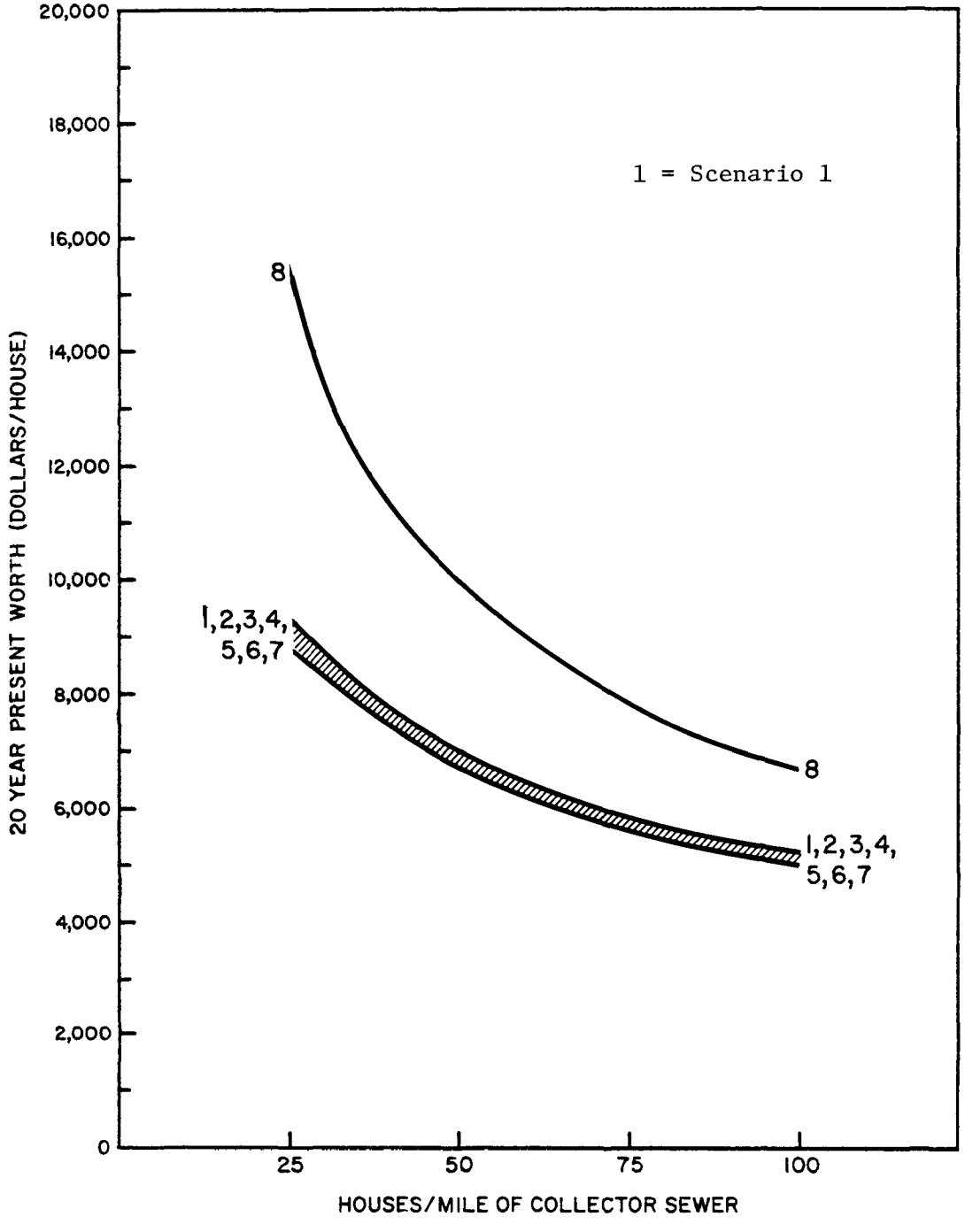

Figure IV-A-4-5. Costs of pressure sewers with septic tank effluent pumps (no transmission/treatment) for 8 scenarios; 0% growth.

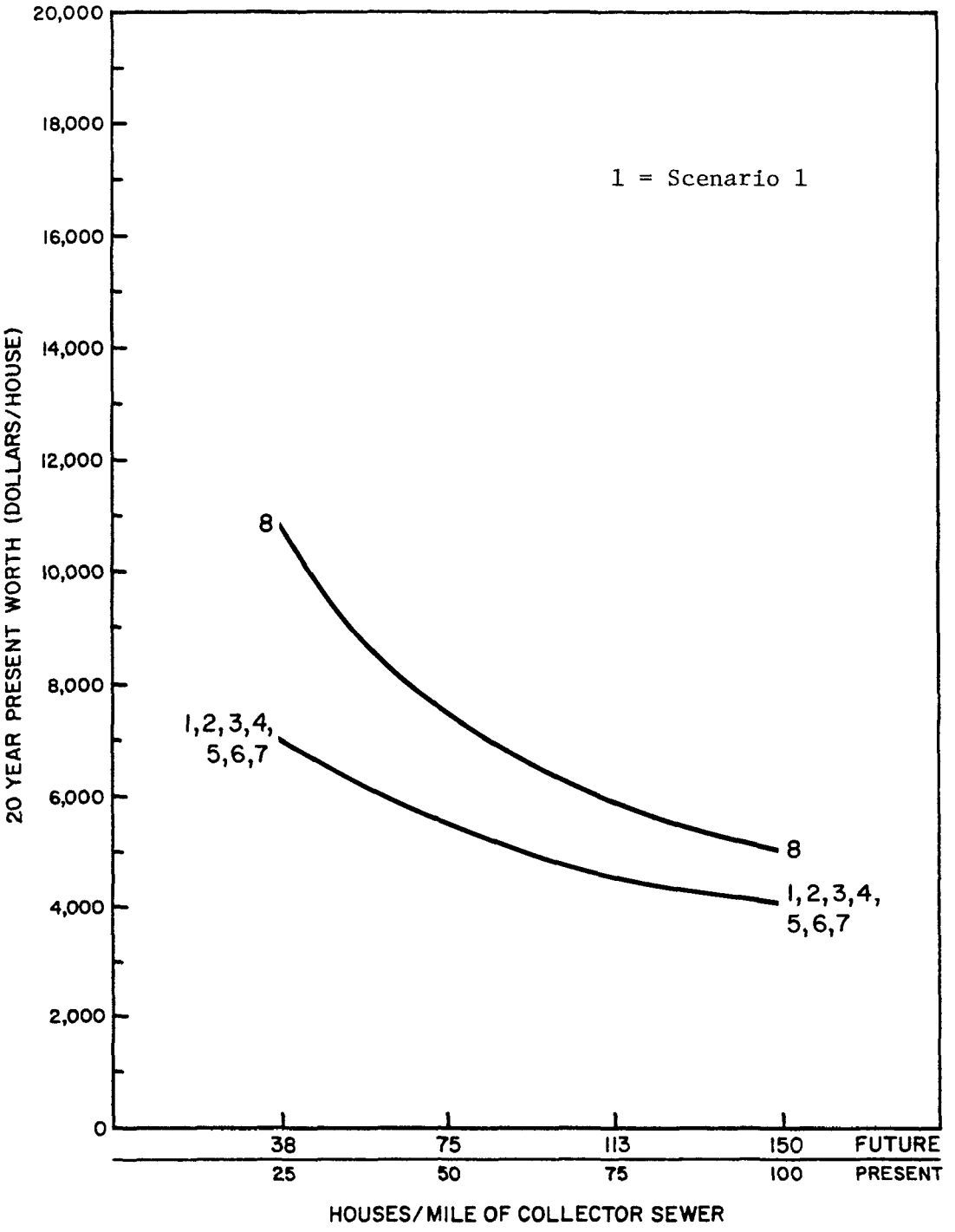

Figure IV-A-4-6. Costs of pressure sewers with septic tank effluent pumps (no transmission/treatment) for 8 scenarios; 50% growth.

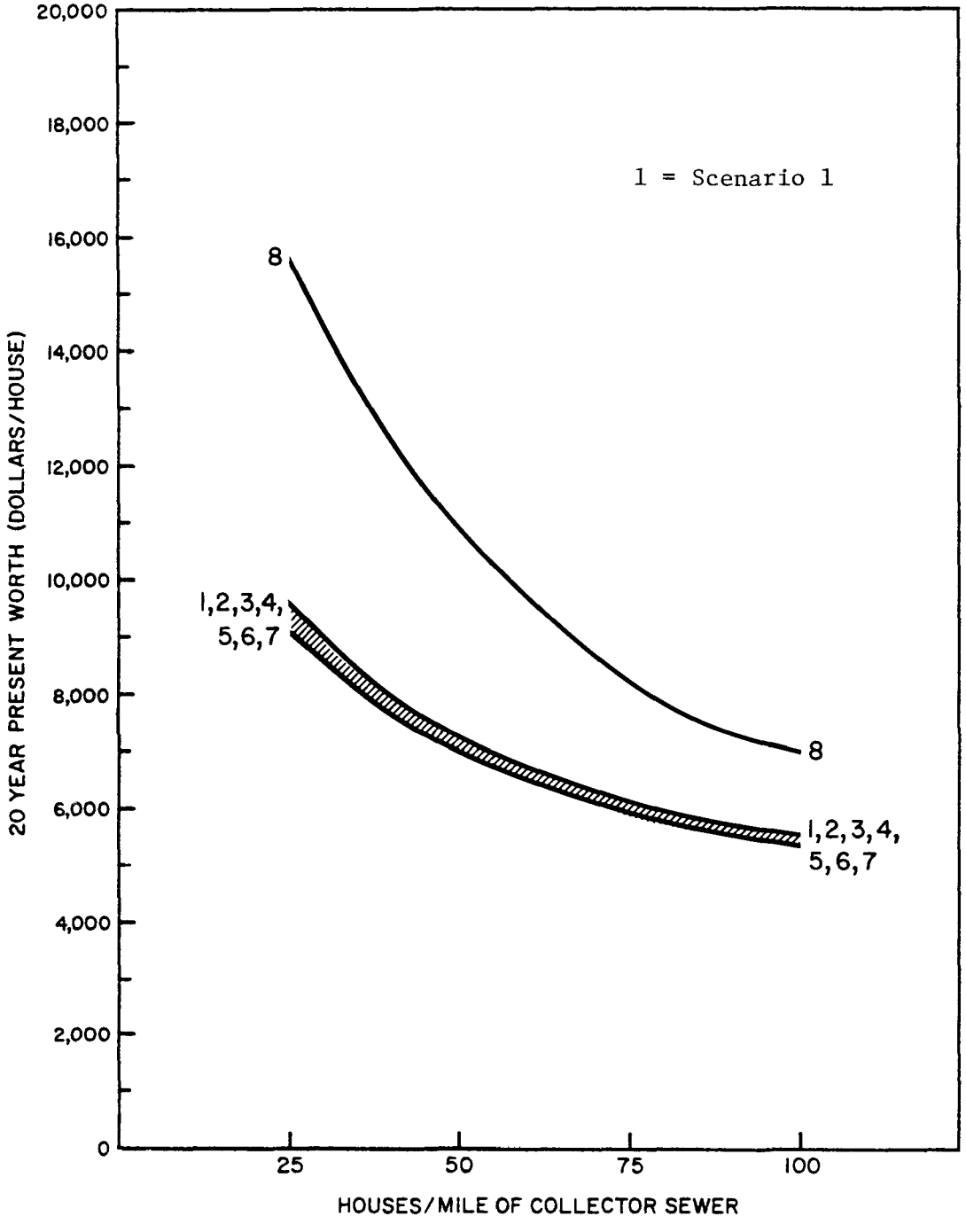

Figure IV-A-4-7. Costs of pressure sewers with grinder pumps (no transmission/treatment) for 8 scenarios; 0% growth.

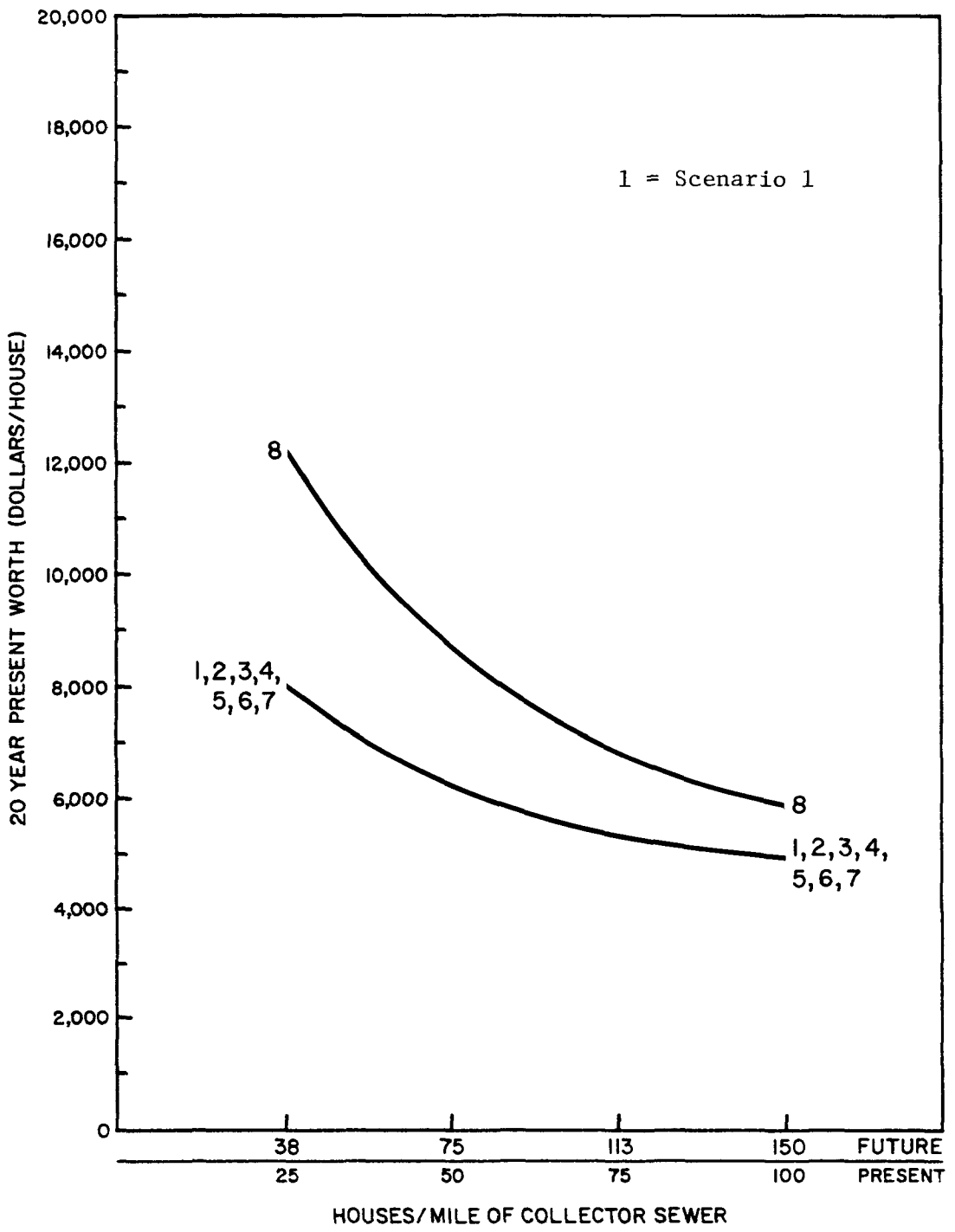

Figure IV-A-4-8. Costs of pressure sewers with grinder pumps (no transmission/treatment) for 8 scenarios; 50% growth.

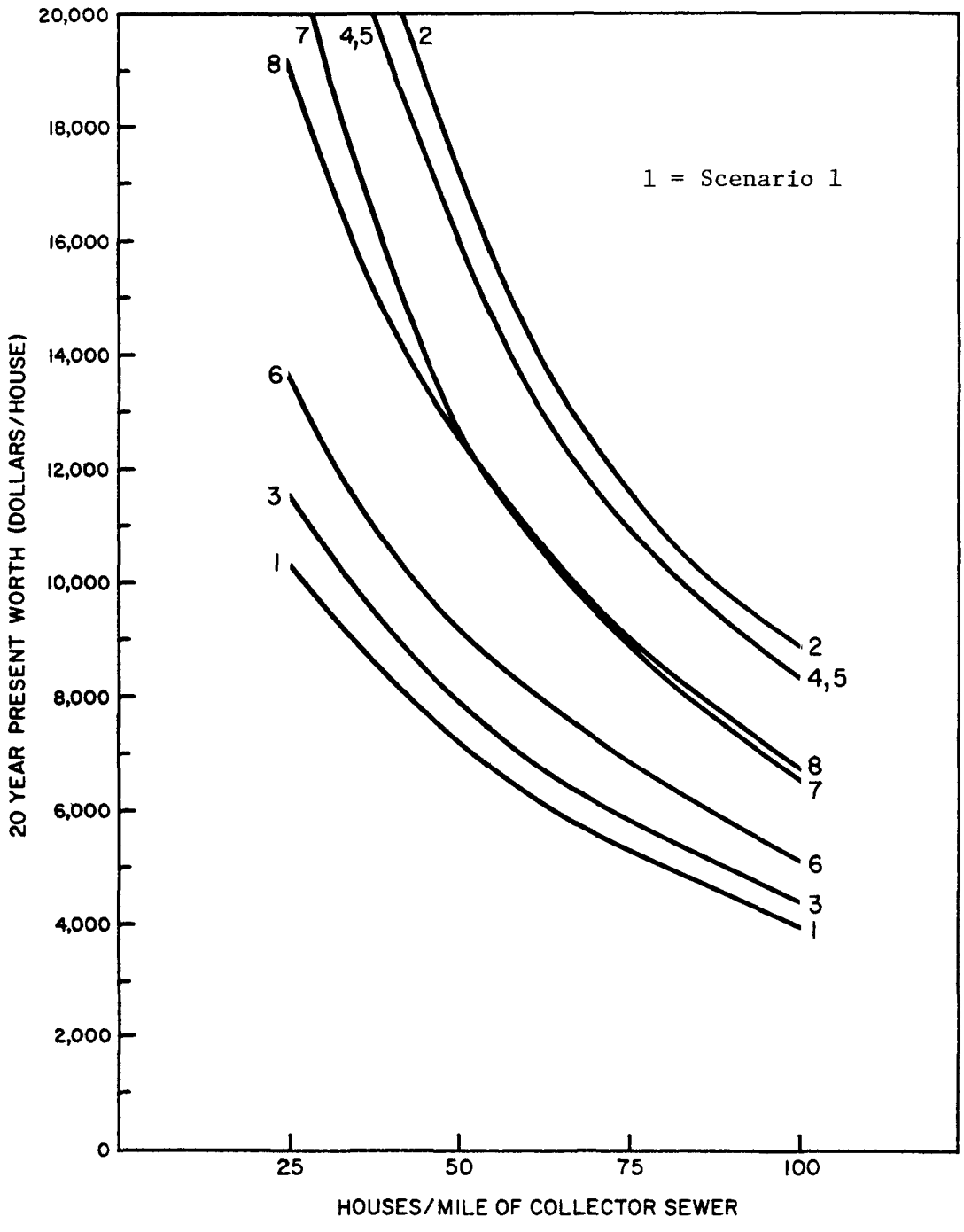

Figure IV-A-4-9. Costs of conventional gravity sewers, transmission, and treatment for 8 scenarios; 0% growth.

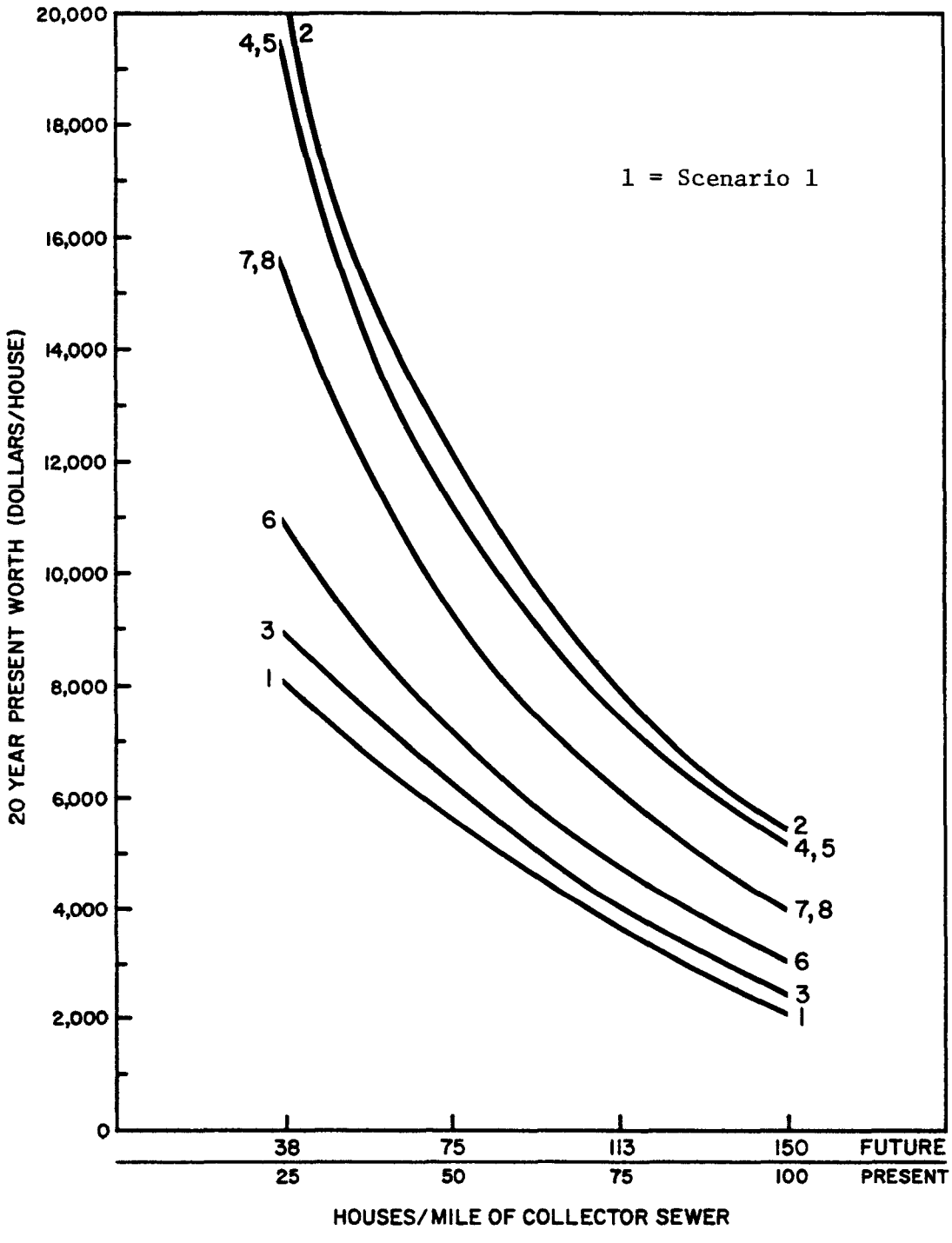

Figure IV-A-4-10. Costs of conventional gravity sewers, transmission, and treatment for 8 scenarios; 50% growth.

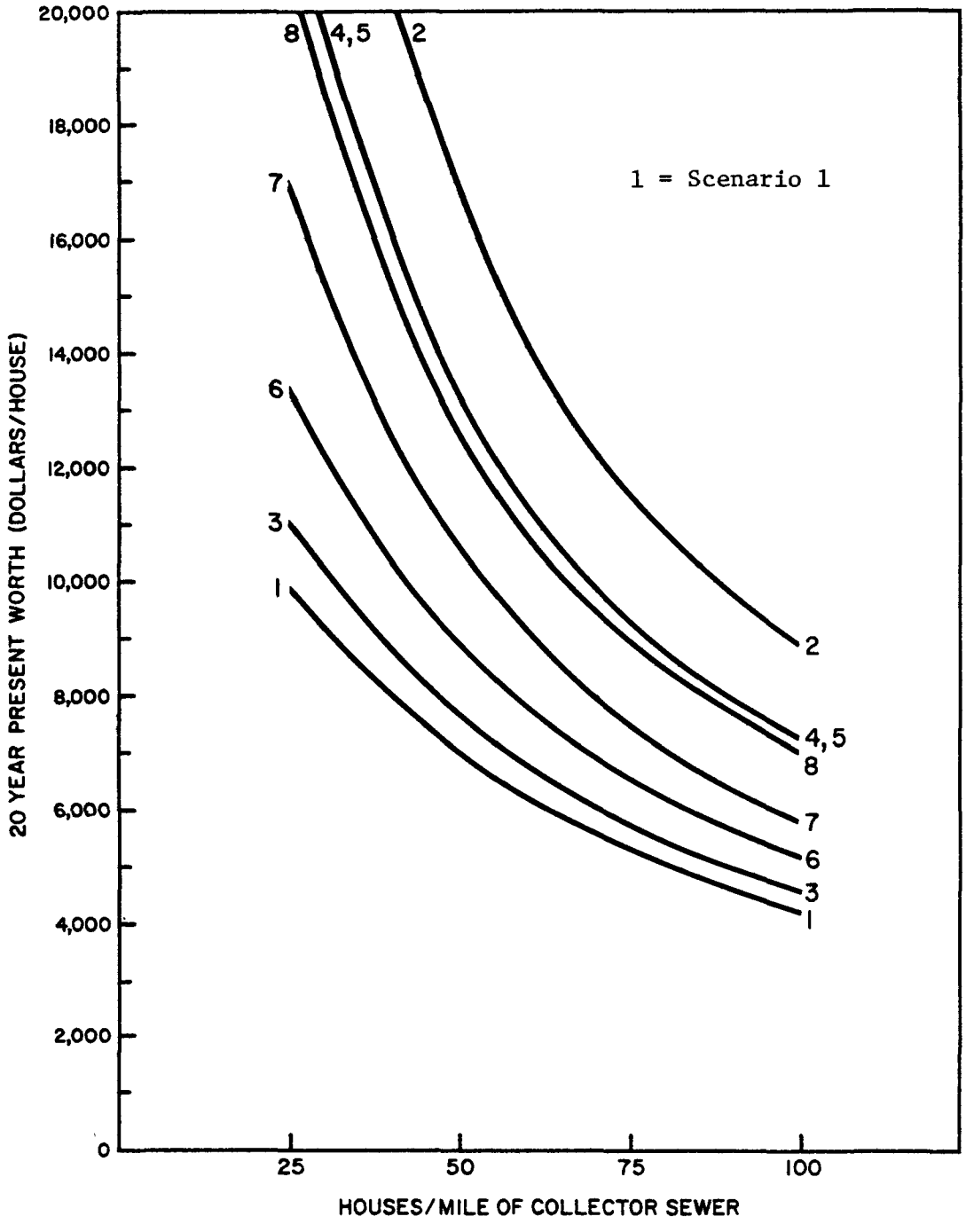

Figure IV-A-4-11. Costs of small diameter gravity sewers with septic tanks, transmission, and treatment for 8 scenarios; 0% growth.

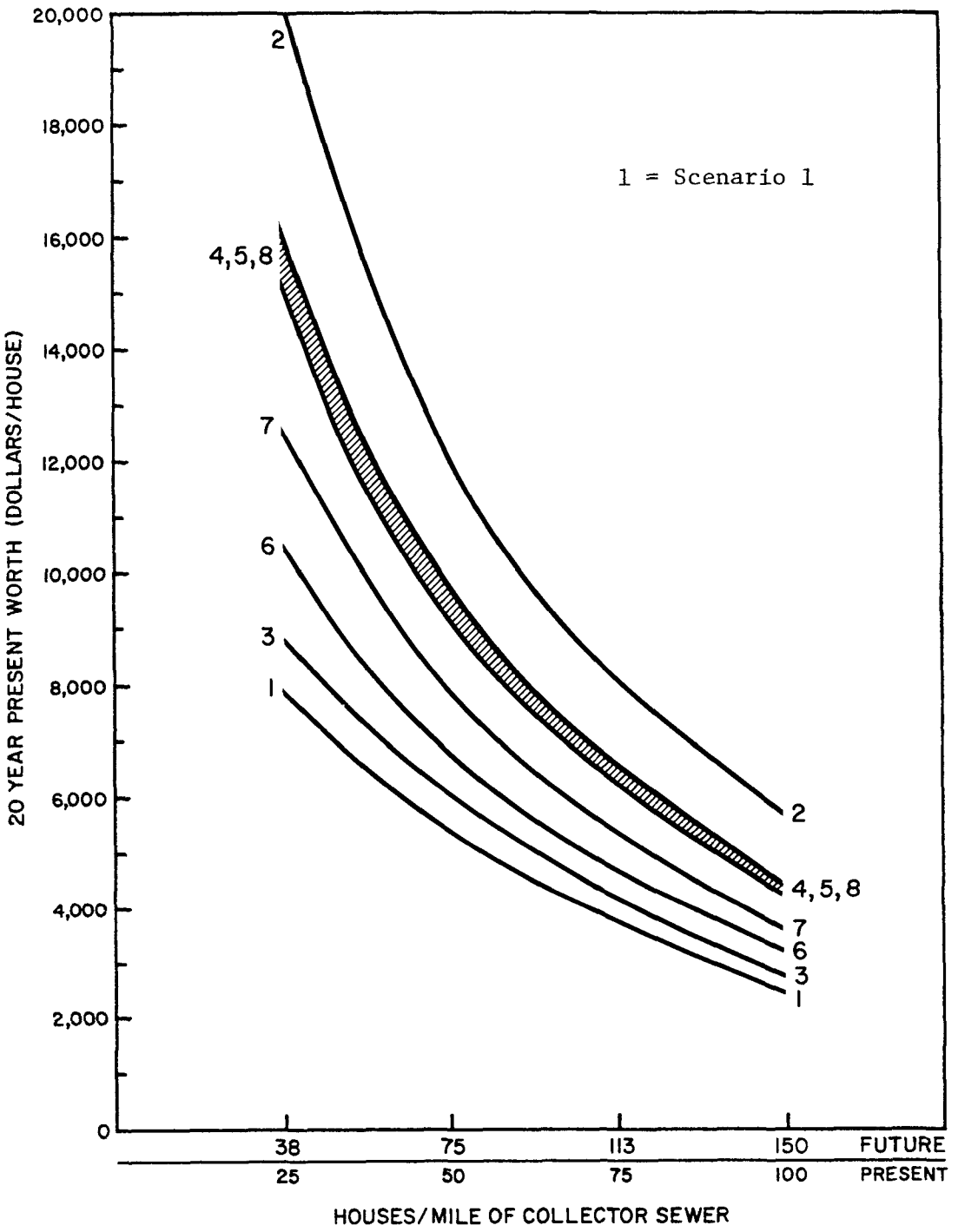

Figure IV-A-4-12. Costs of small diameter gravity sewers with septic tanks, transmission, and treatment for 8 scenarios; 50% growth.

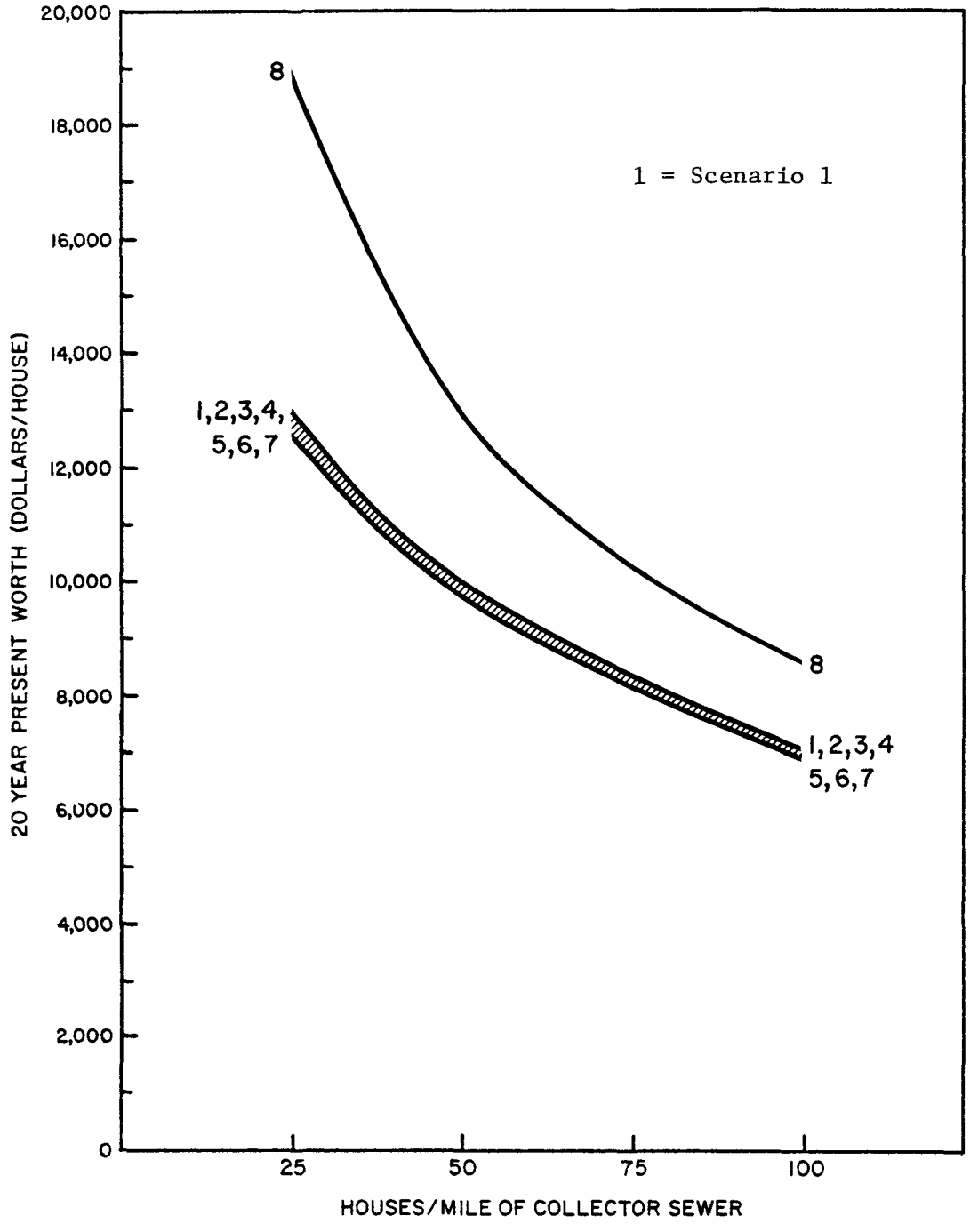

Figure IV-A-4-13. Costs of pressure sewers with septic tank effluent pumps, transmission, and treatment for 8 scenarios; 0% growth.

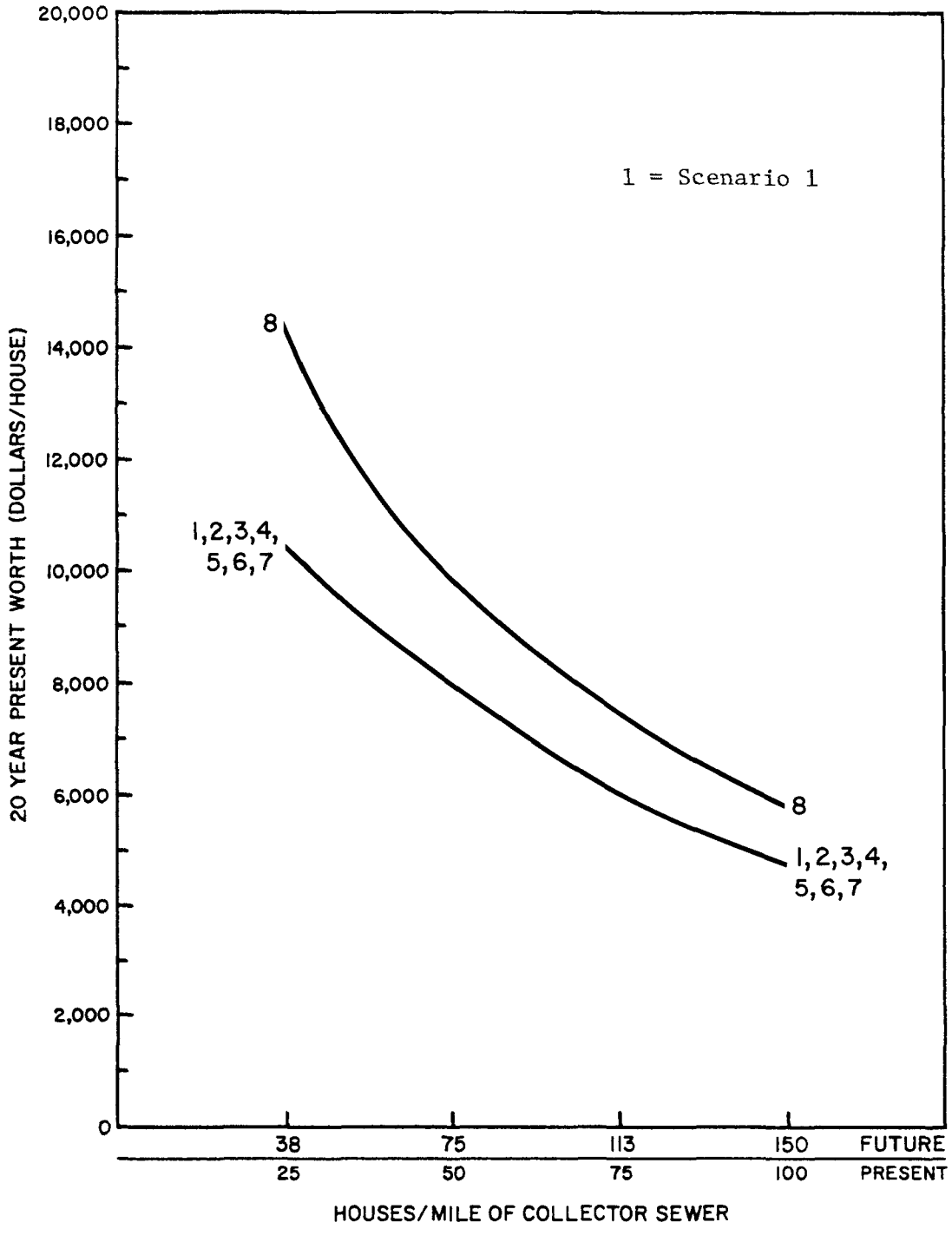

Figure IV-A-4-14. Costs of pressure sewers with septic tank effluent pumps, transmission, and treatment for 8 scenarios; 50% growth.

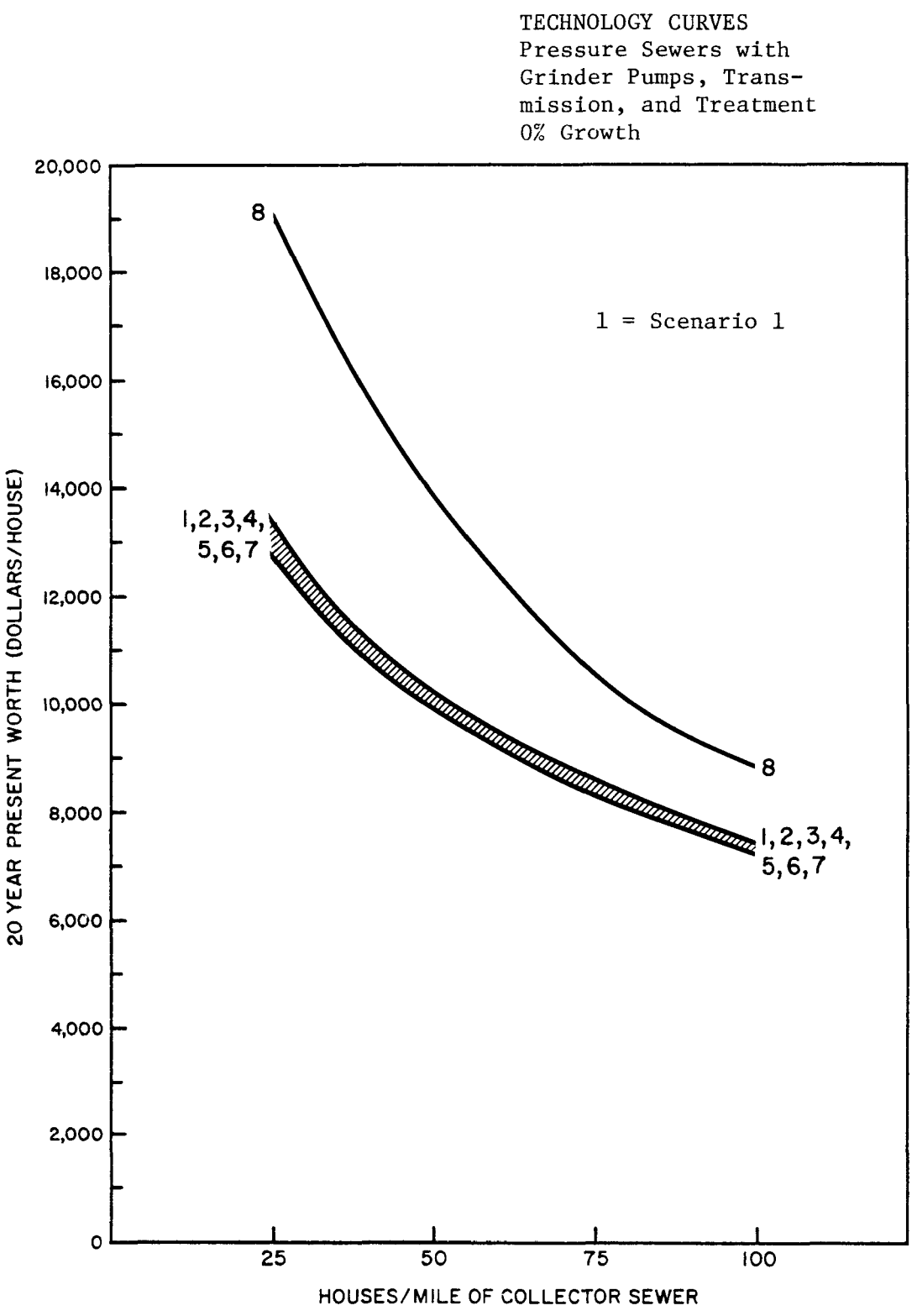

Figure IV-A-4-15. Costs of pressure sewers with grinder pumps, transmission, and treatment for 8 scenarios; 0% growth.

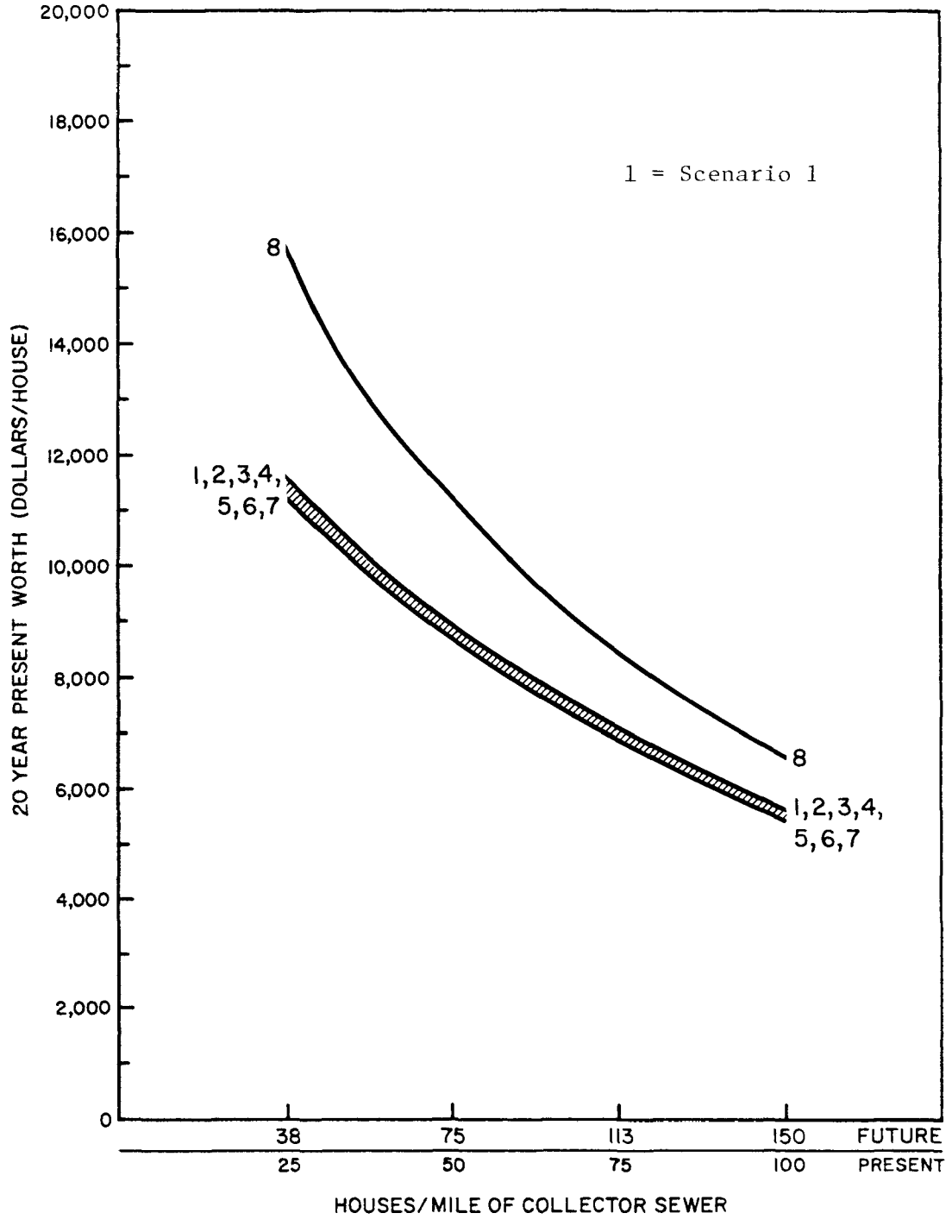

Figure IV-A-4-16. Costs of pressure sewers with grinder pumps, transmission, and treatment for 8 scenarios; 50% growth.

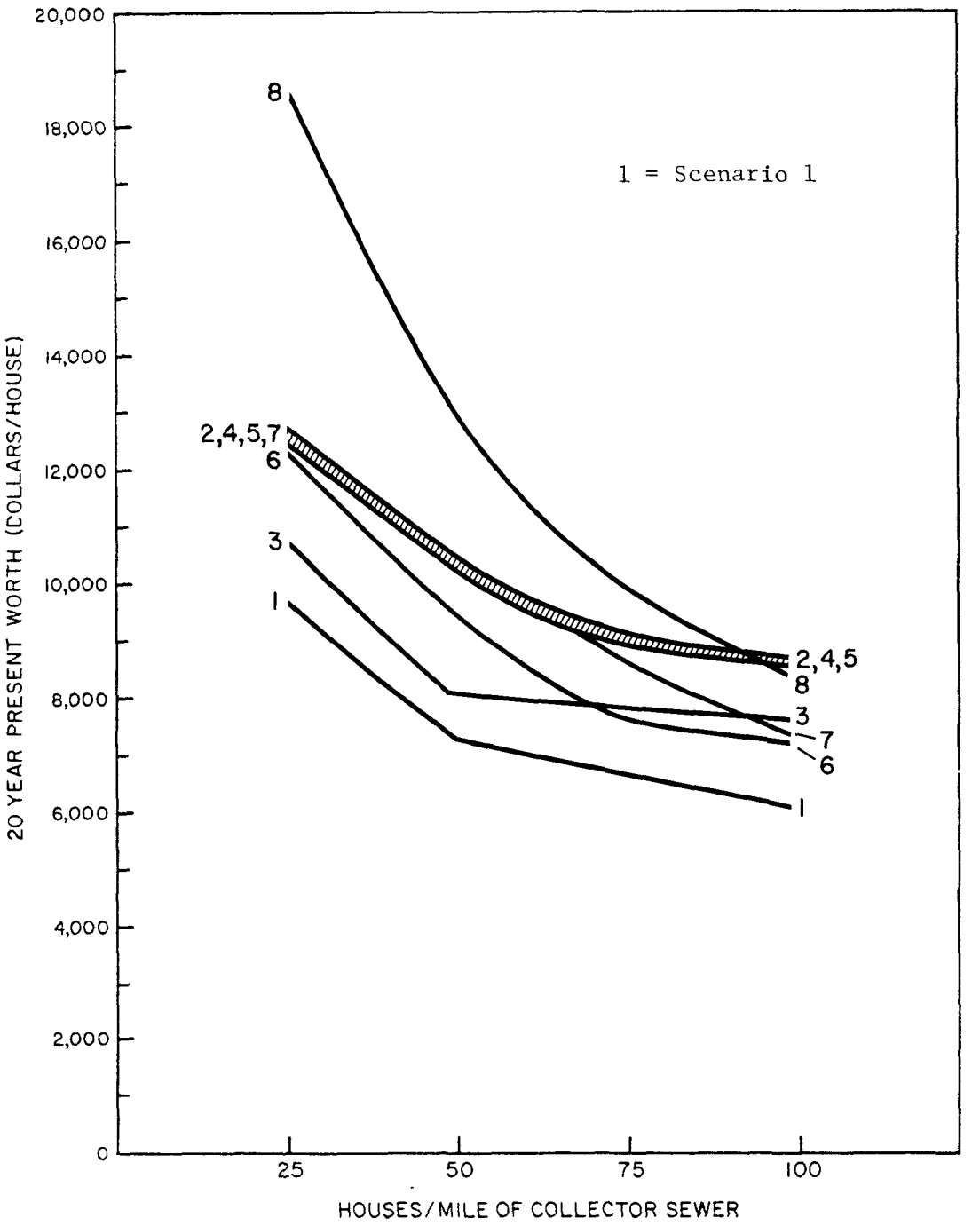

Figure IV-A-4-17. Costs of collection, transmission, and land application @ rapid infiltration for 8 scenarios; 0% growth. Collection methods by density included in these systems are identified in Table IV-A-4-10 (Appendix pg. B-9).

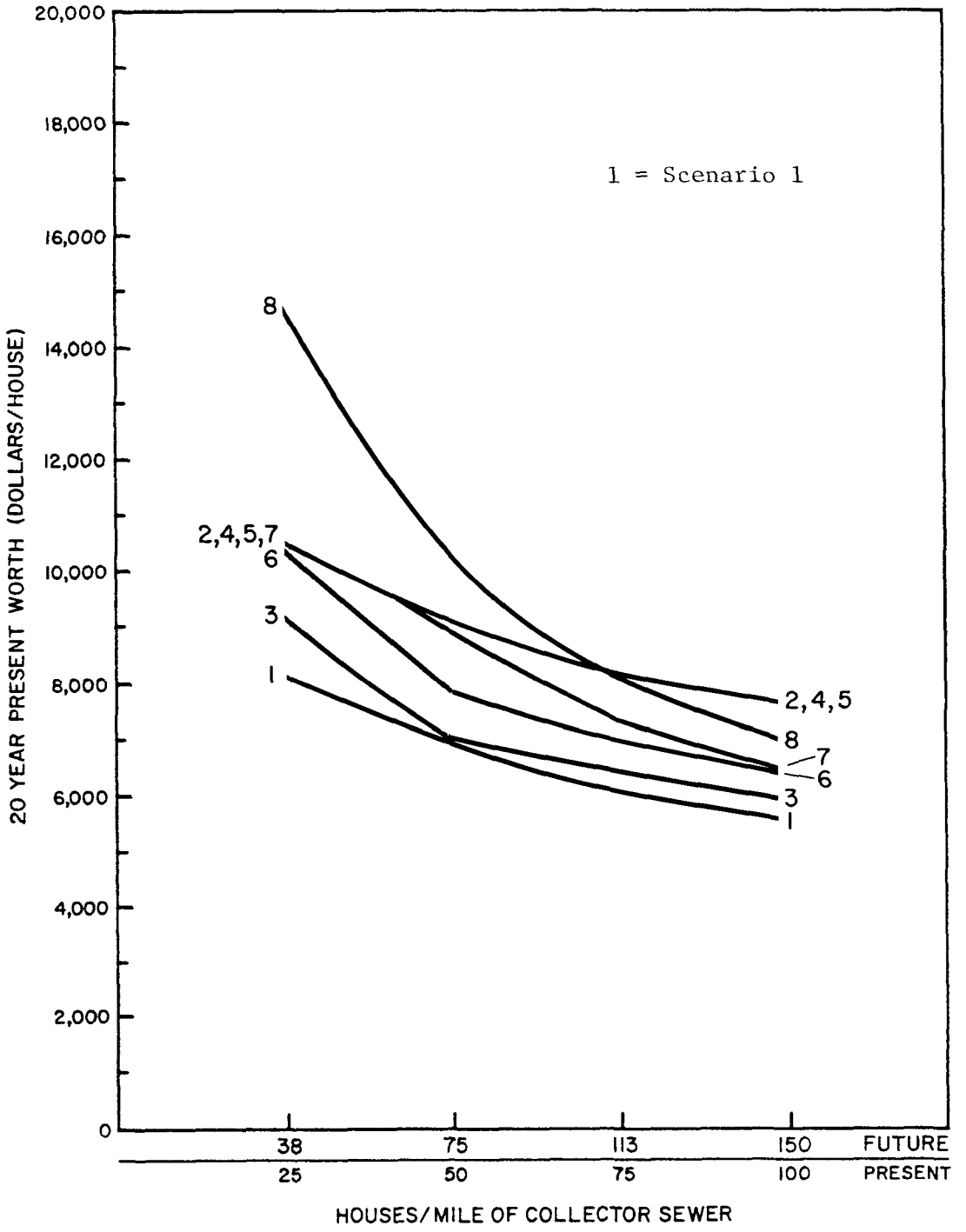

Figure IV-A-4-18. Costs of collection, transmission, and land application @ rapid infiltration for 8 scenarios; 50% growth. Collection methods by density included in these systems are identified in Table IV-A-4-10 (Appendix pg. B-9).

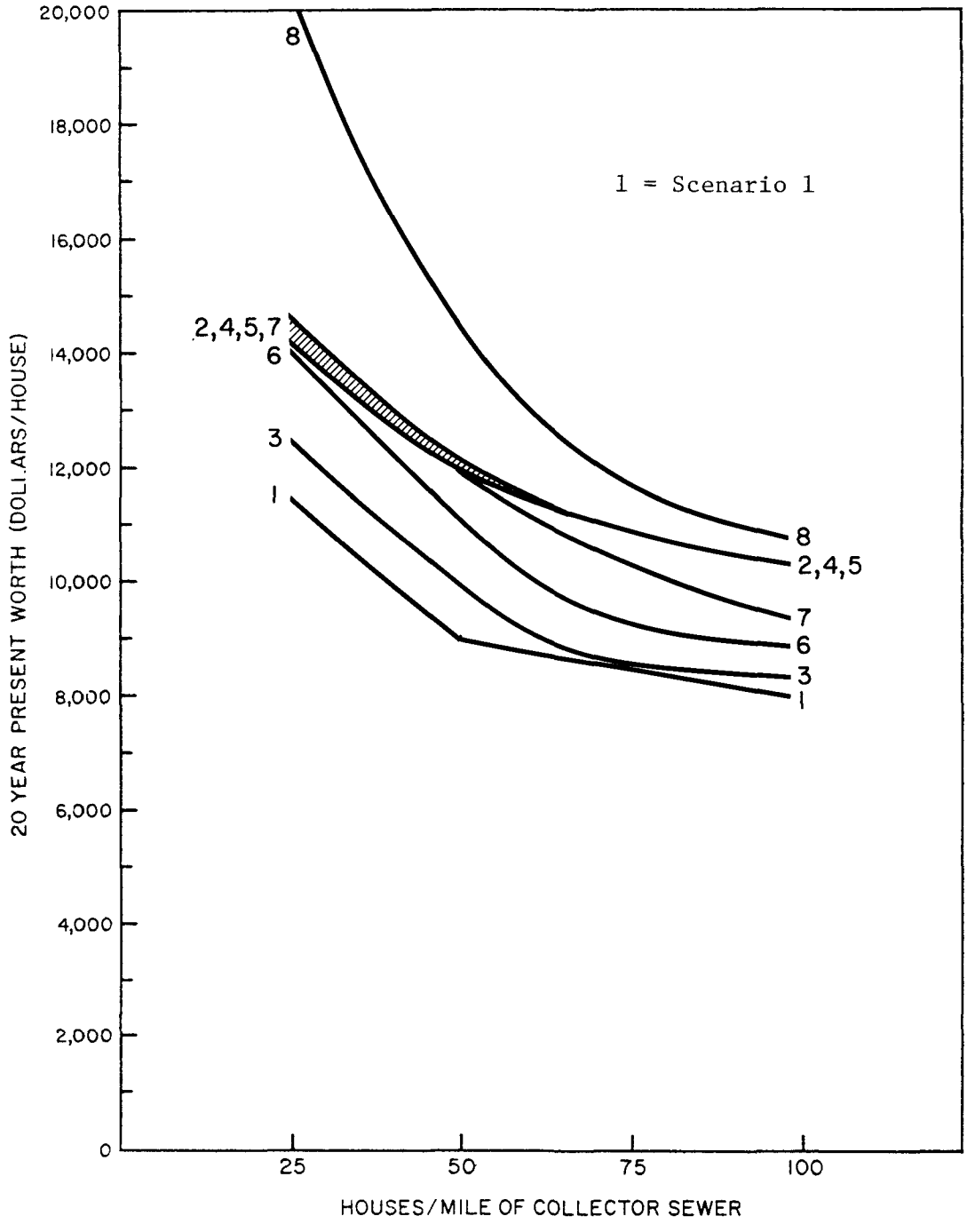

Figure IV-A-4-19. Costs of collection, transmission, and land application @ slow rate for 8 scenarios; 0% growth. Collection methods by density included in these systems are identified in Table IV-A-4-11 (Appendix pg. B-10).

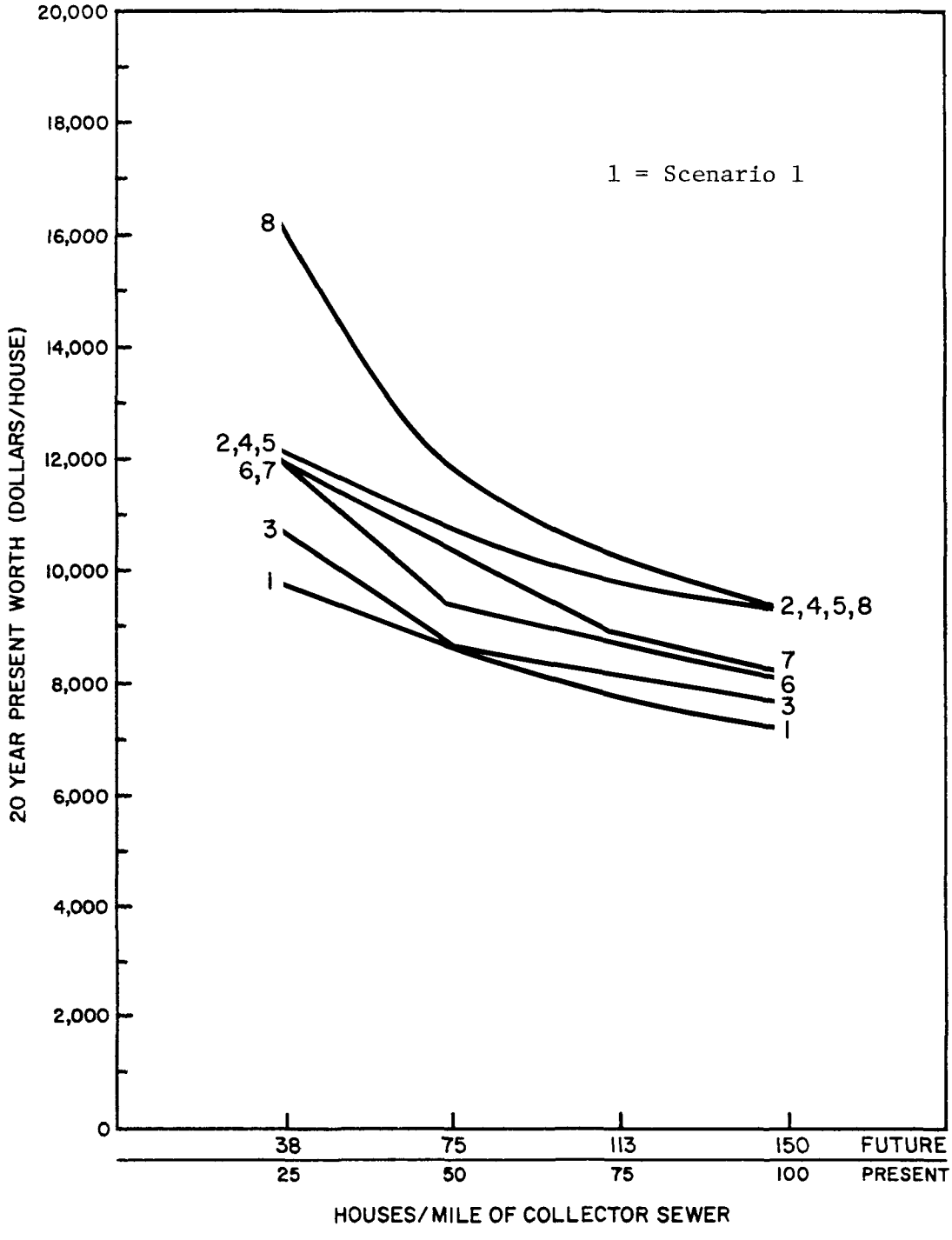

Figure IV-A-4-20. Costs of collection, transmission, and land application @ slow rate for 8 scenarios; 50% growth. Collection methods by density included in these systems are identified in Table IV-A-4-11 (Appendix pg. B-10).

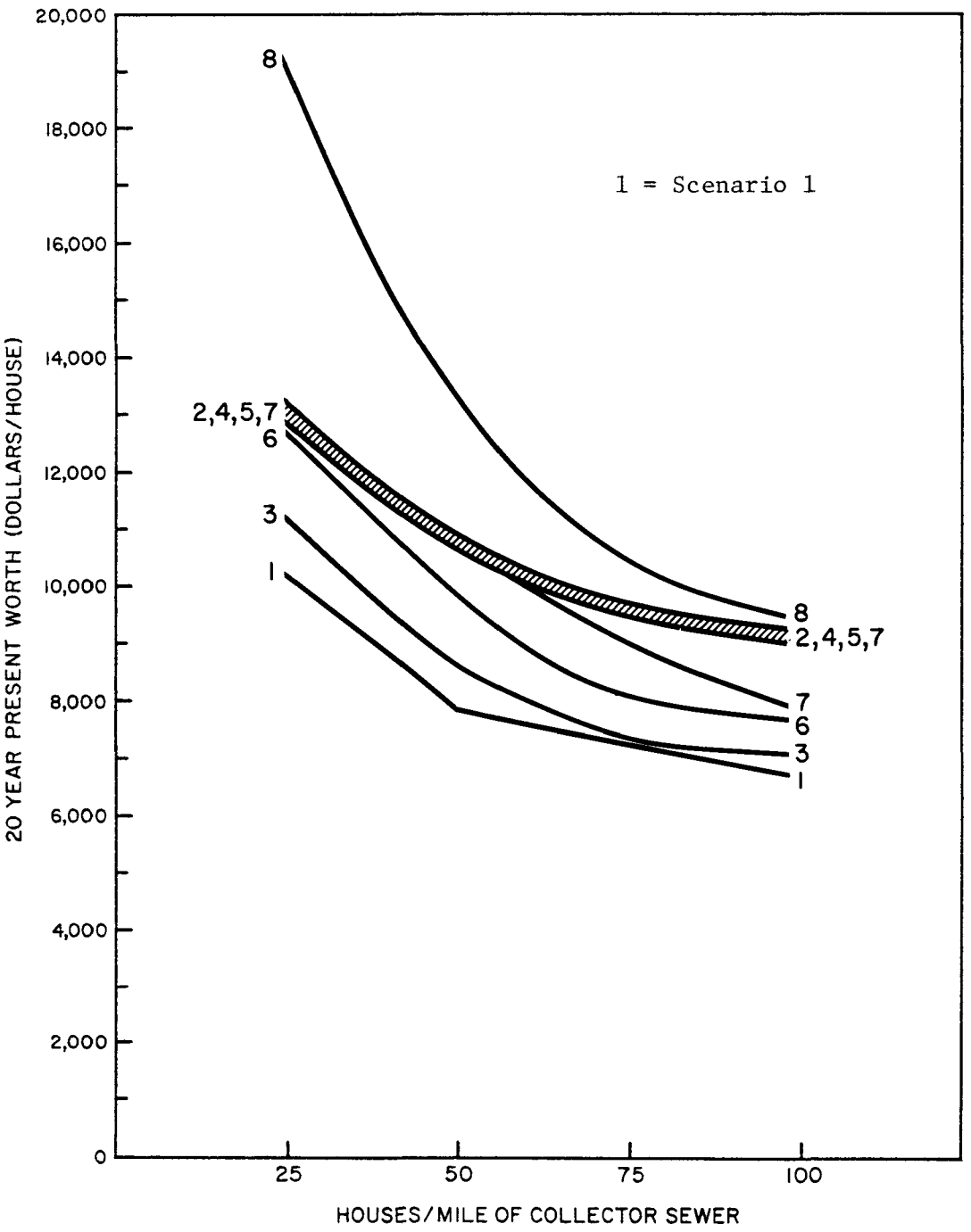

Figure IV-A-4-21. Costs of collection, transmission, and land application @ overland flow for 8 scenarios; 0% growth. Collection methods by density included in these systems are identified in Table IV-A-4-12 (Appendix pg. B-11).

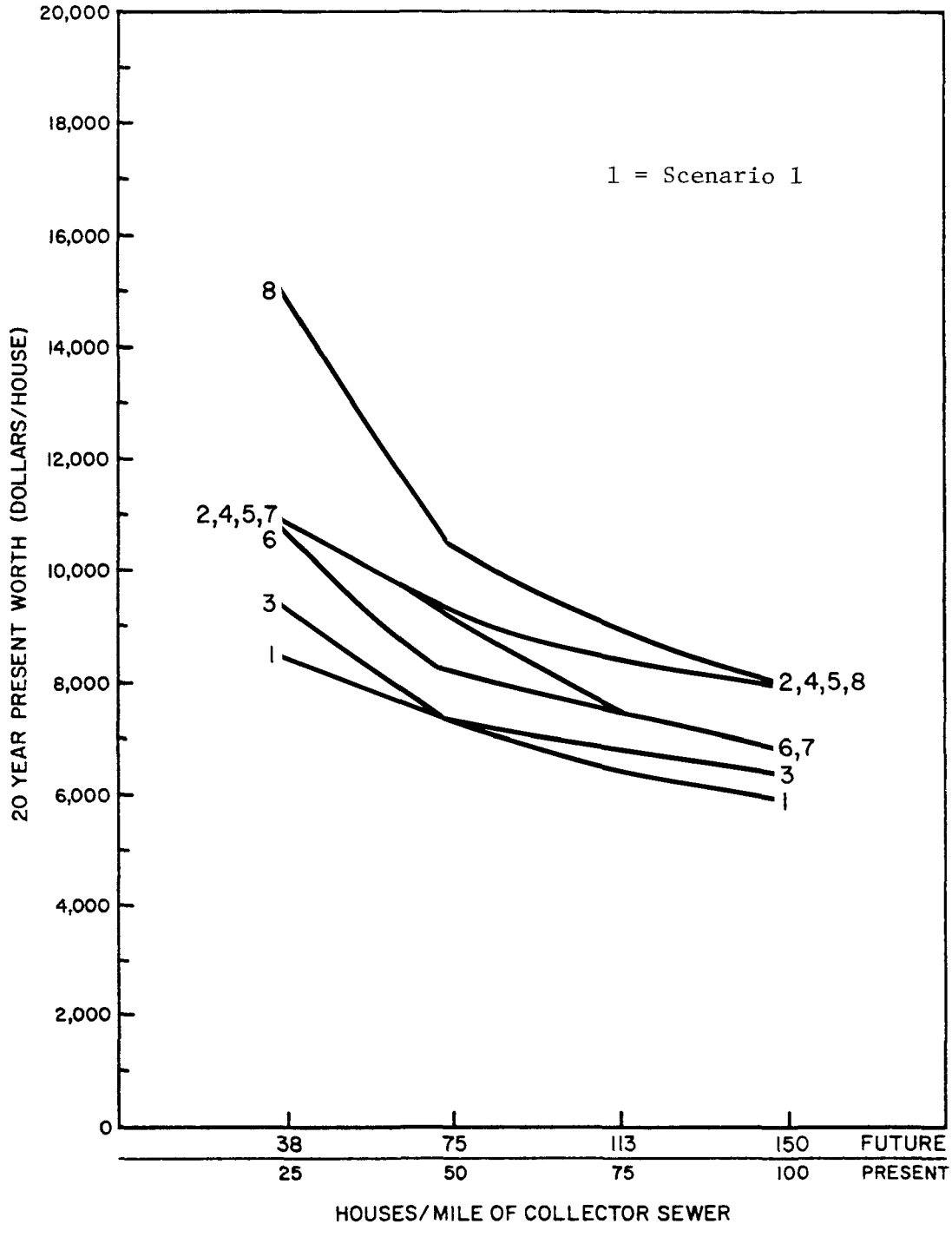

Figure IV-A-4-22. Costs of collection, transmission, and land application @ overland flow for 8 scenarios; 50% growth. Collection methods by density included in these systems are identified in Table IV-A-4-12 (Appendix pg. B-11).

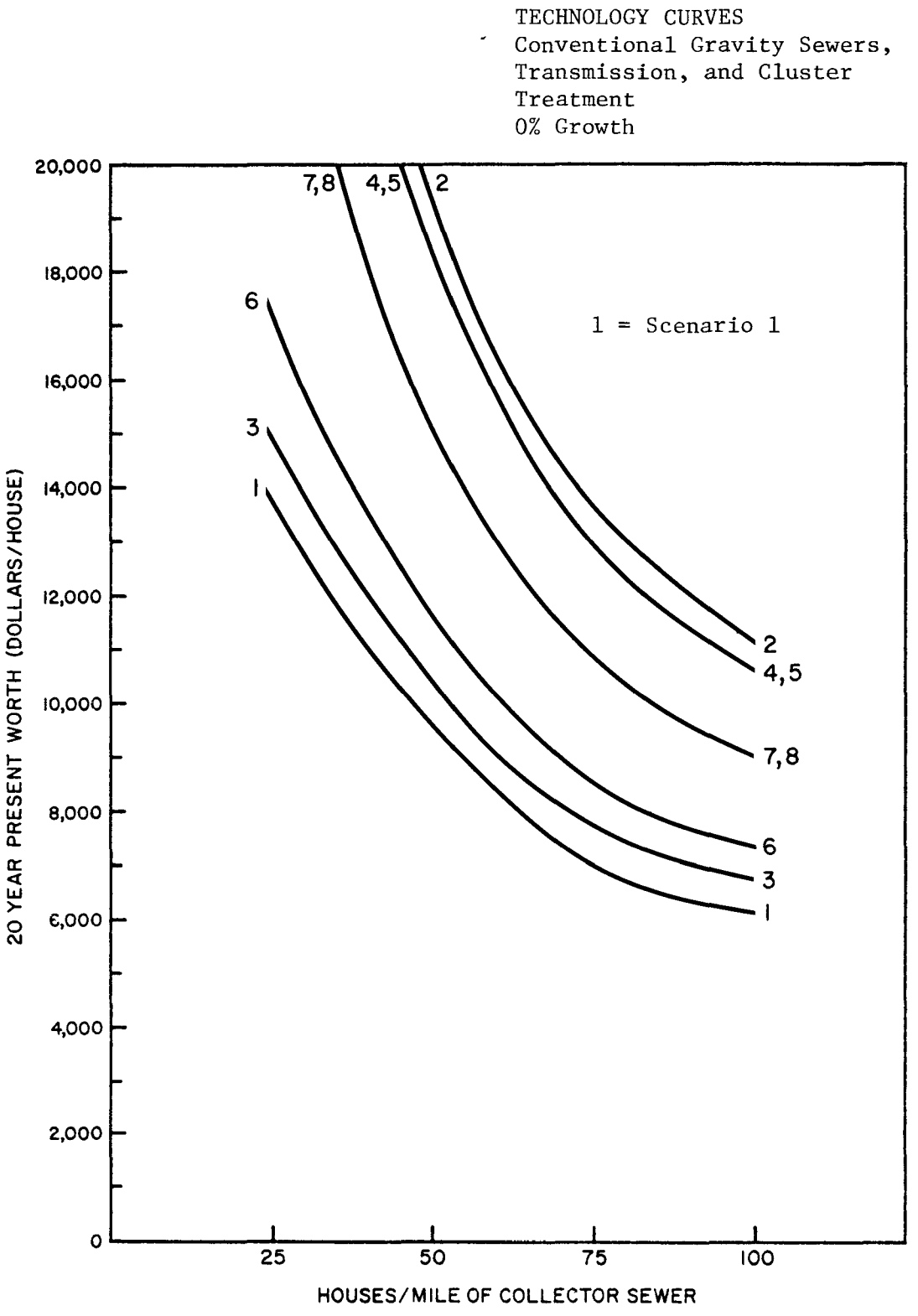

Figure IV-A-4-23. Costs of conventional gravity sewers, transmission, and cluster treatment for 8 scenarios; 0% growth.

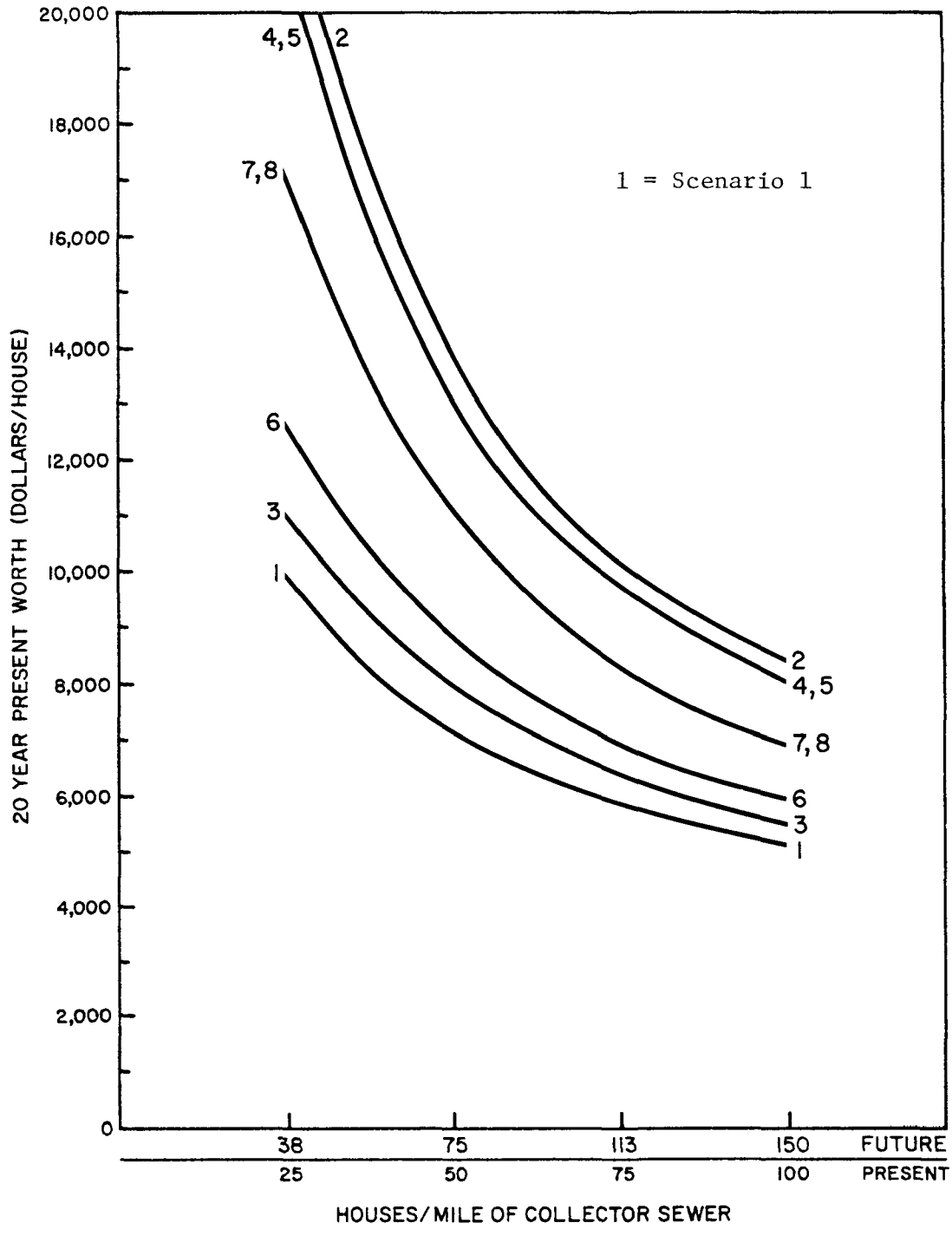

Figure IV-A-4-24. Costs of conventional gravity sewers, transmission, and cluster treatment for 8 scenarios; 50% growth.

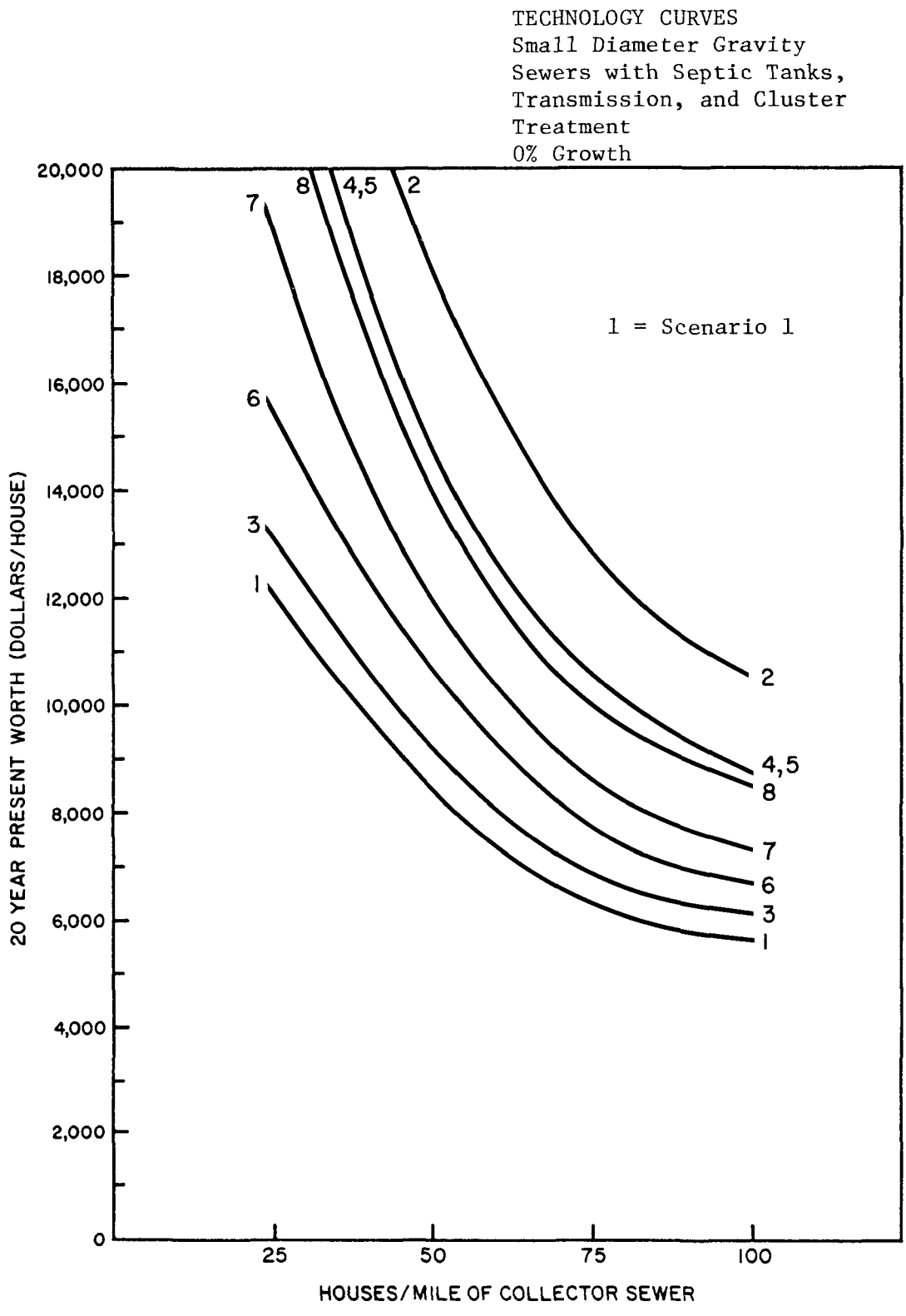

Figure IV-A-4-25. Costs of small diameter gravity sewers with septic tanks, transmission, and cluster treatment for 8 scenarios; 0% growth.

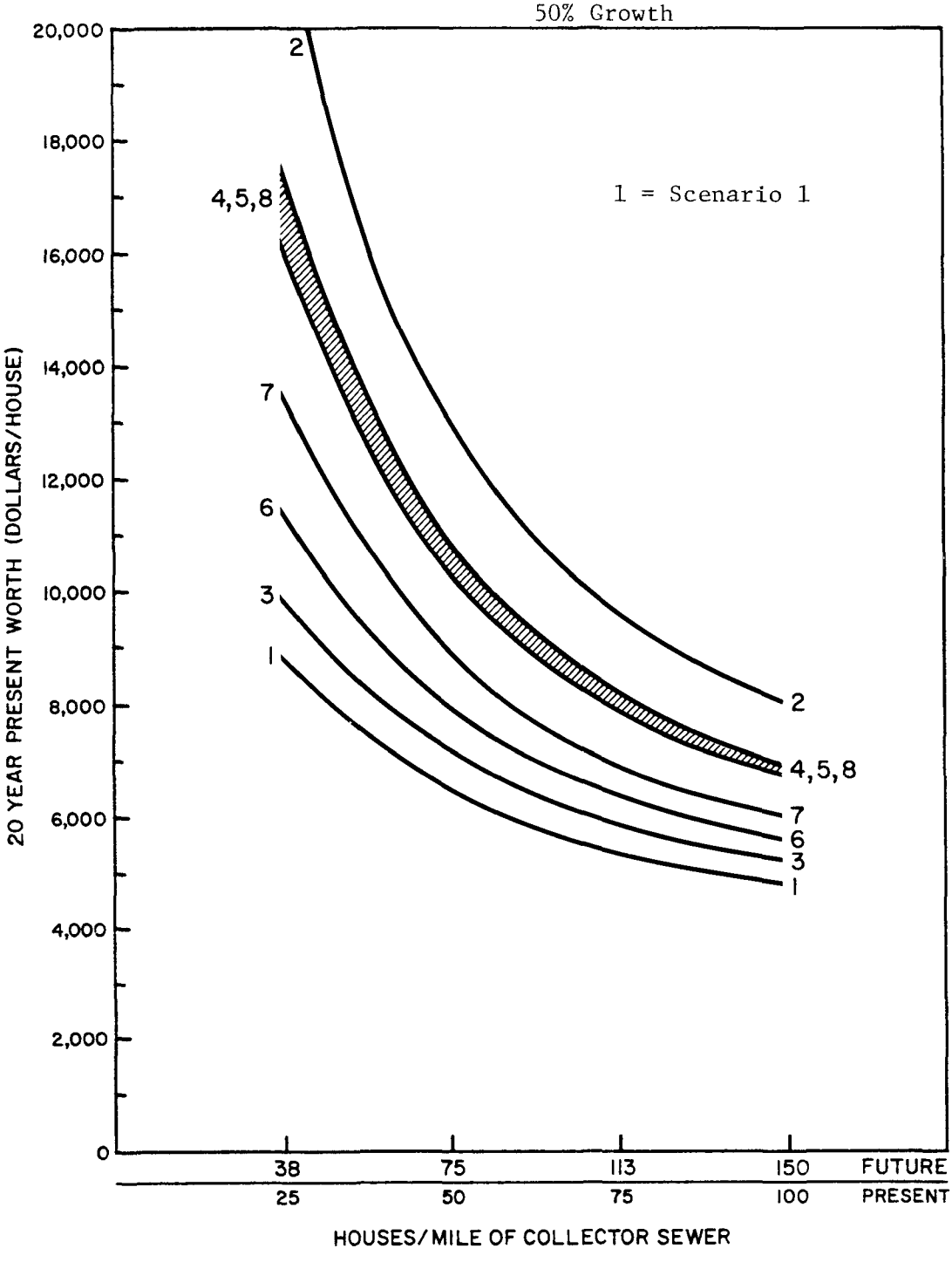

Figure IV-A-4-26. Costs of small diameter gravity sewers with septic tanks, transmission, and cluster treatment for 8 scenarios; 50% growth.

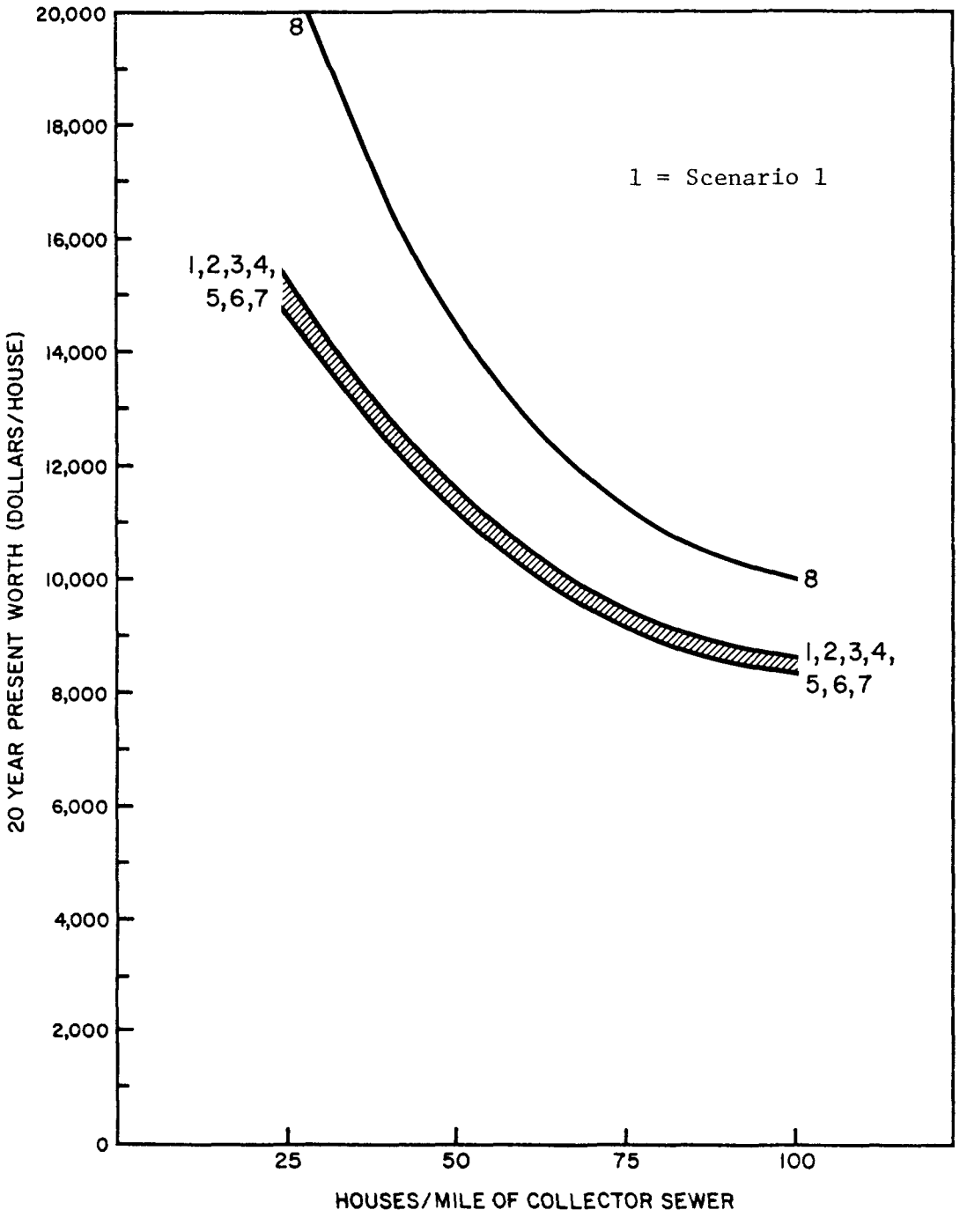

Figure IV-A-4-27. Costs of pressure sewers with septic tank effluent pumps, transmission, and cluster treatment for 8 scenarios; 0% growth.

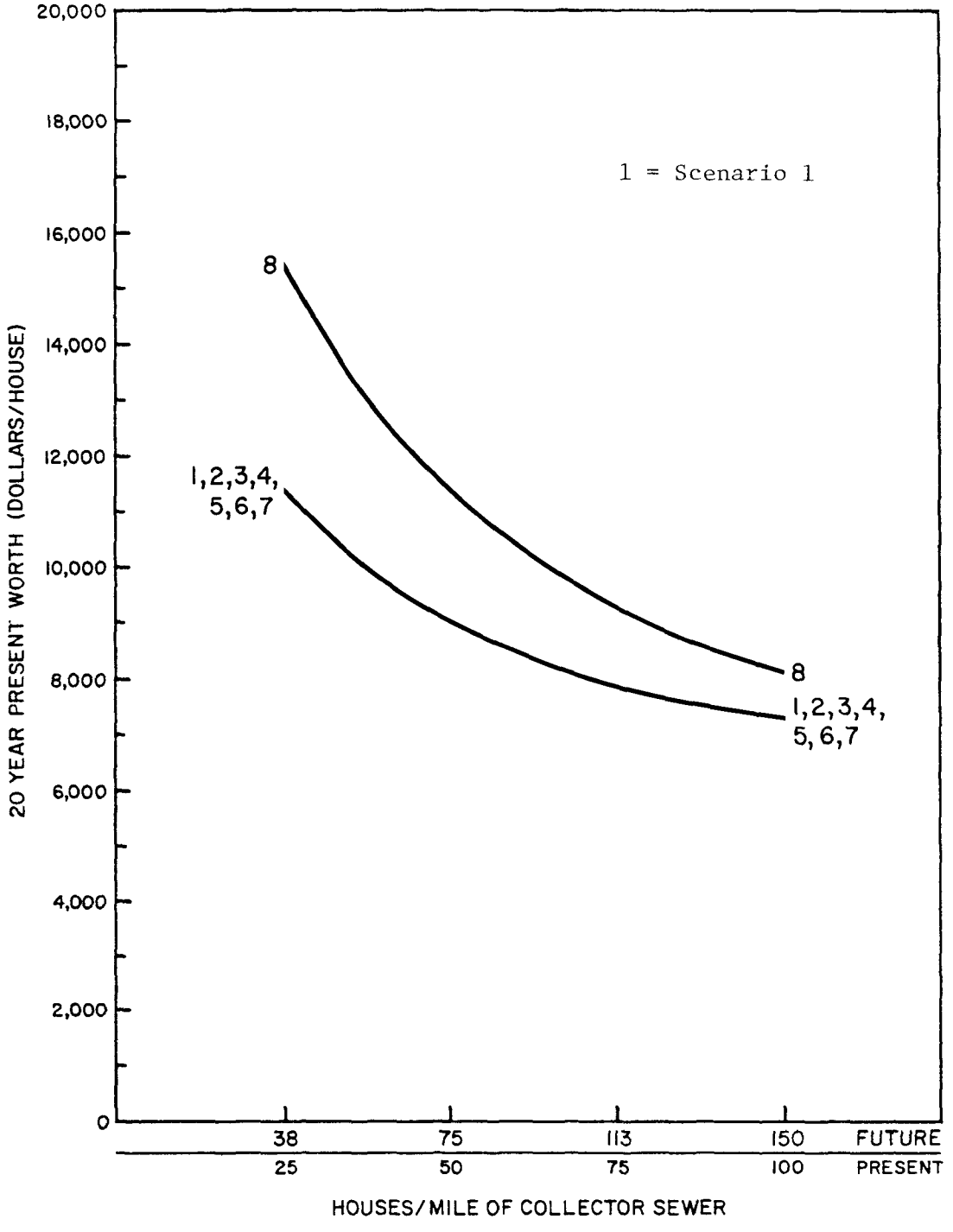

Figure IV-A-4-28. Costs of pressure sewers with septic tank effluent pumps, transmission, and cluster treatment for 8 scenarios; 50% growth.

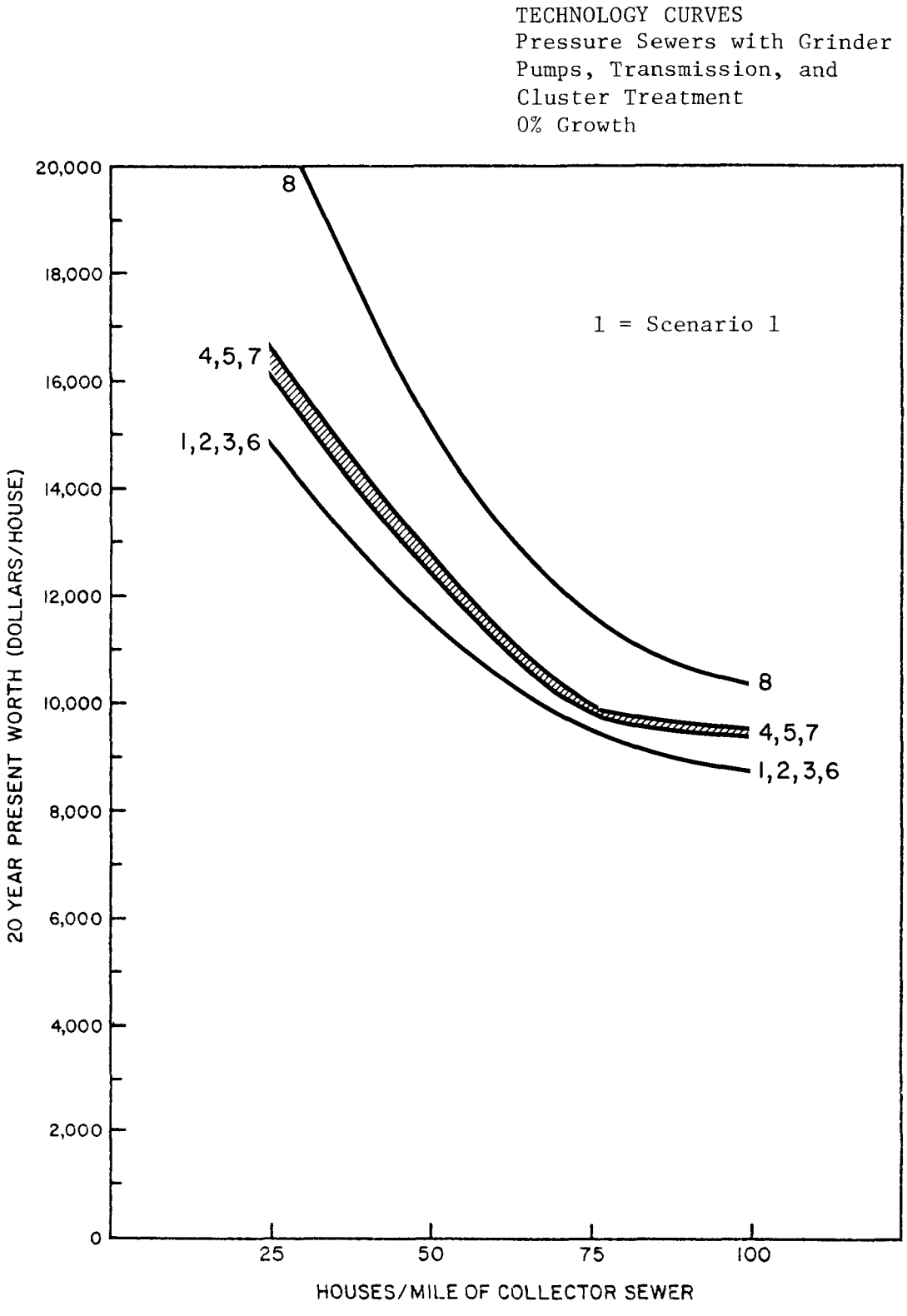

Figure IV-A-4-29. Costs of pressure sewers with grinder pumps, transmission, and cluster treatment for 8 scenarios; 0% growth.

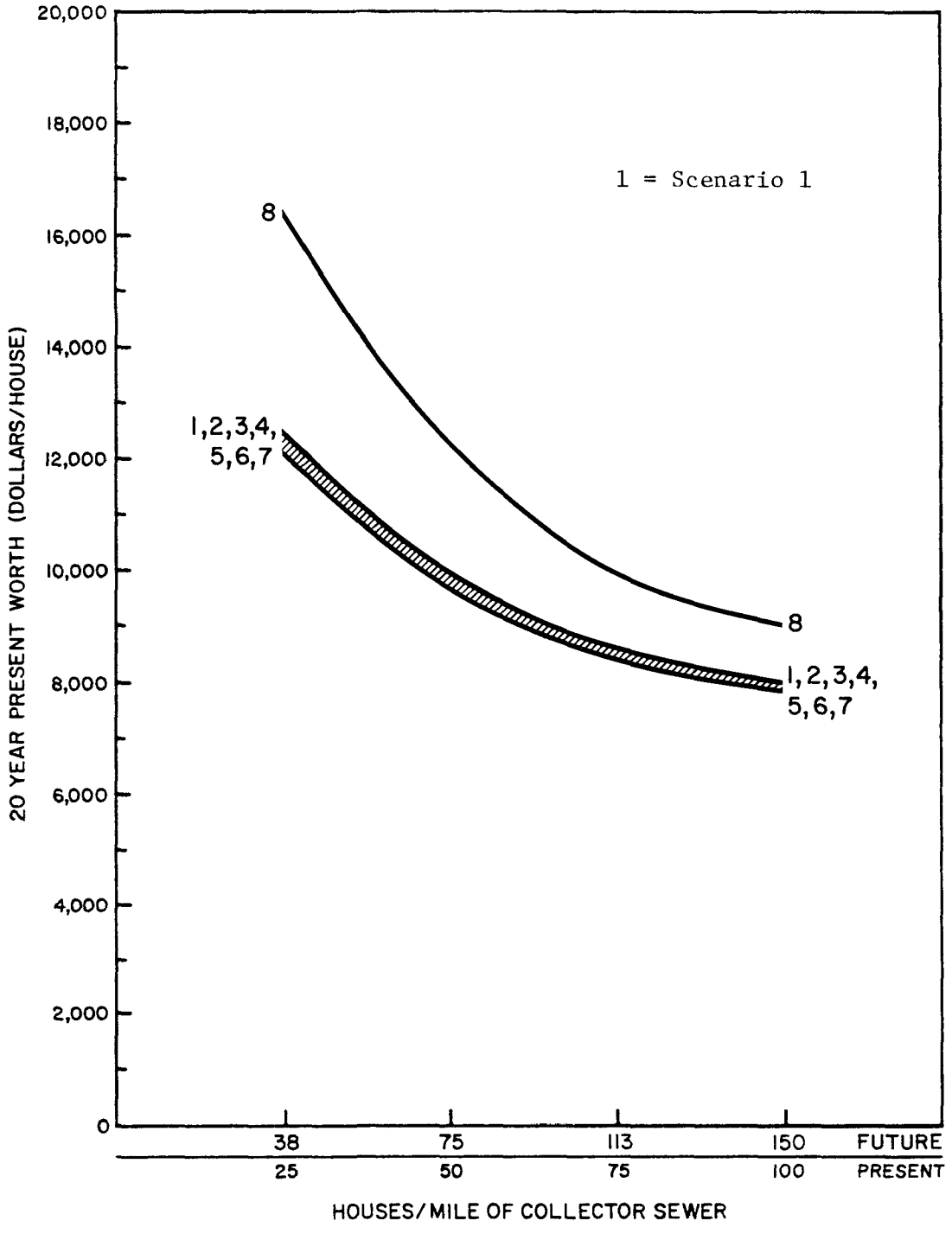

Figure IV-A-4-30. Costs of pressure sewers with grinder pumps, transmission, and cluster treatment for 8 scenarios; 50% growth.

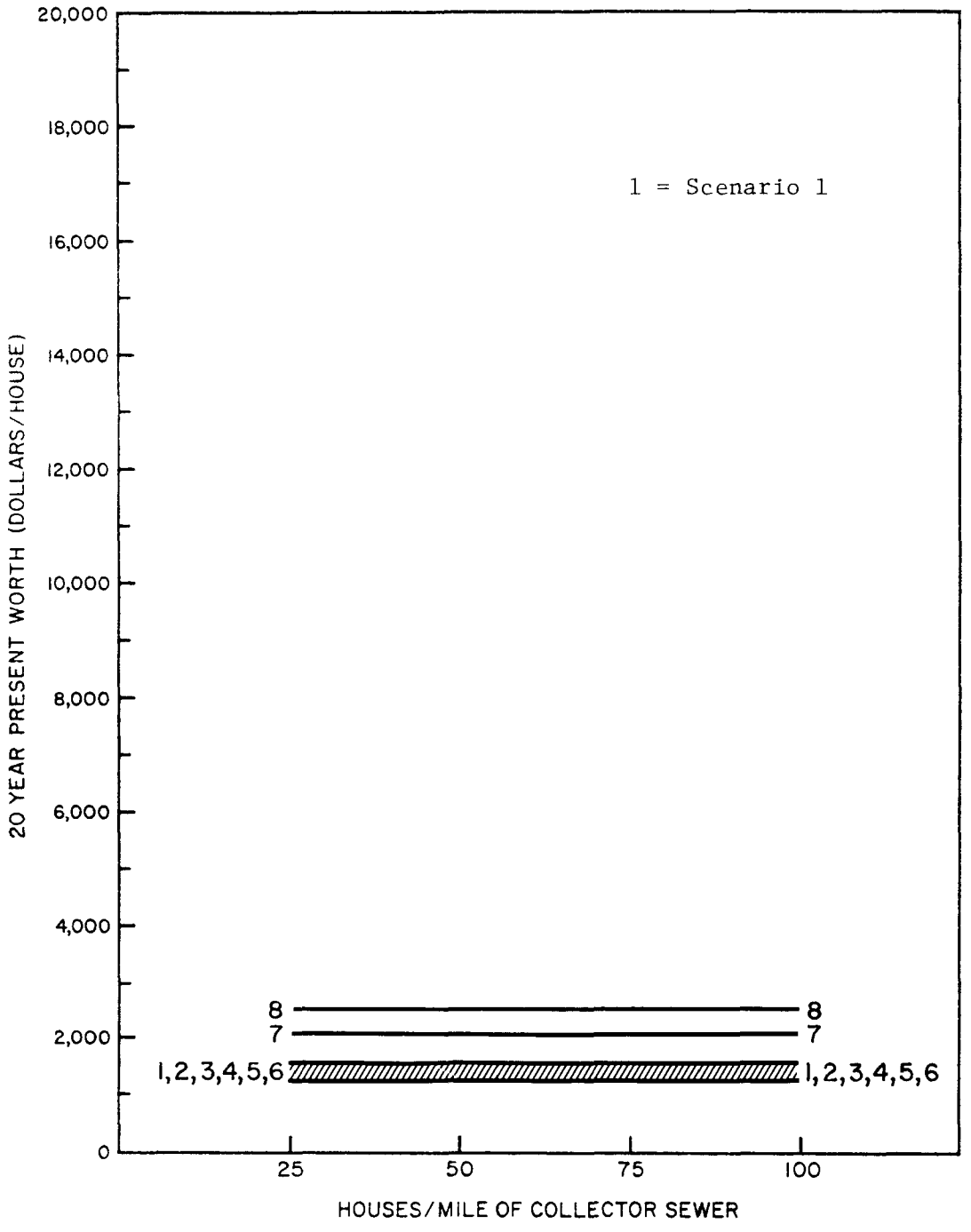

Figure IV-A-4-31. Costs for on-site systems for 8 scenarios, 10% replacement level; 0% growth.

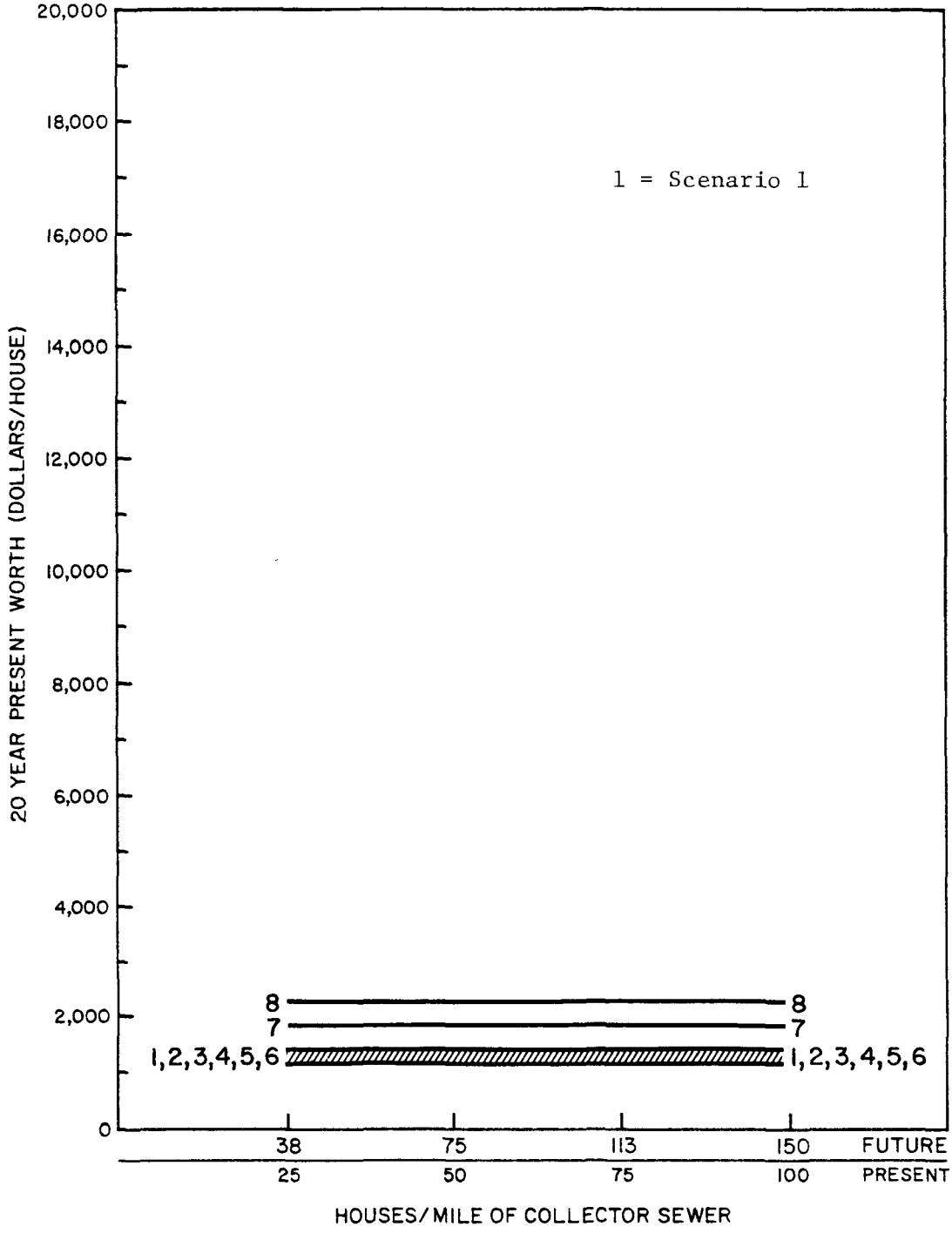

Figure IV-A-4-32. Costs for on-site systems for 8 scenarios, 10% replacement level; 50% growth.

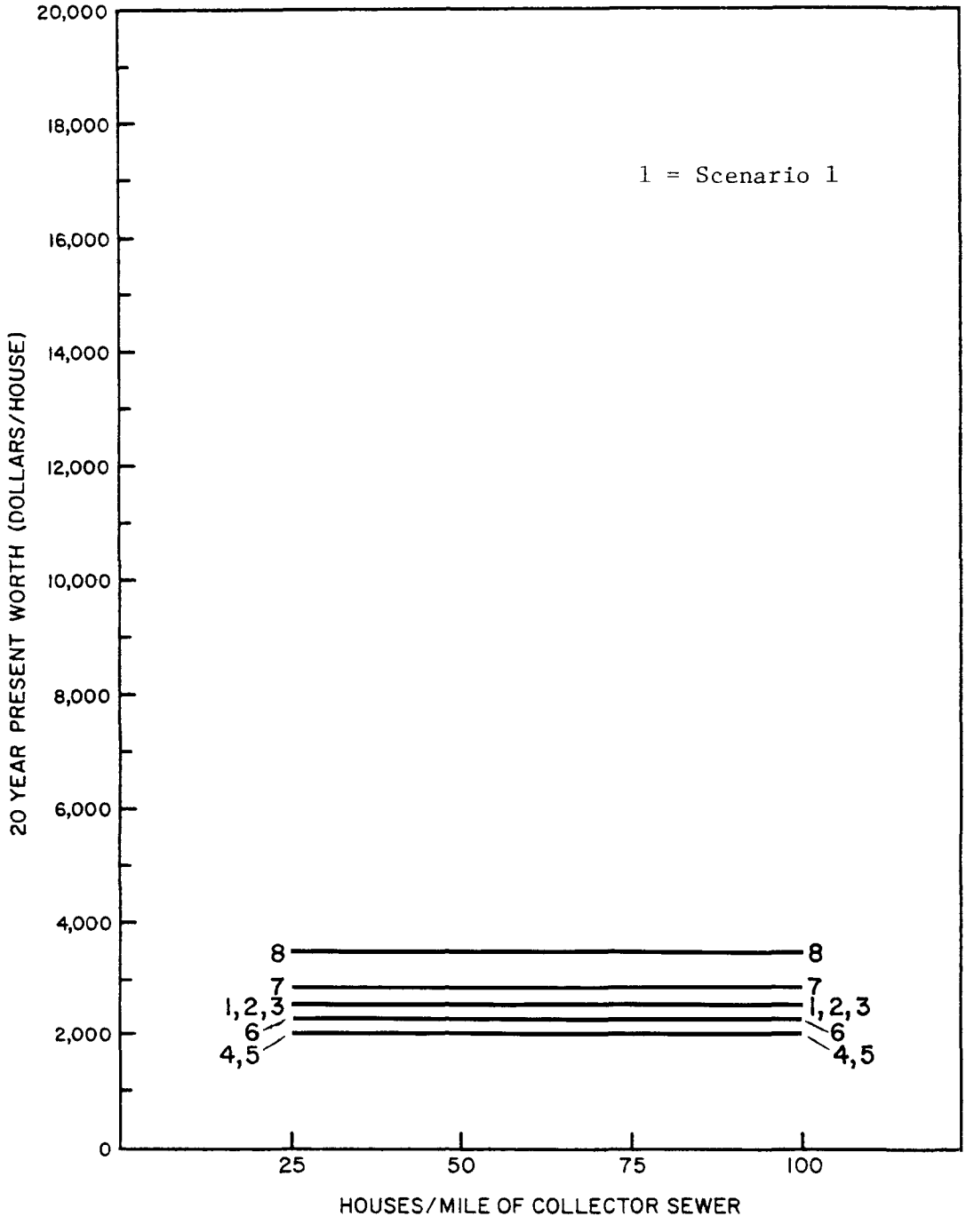

Figure IV-A-4-33. Costs for on-site systems for 8 scenarios, 20% replacement level; 0% growth.

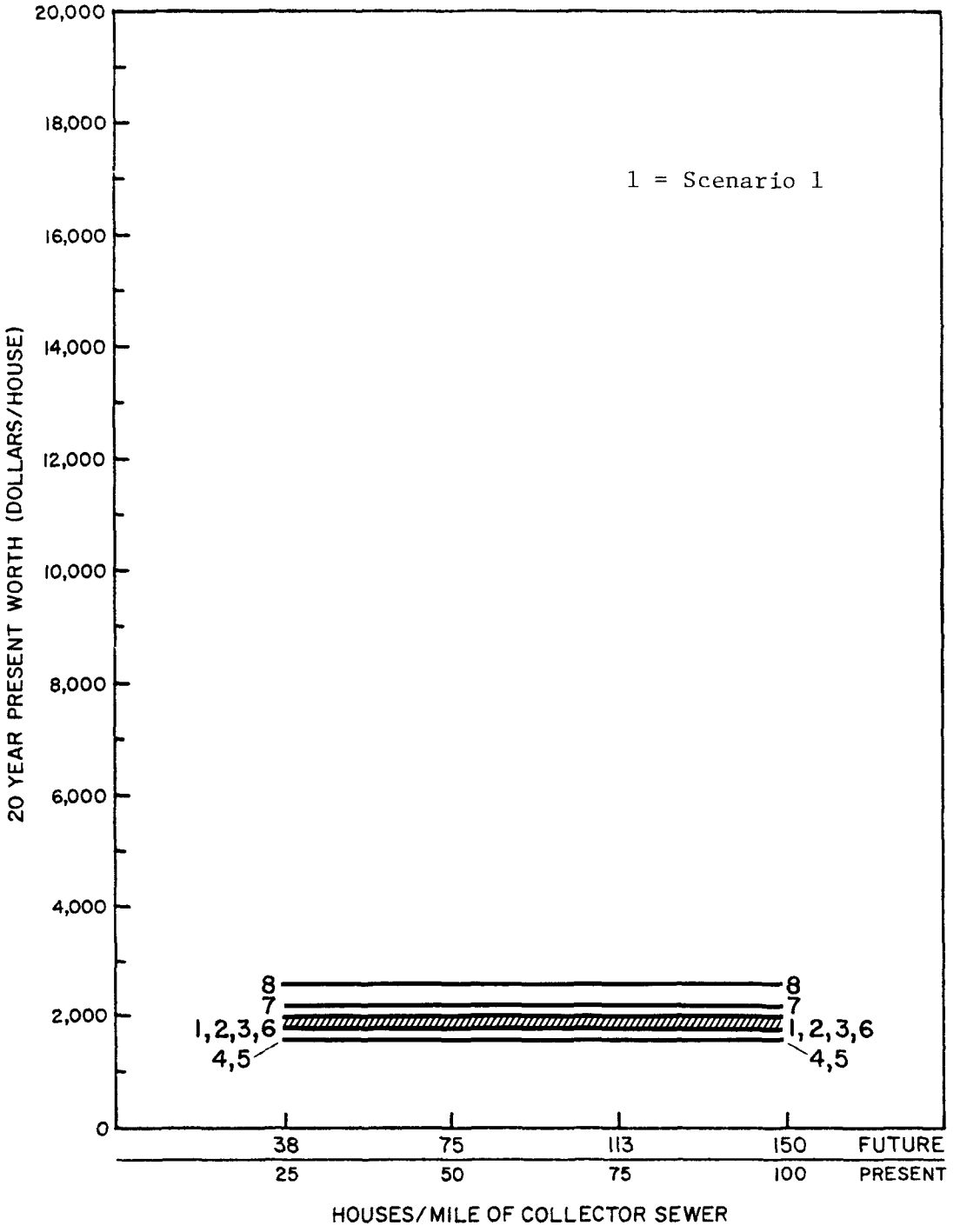

Figure IV-A-4-34. Costs for on-site systems for 8 scenarios, 20% replacement level; 50% growth.

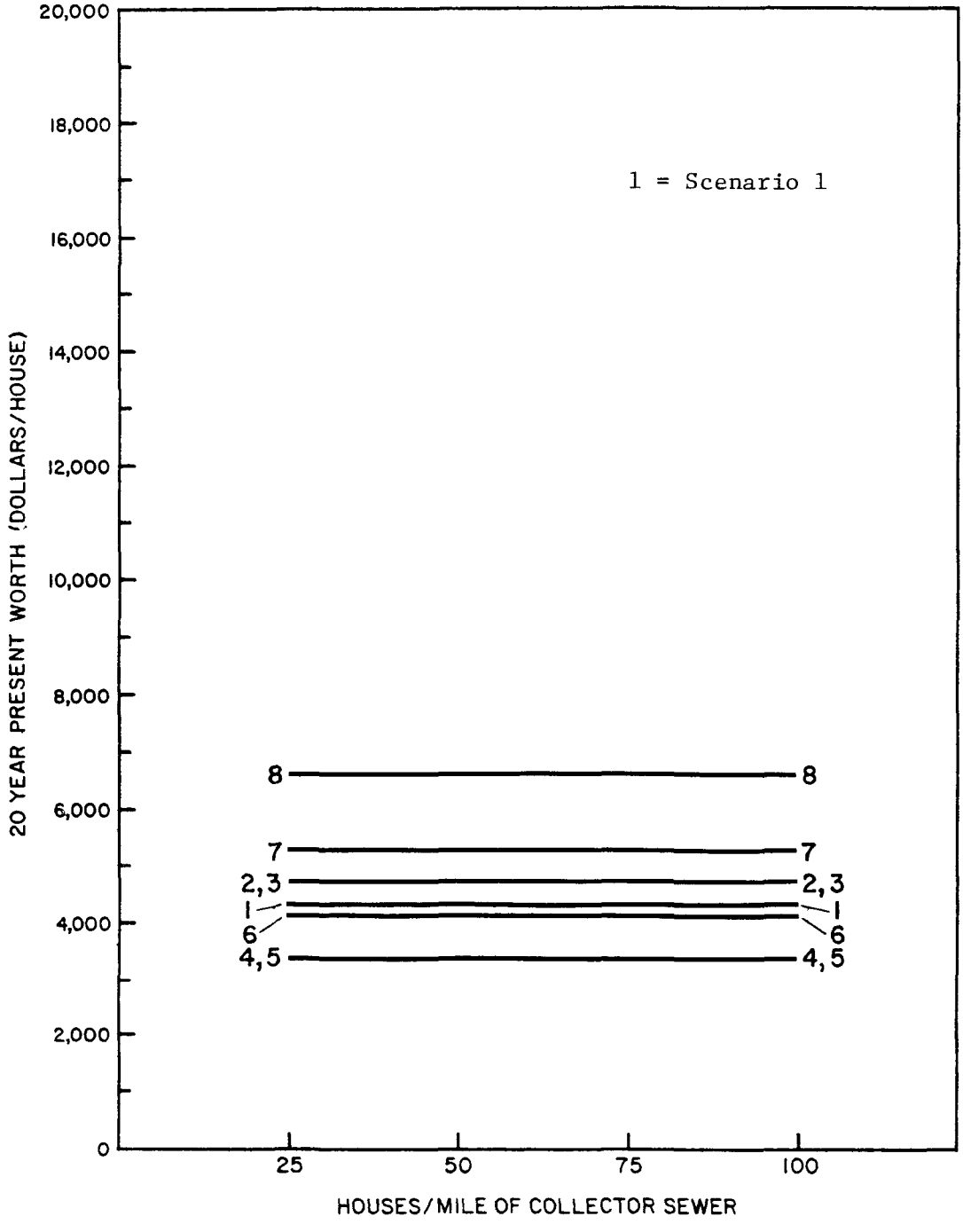

Figure IV-A-4-35. Costs for on-site systems for 8 scenarios, 50% replacement level; 0% growth.

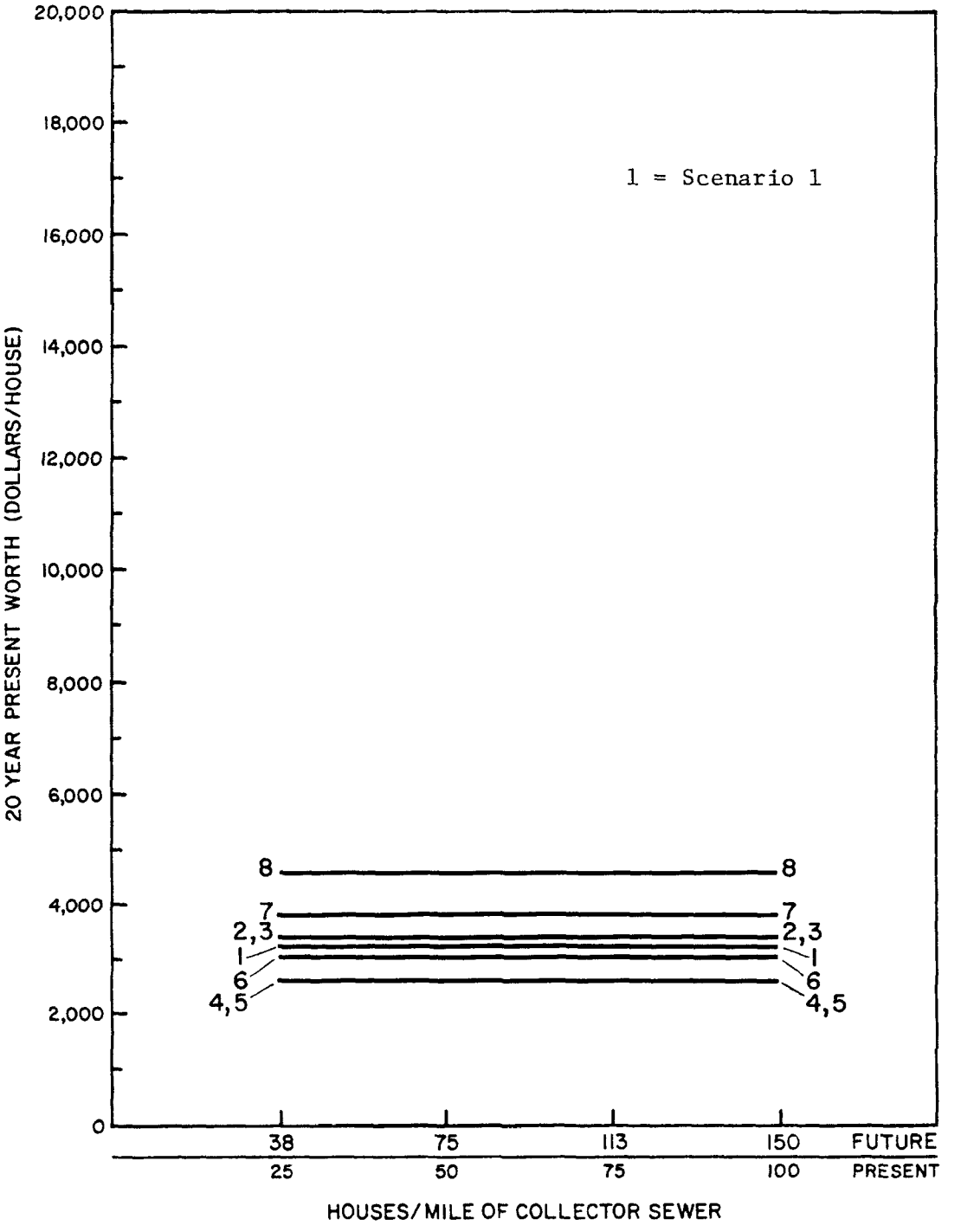

Figure IV-A-4-36. Costs for on-site systems for 8 scenarios, 50% replacement level; 50% growth.

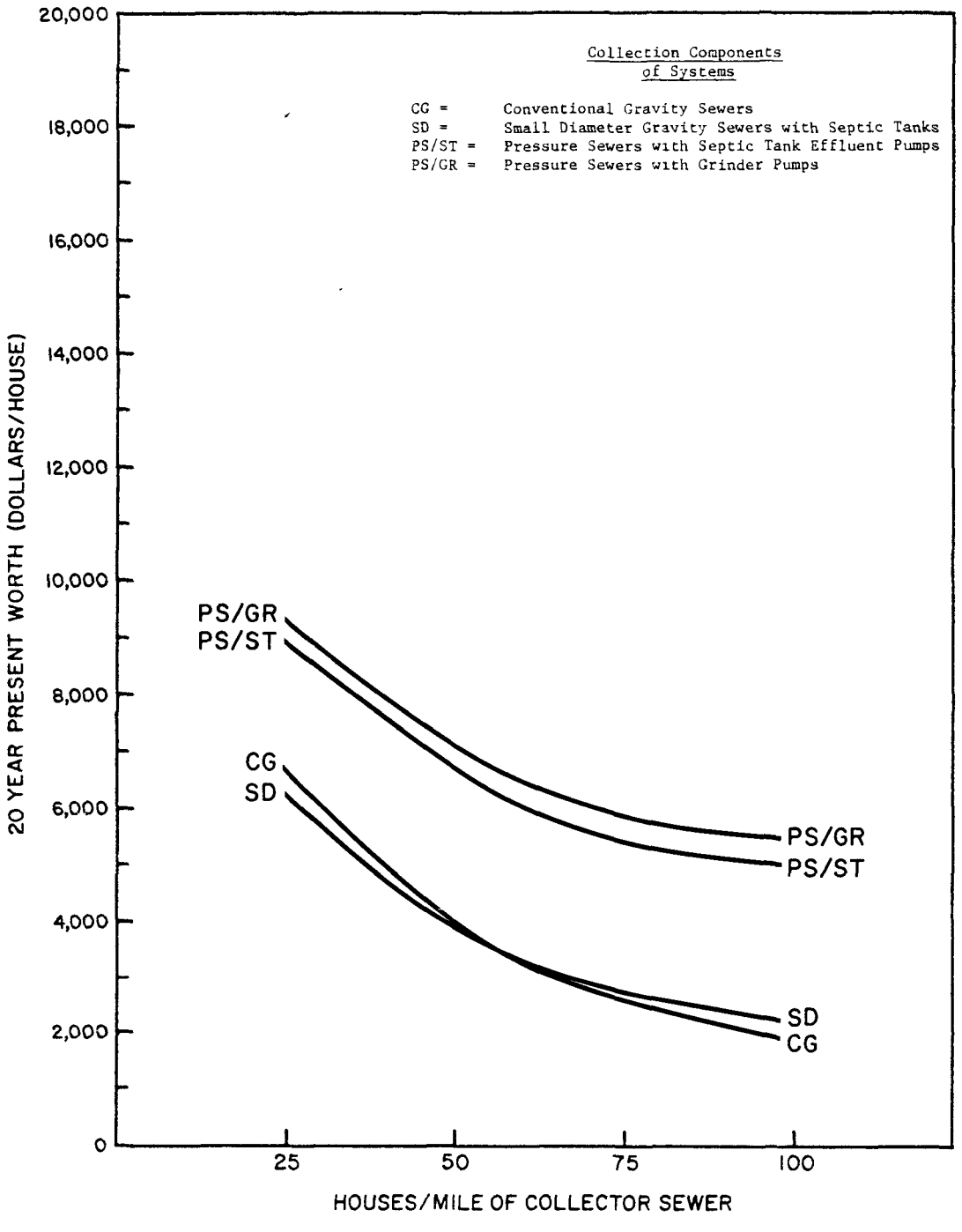

Figure IV-A-4-37. Cost comparison of collection technologies (no transmission/treatment) for Scenario 1; 0% growth.

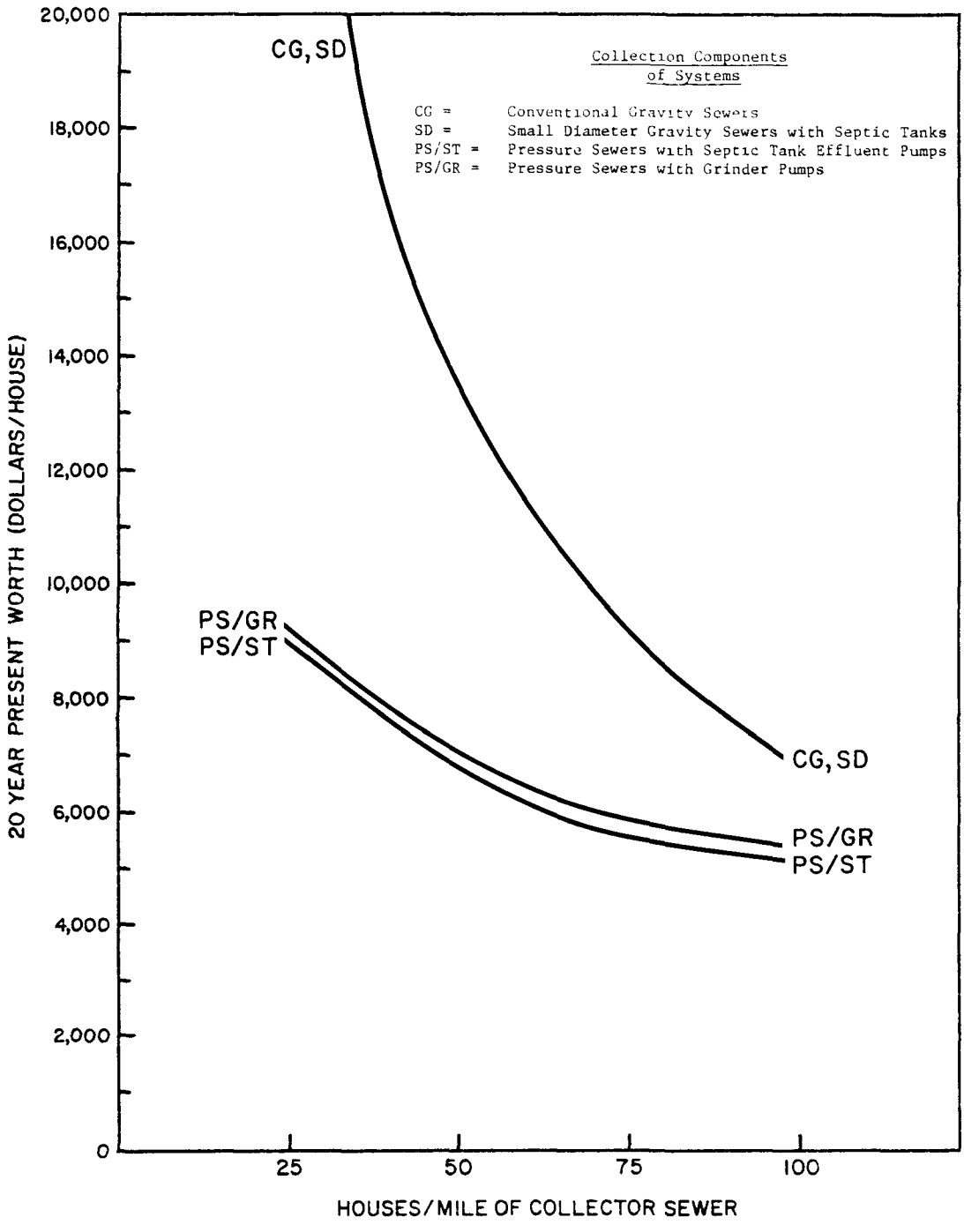

Figure IV-A-4-38. Cost comparison of collection technologies (no transmission/treatment) for Scenario 2; 0% growth.

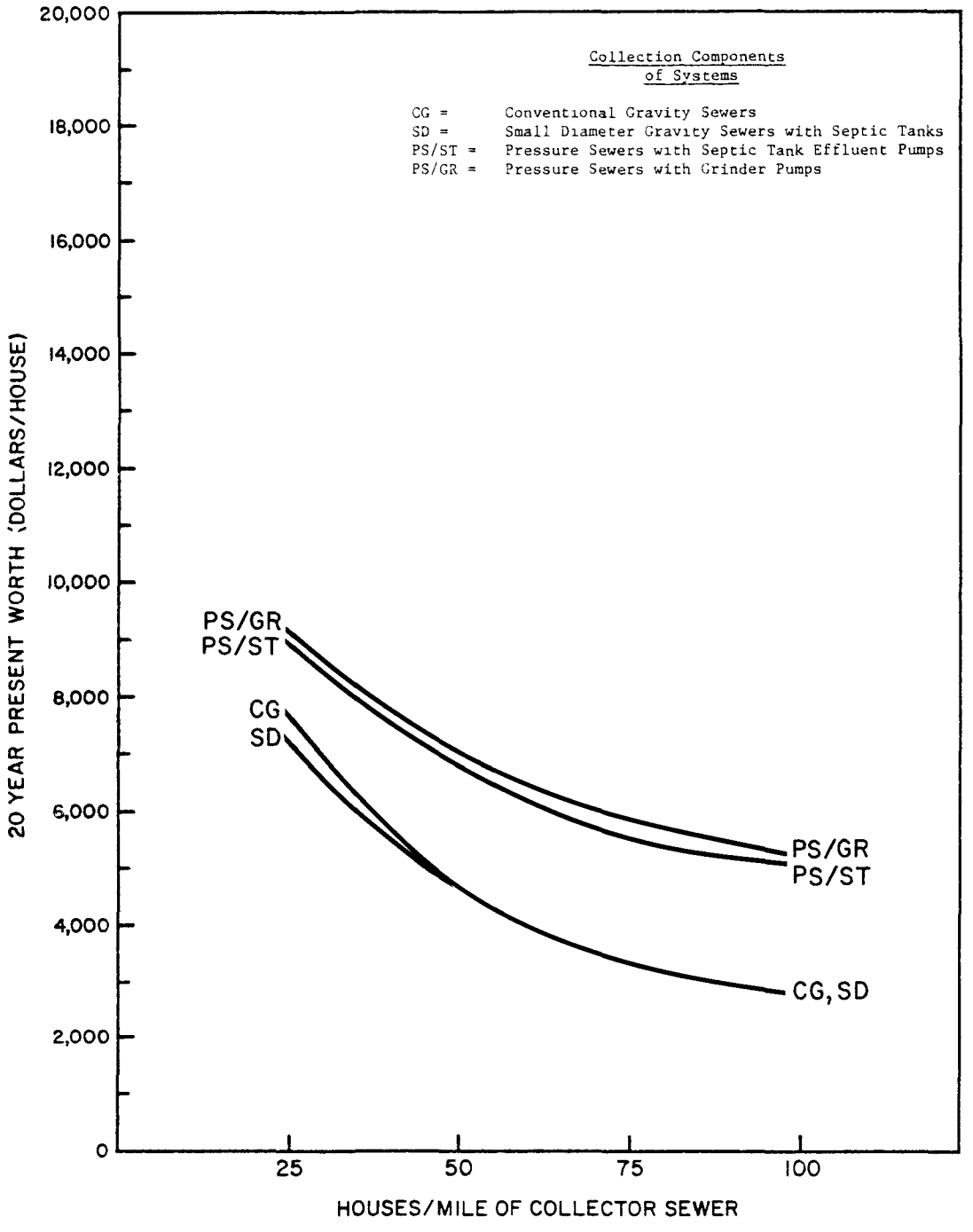

Figure IV-A-4-39. Cost comparison of collection technologies (no transmission/treatment) for Scenario 3; 0% growth.

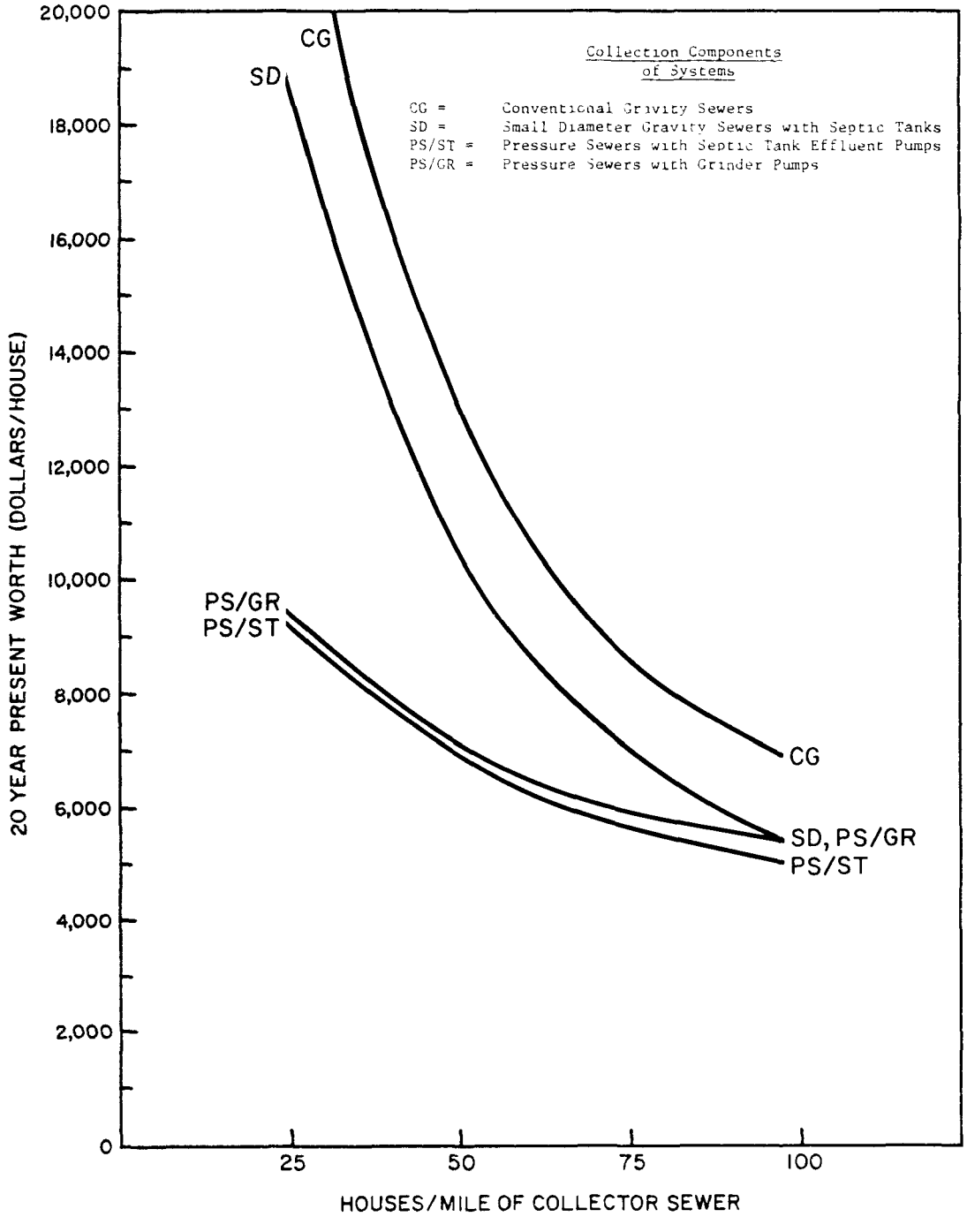

Figure IV-A-4-40. Cost comparison of collection technologies (no transmission/treatment) for Scenario 4; 0% growth.

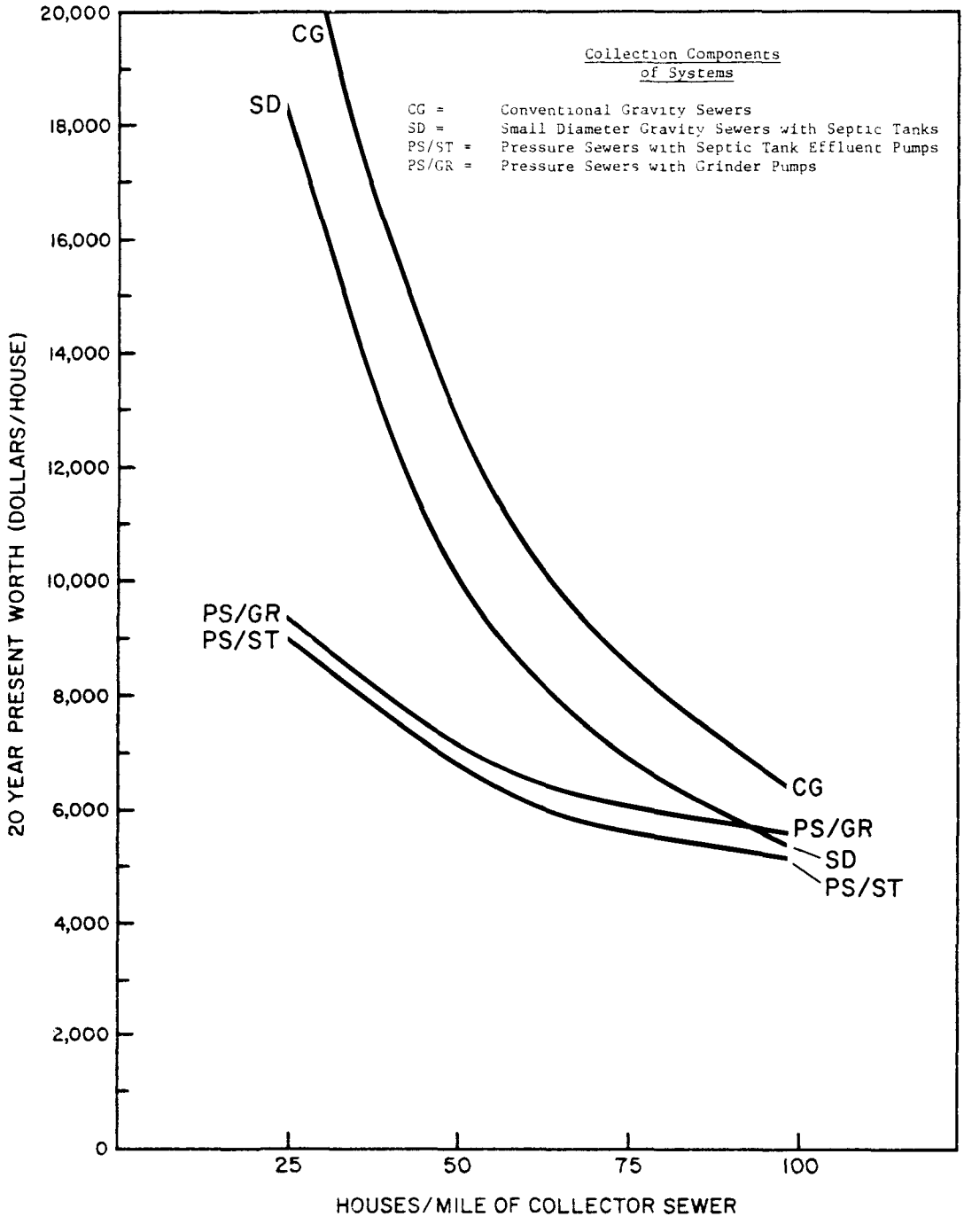

Figure IV-A-4-41. Cost comparison of collection technologies (no transmission/treatment) for Scenario 5; 0% growth.

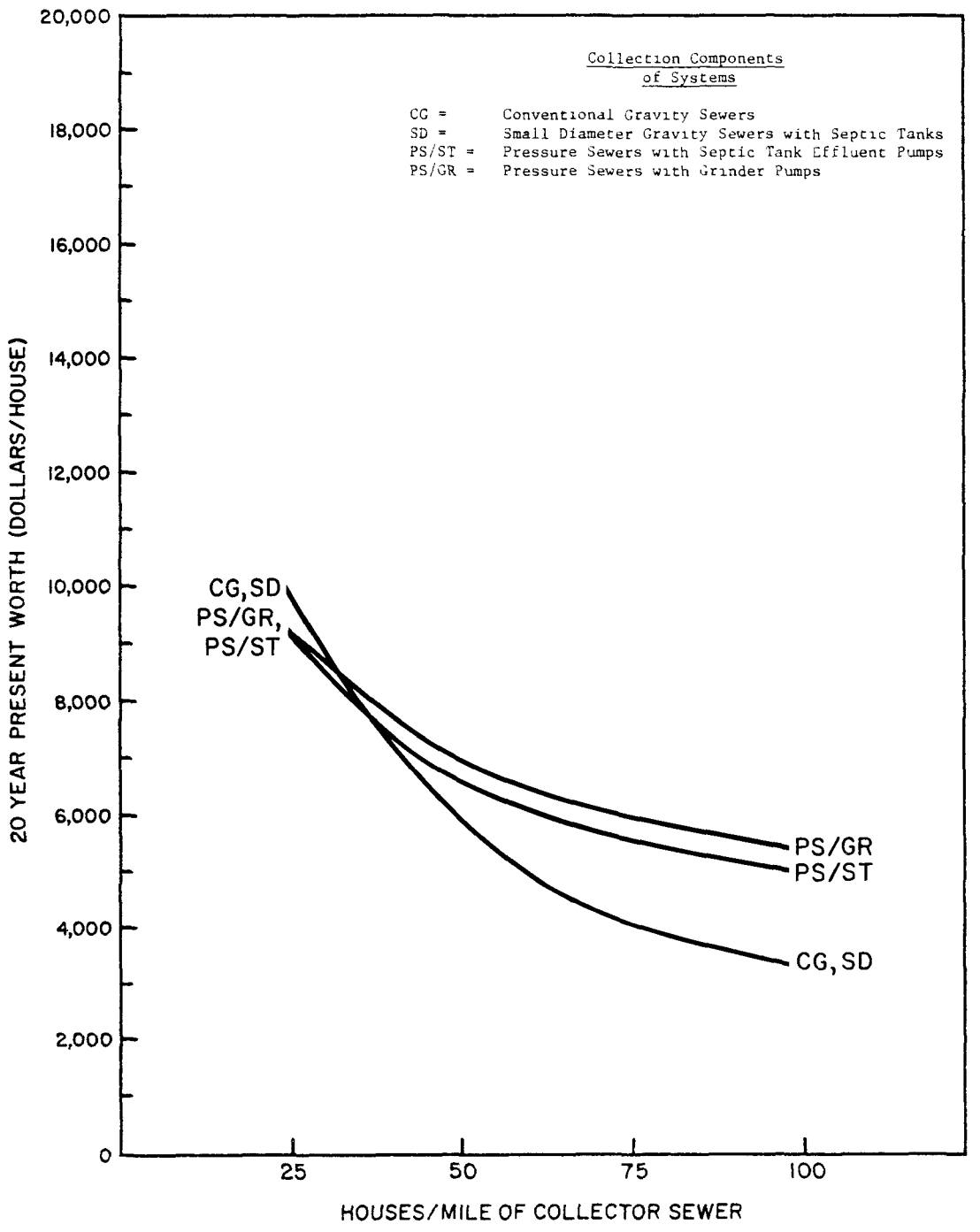

Figure IV-A-4-42. Cost comparison of collection technologies (no transmission/treatment) for Scenario 6; 0% growth.

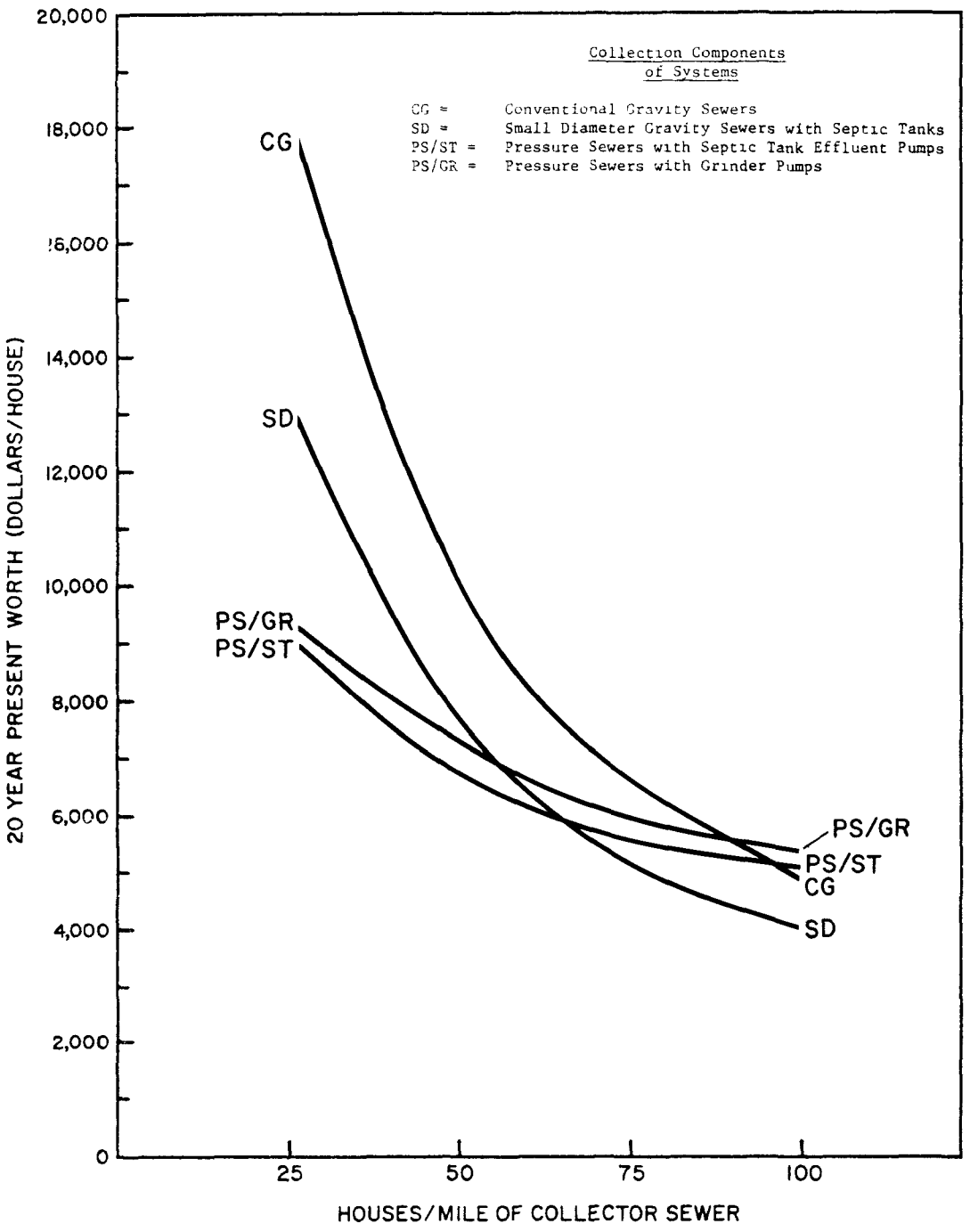

Figure IV-A-4-43. Cost comparison of collection technologies (no transmission/treatment) for Scenario 7; 0% growth.

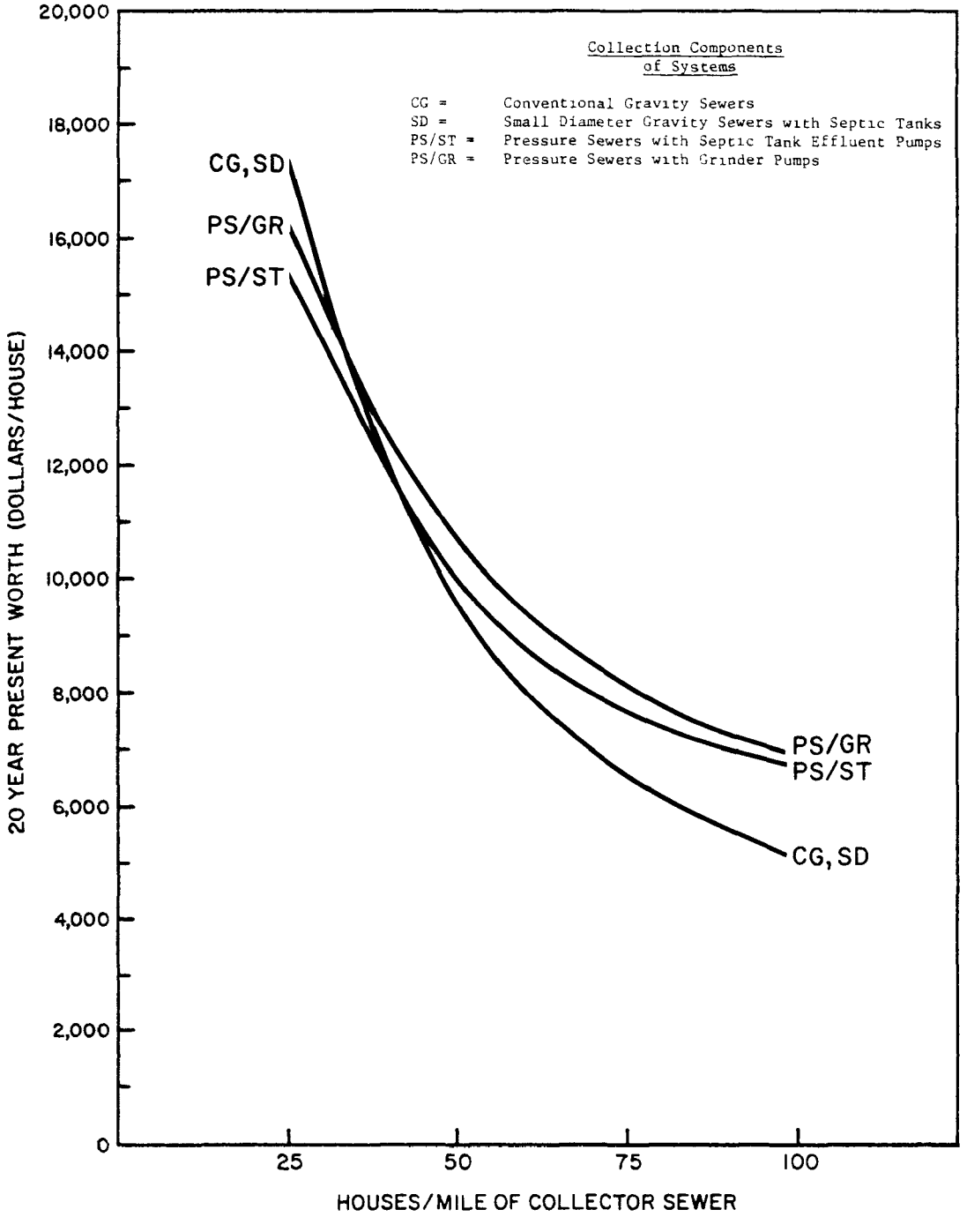

Figure IV-A-4-44. Cost comparison of collection technologies (no transmission/treatment) for Scenario 8; 0% growth.

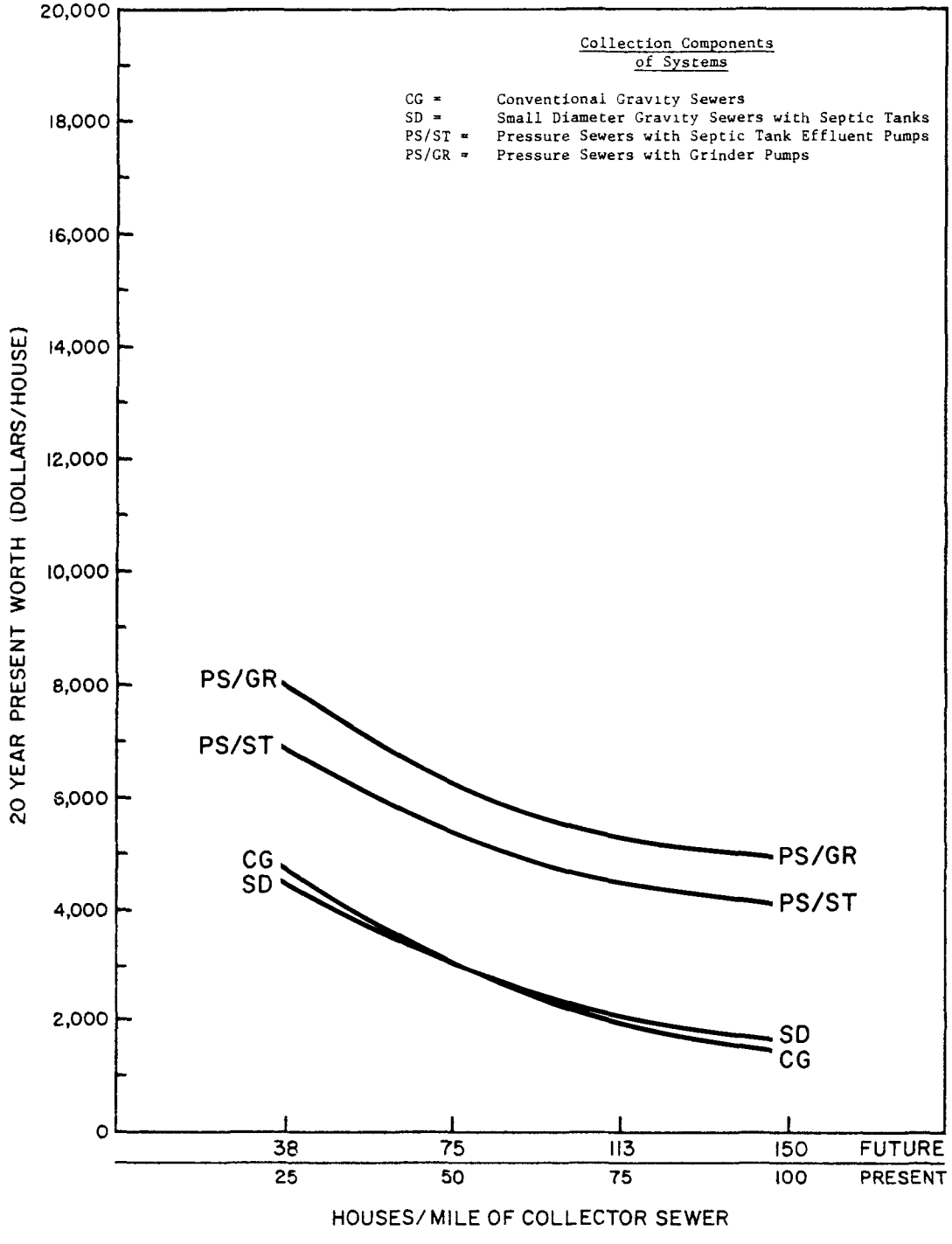

Figure IV-A-4-45. Cost comparison of collection technologies (no transmission/treatment) for Scenario 1; 50% growth.

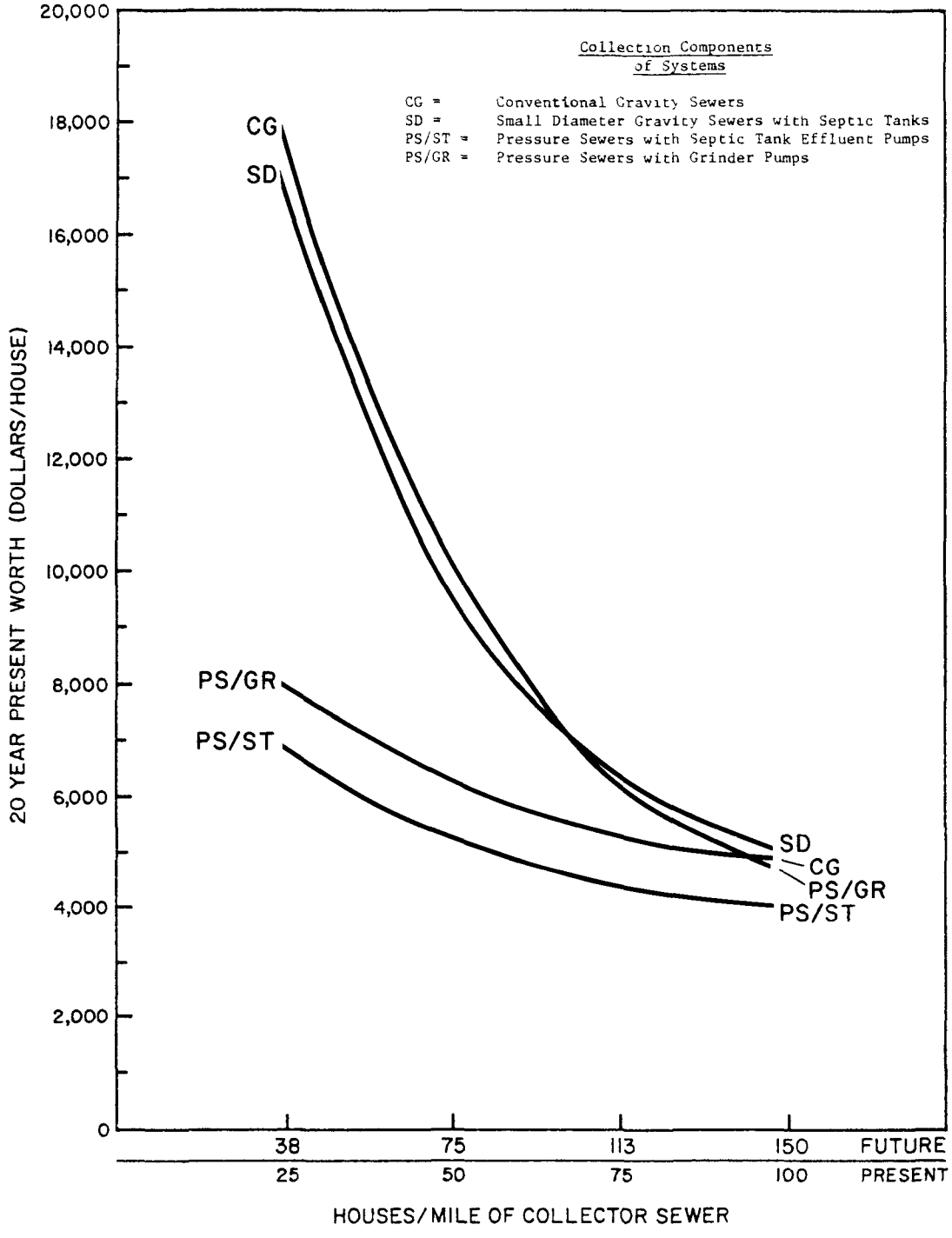

Figure IV-A-4-46. Cost comparison of collection technologies (no transmission/treatment) for Scenario 2; 50% growth.

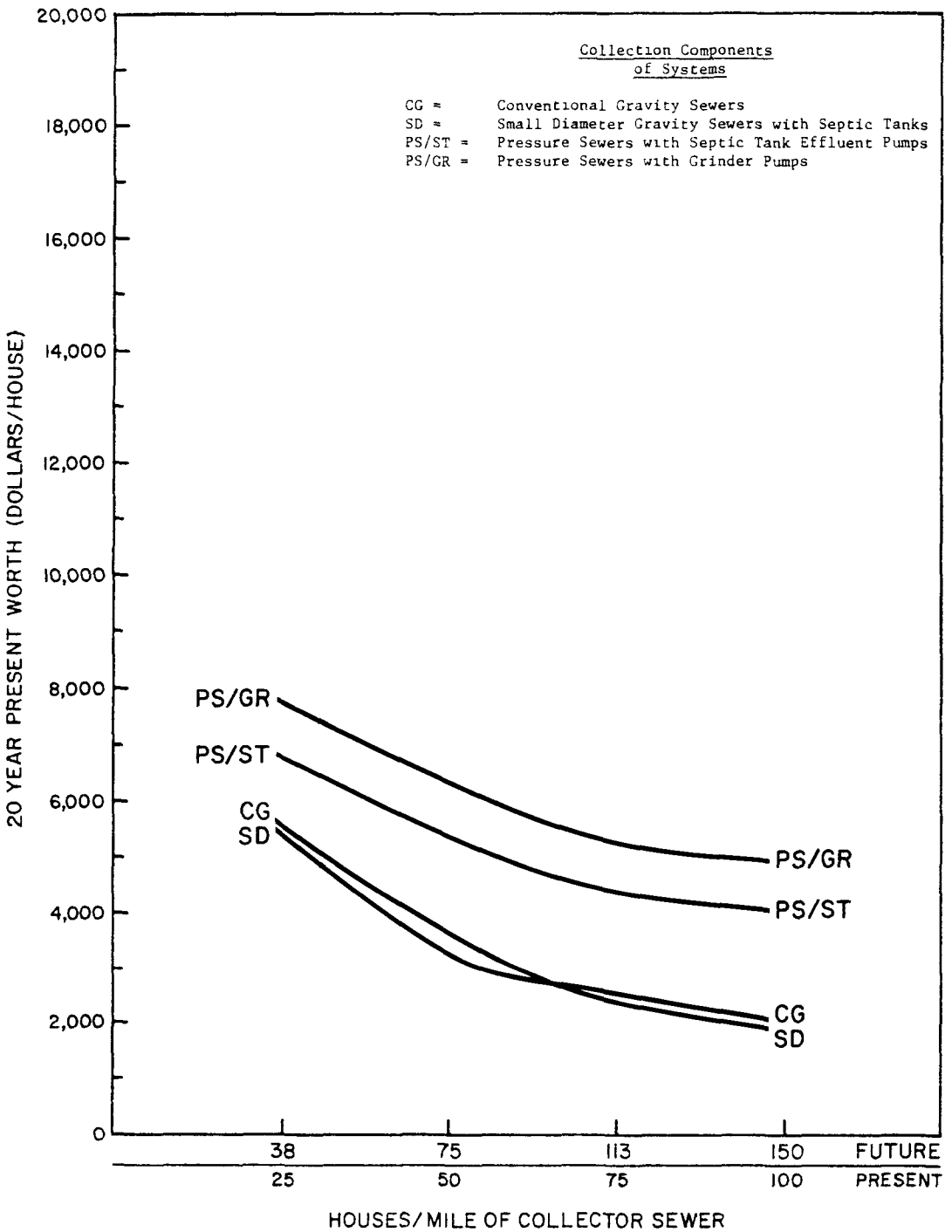

Figure IV-A-4-47. Cost comparison of collection technologies (no transmission/treatment) for Scenario 3; 50% growth.

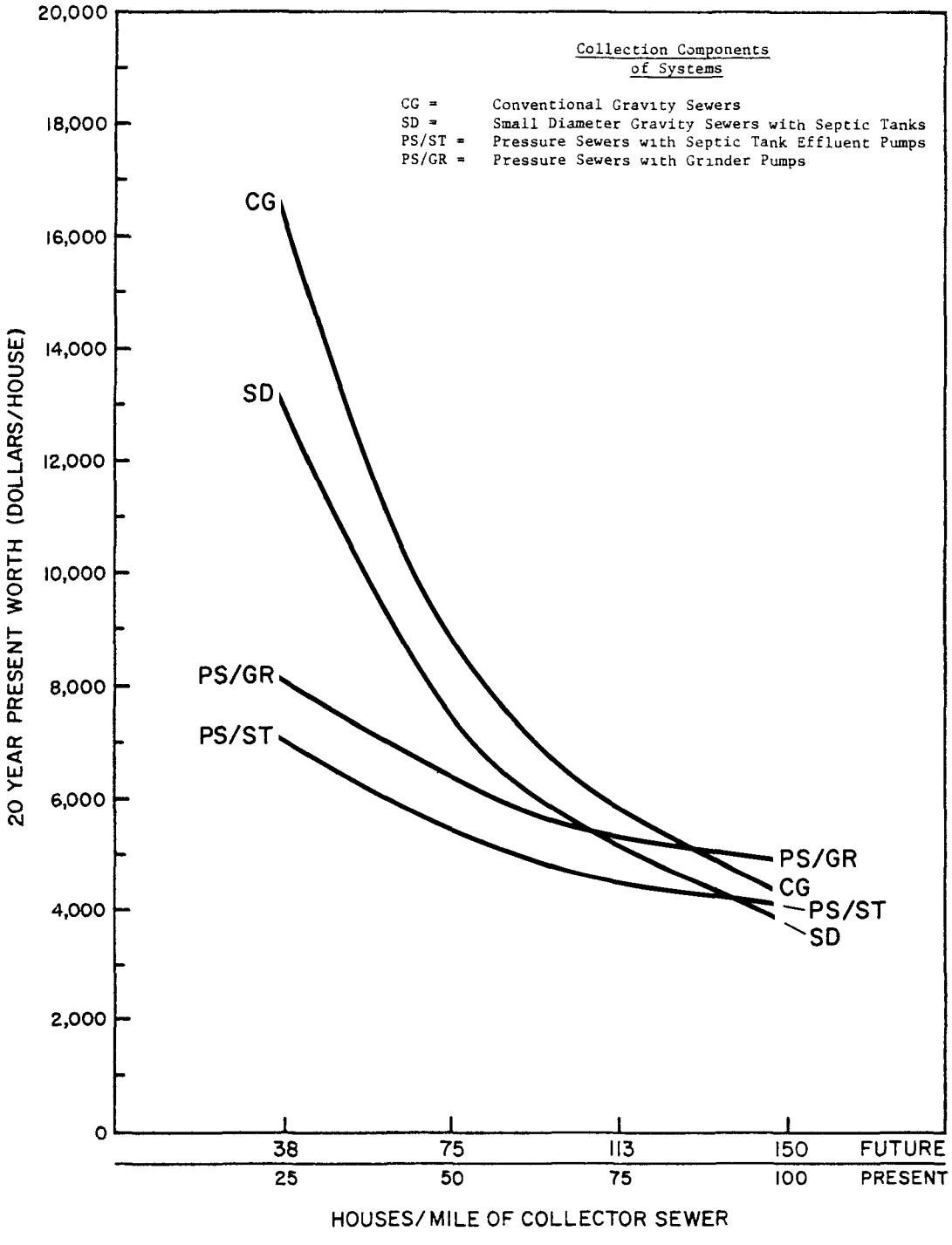

Figure IV-A-4-48. Cost comparison of collection technologies (no transmission/treatment) for Scenario 4; 50% growth.

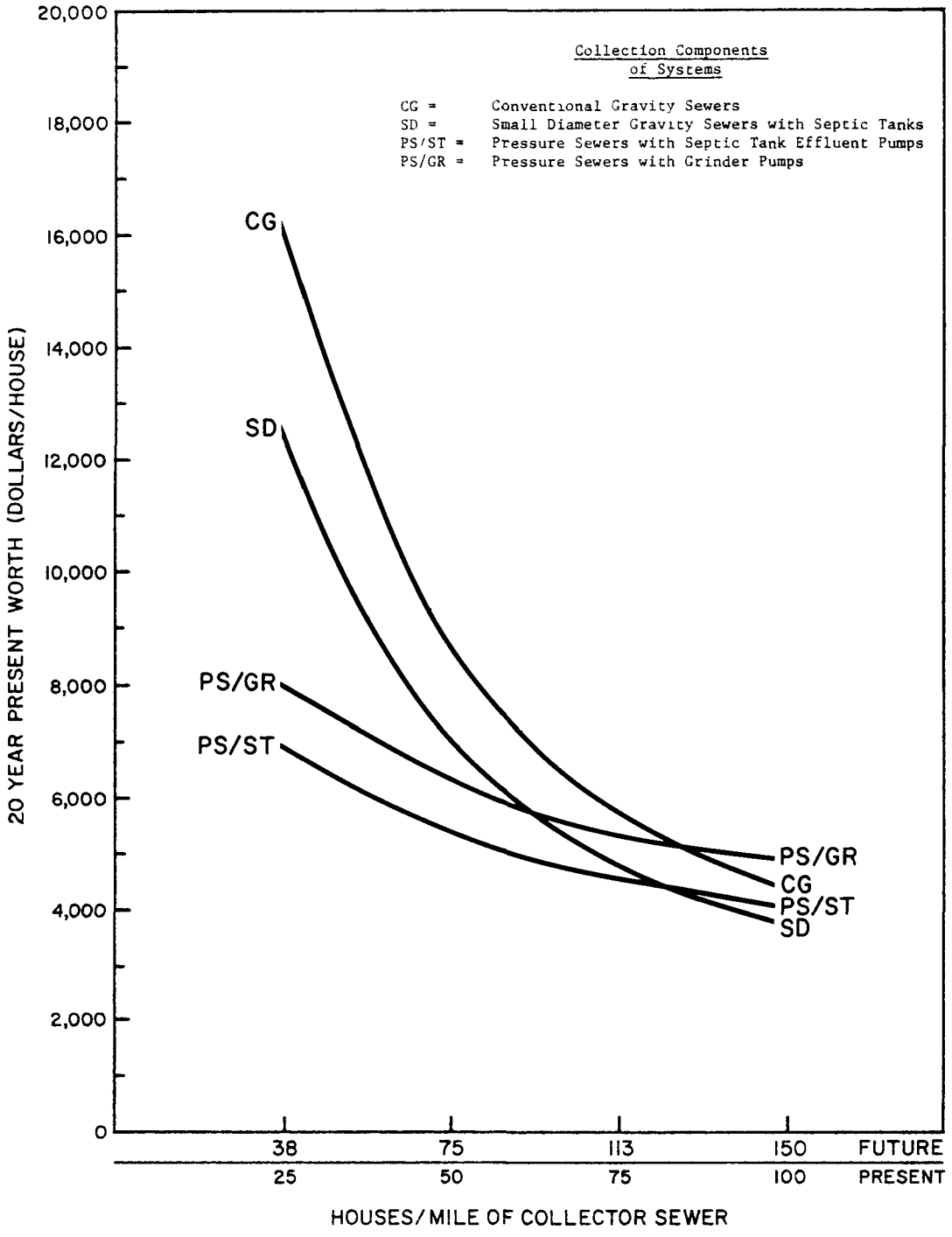

Figure IV-A-4-49. Cost comparison of collection technologies (no transmission/treatment) for Scenario 5; 50% growth.

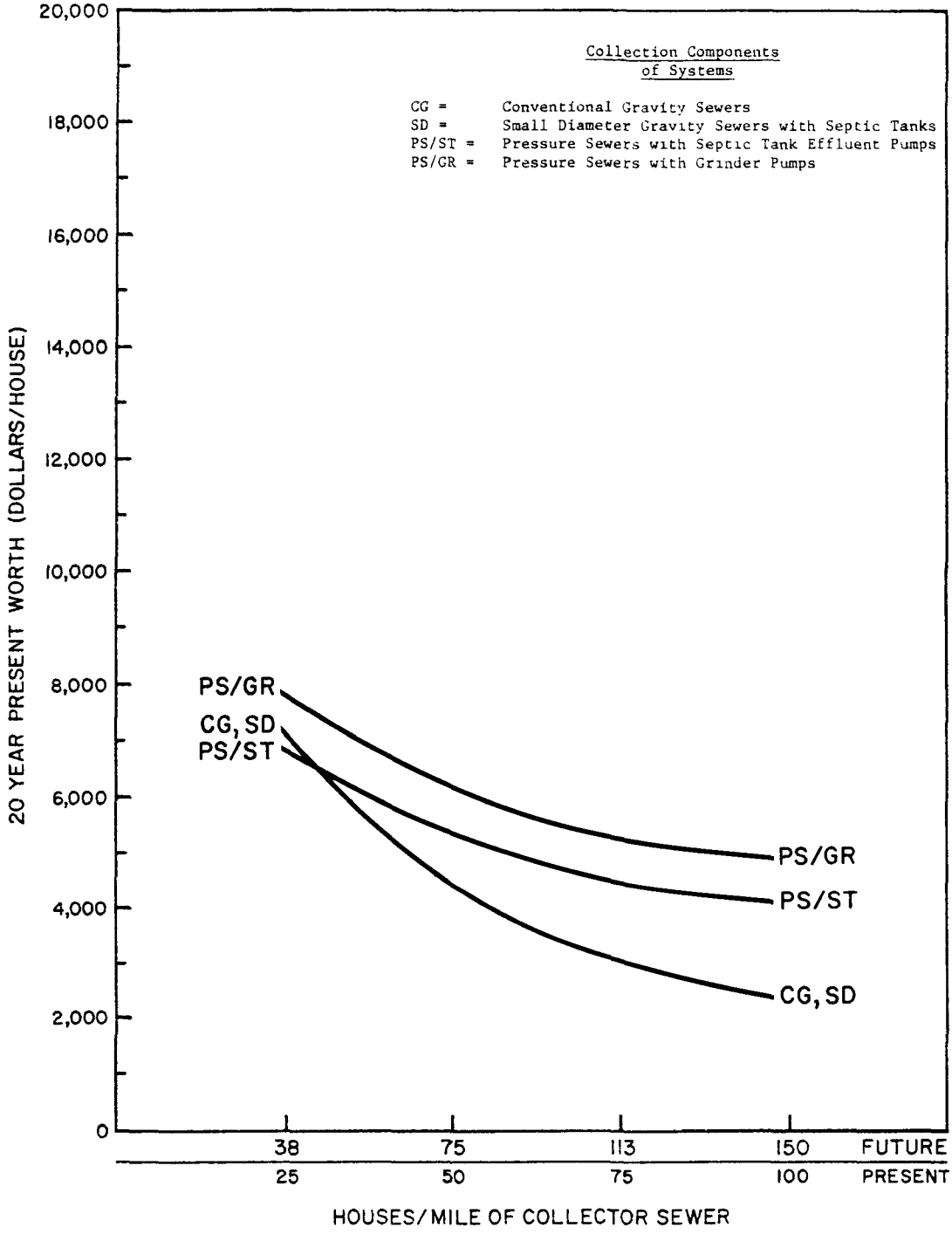

Figure IV-A-4-50. Cost comparison of collection technologies (no transmission/treatment) for Scenario 6; 50% growth.

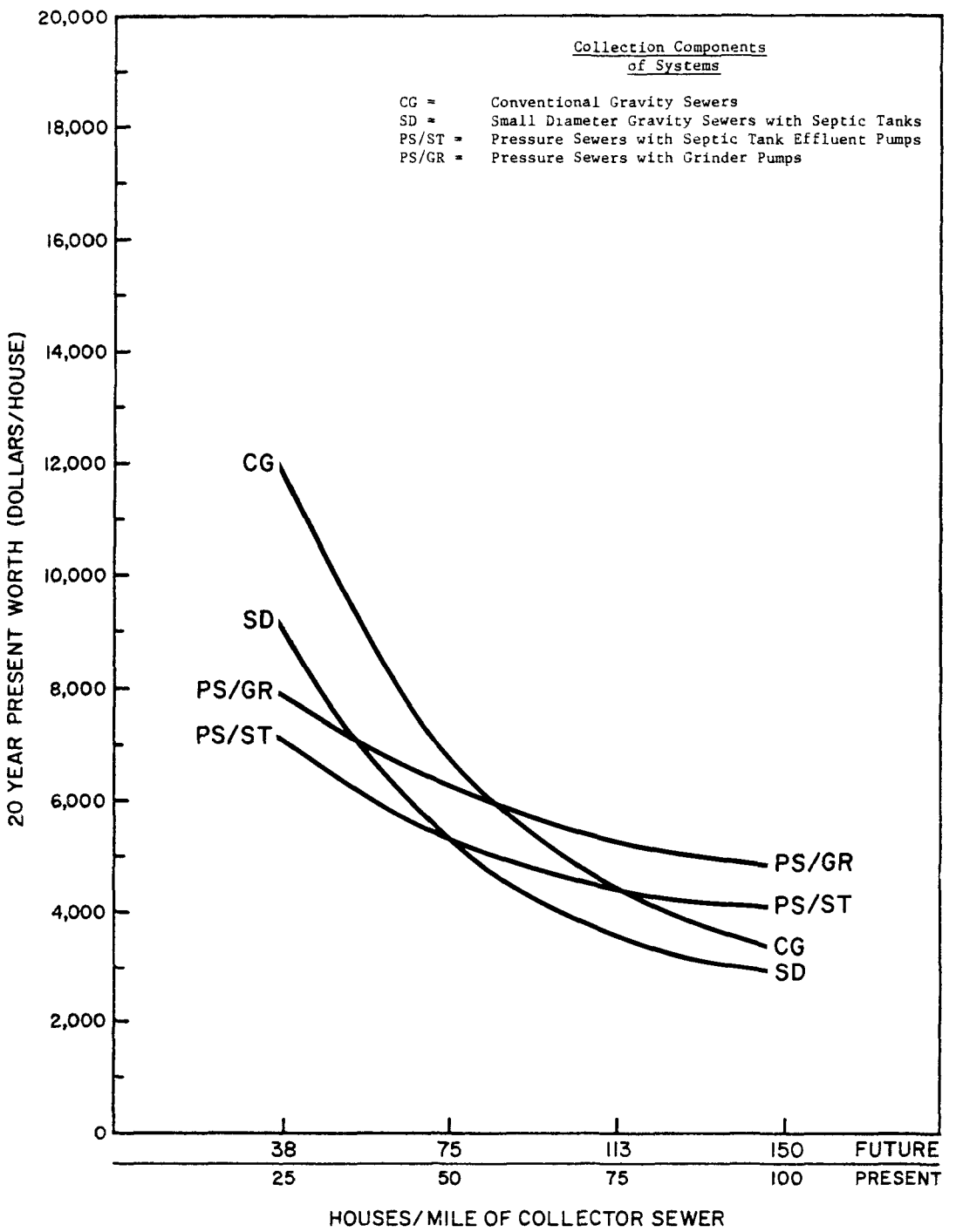

Figure IV-A-4-51. Cost comparison of collection technologies (no transmission/treatment) for Scenario 7; 50% growth.

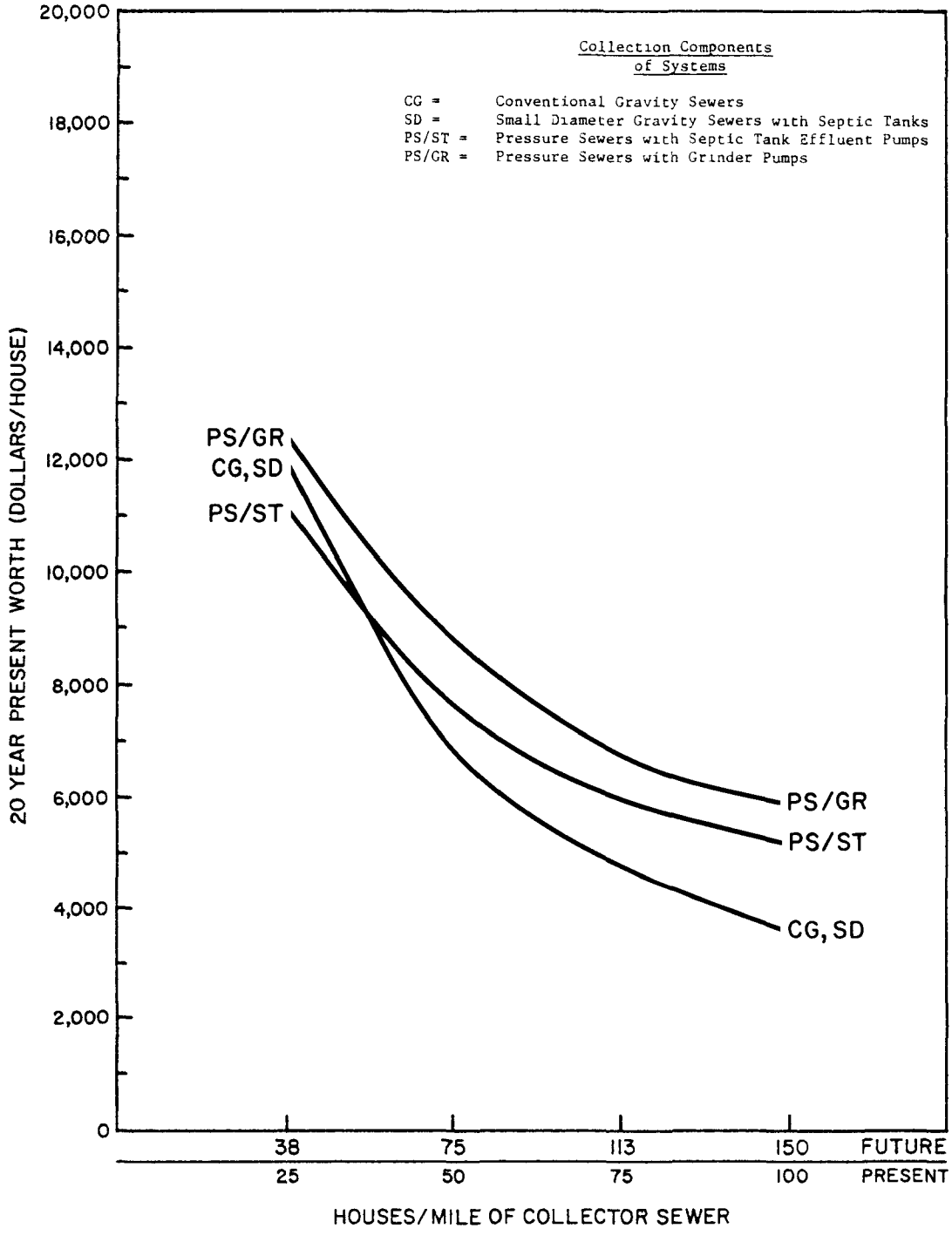

Figure IV-A-4-52. Cost comparison of collection technologies (no transmission/treatment) for Scenario 8; 50% growth.

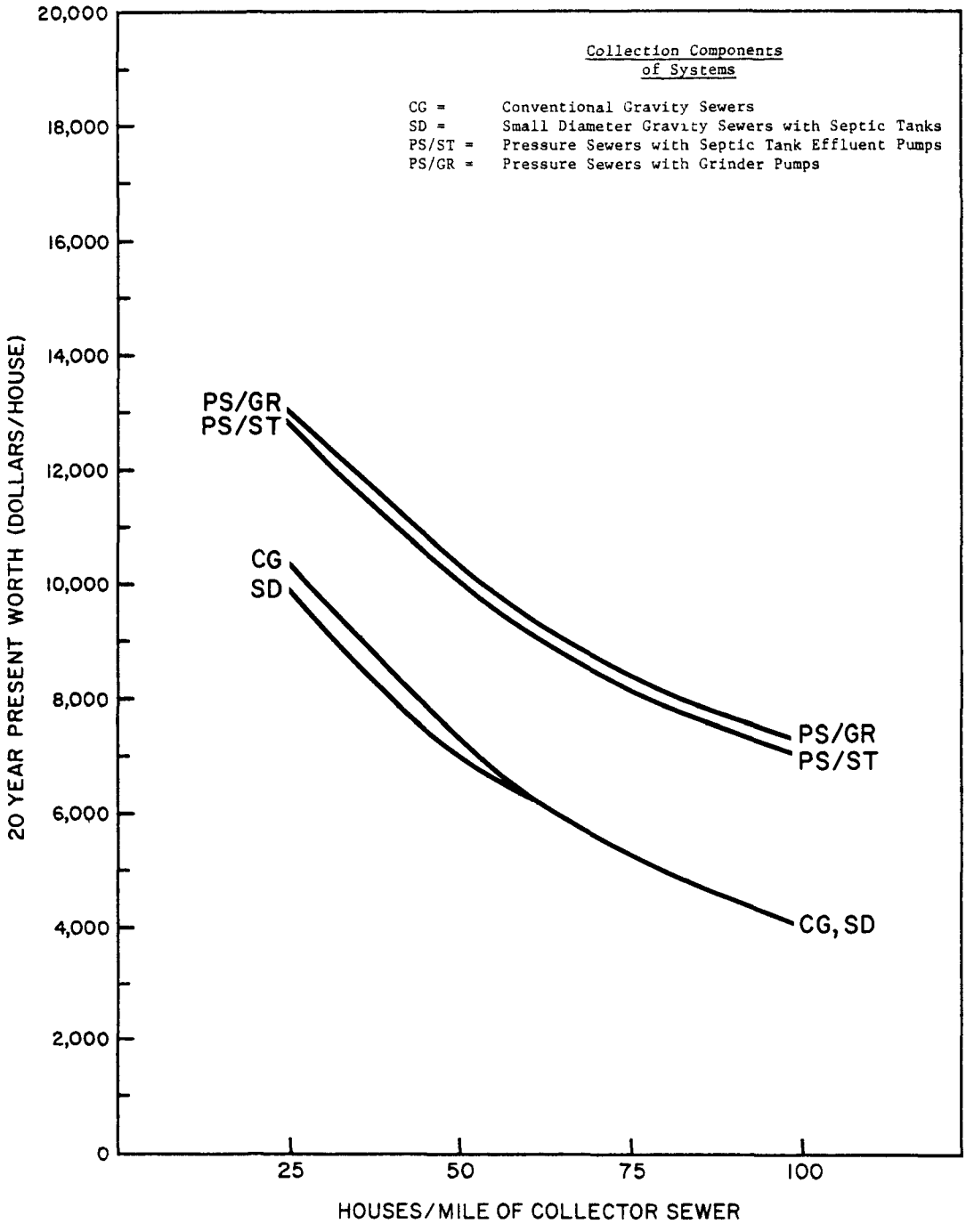

Figure IV-A-4-53. Cost comparison of centralized collection, transmission, and treatment systems for Scenario 1; 0% growth.

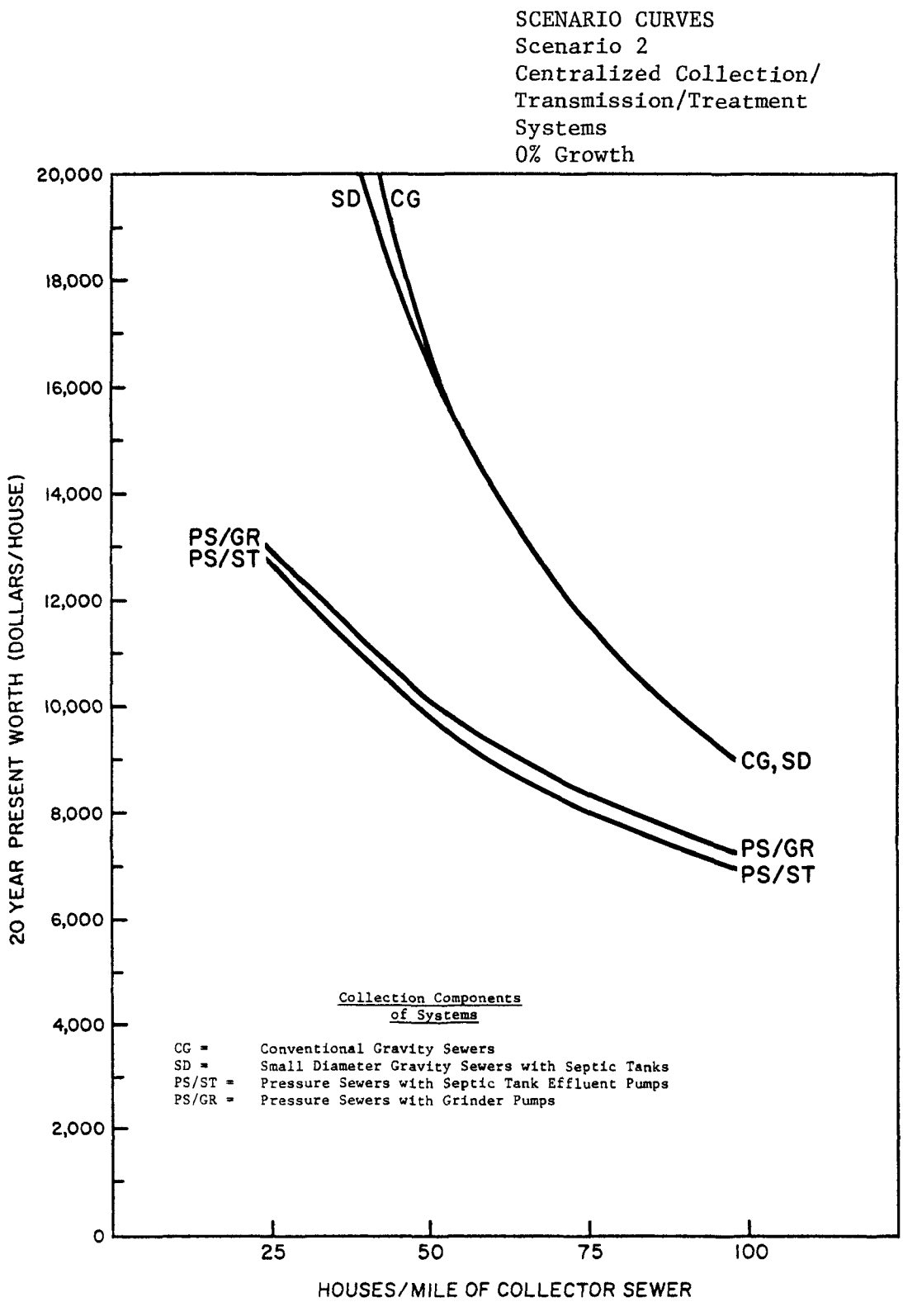

Figure IV-A-4-54. Cost comparison of centralized collection, transmission, and treatment systems for Scenario 2; 0% growth.

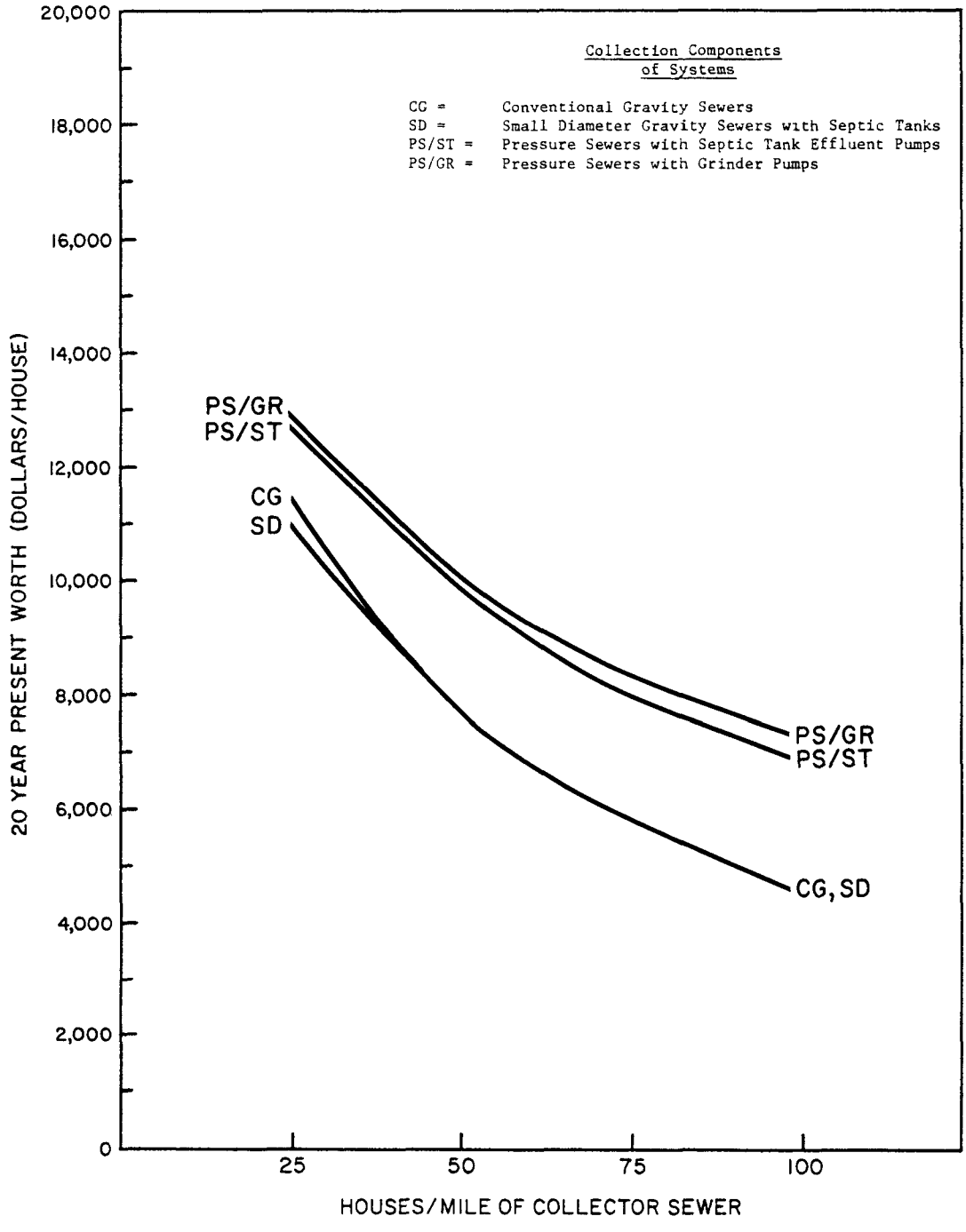

Figure IV-A-4-55. Cost comparison of centralized collection, transmission, and treatment systems for Scenario 3; 0% growth.

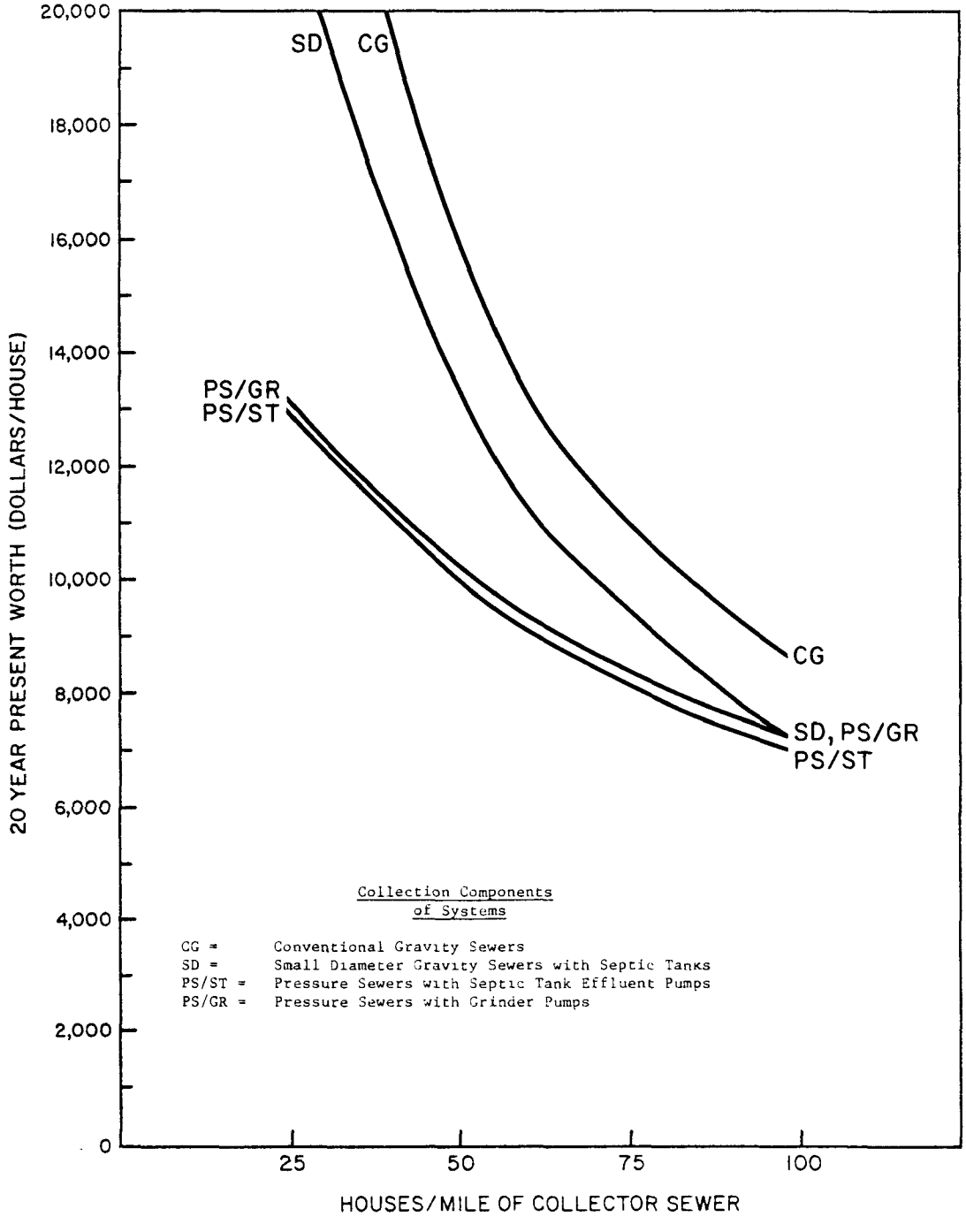

Figure IV-A-4-56. Cost comparison of centralized collection, transmission, and treatment systems for Scenario 4; 0% growth.

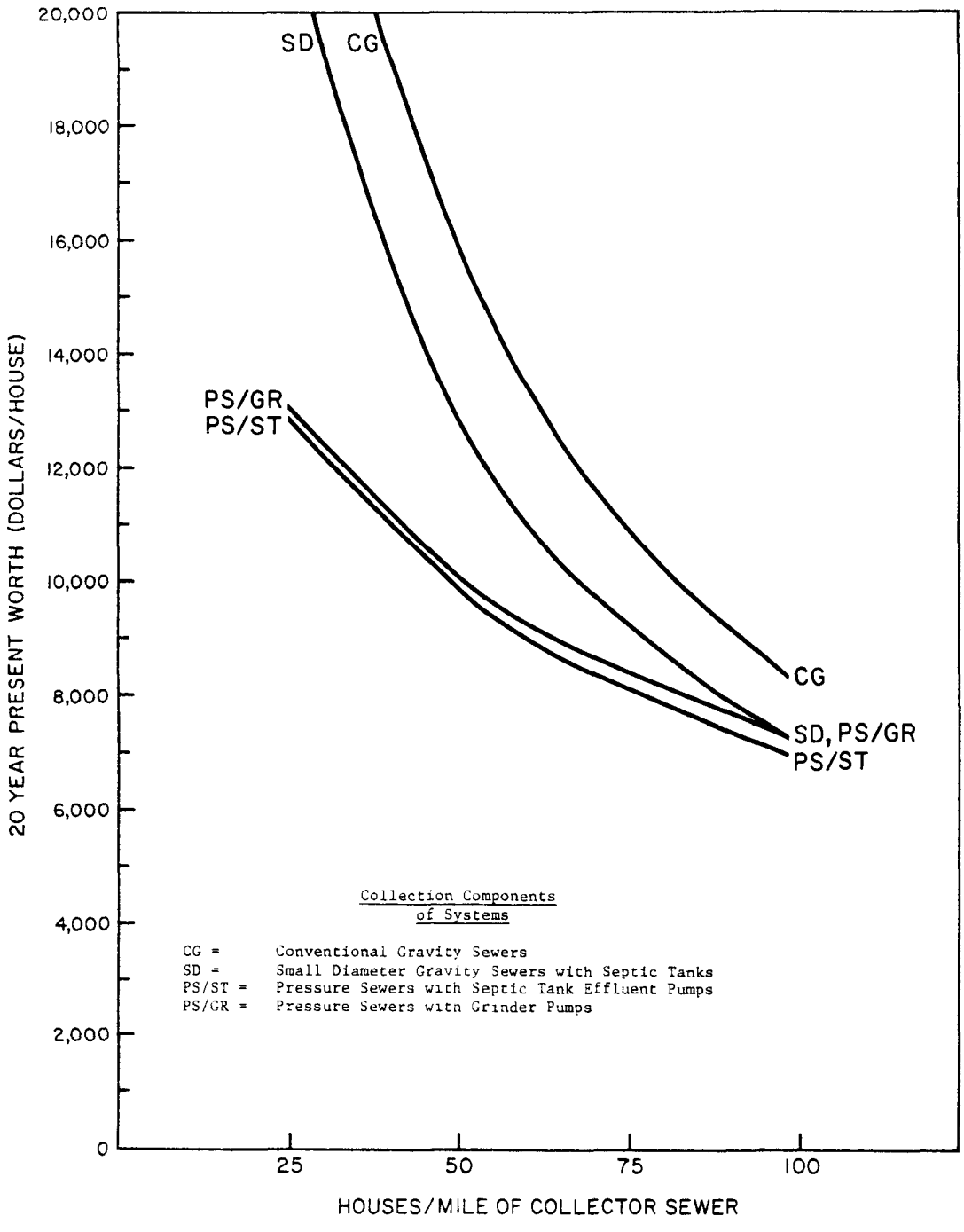

Figure IV-A-4-57. Cost comparison of centralized collection, transmission, and treatment systems for Scenario 5; 0% Growth.

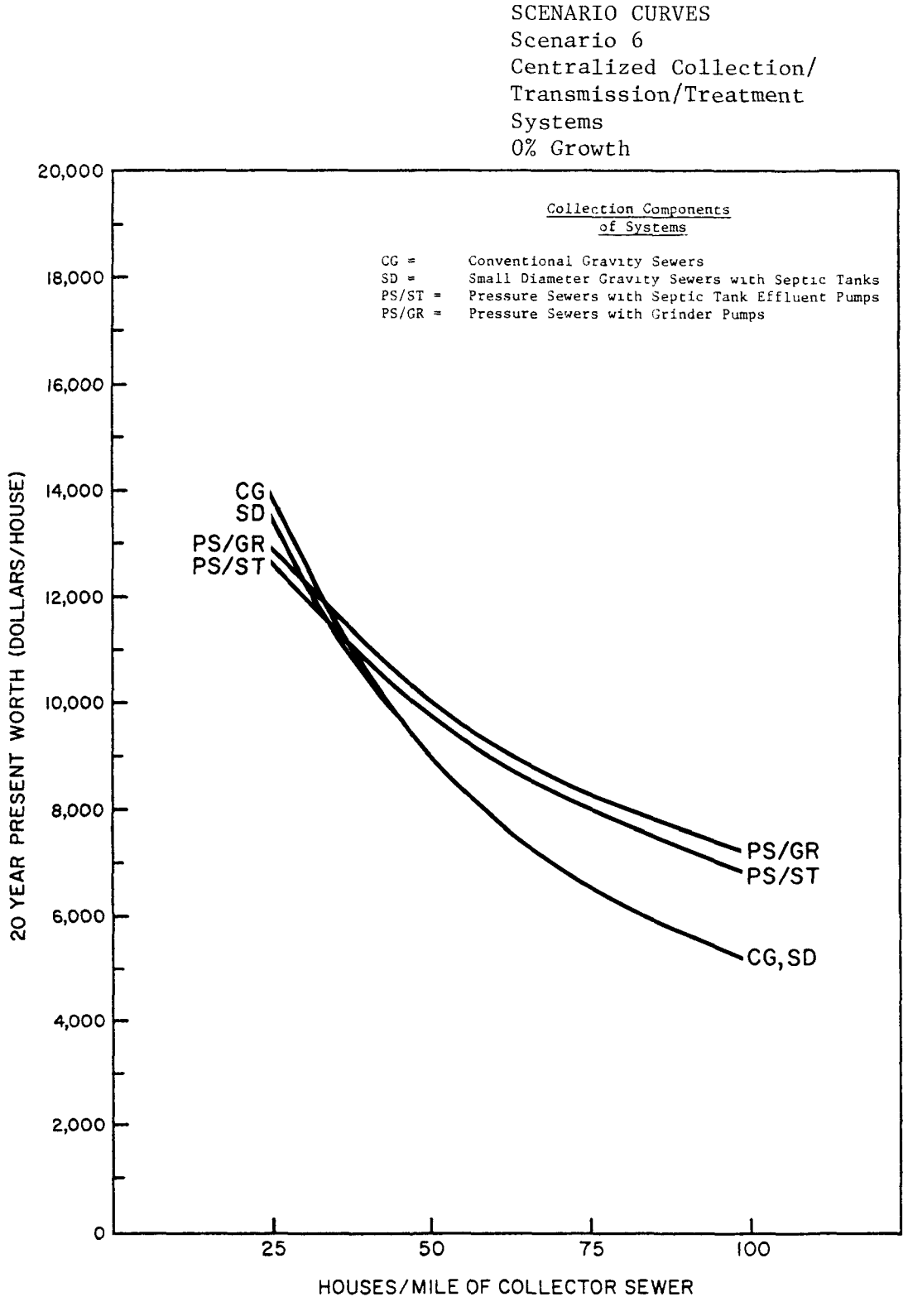

Figure IV-A-4-58. Cost comparison of centralized collection, transmission, and treatment systems for Scenario 6; 0% growth.

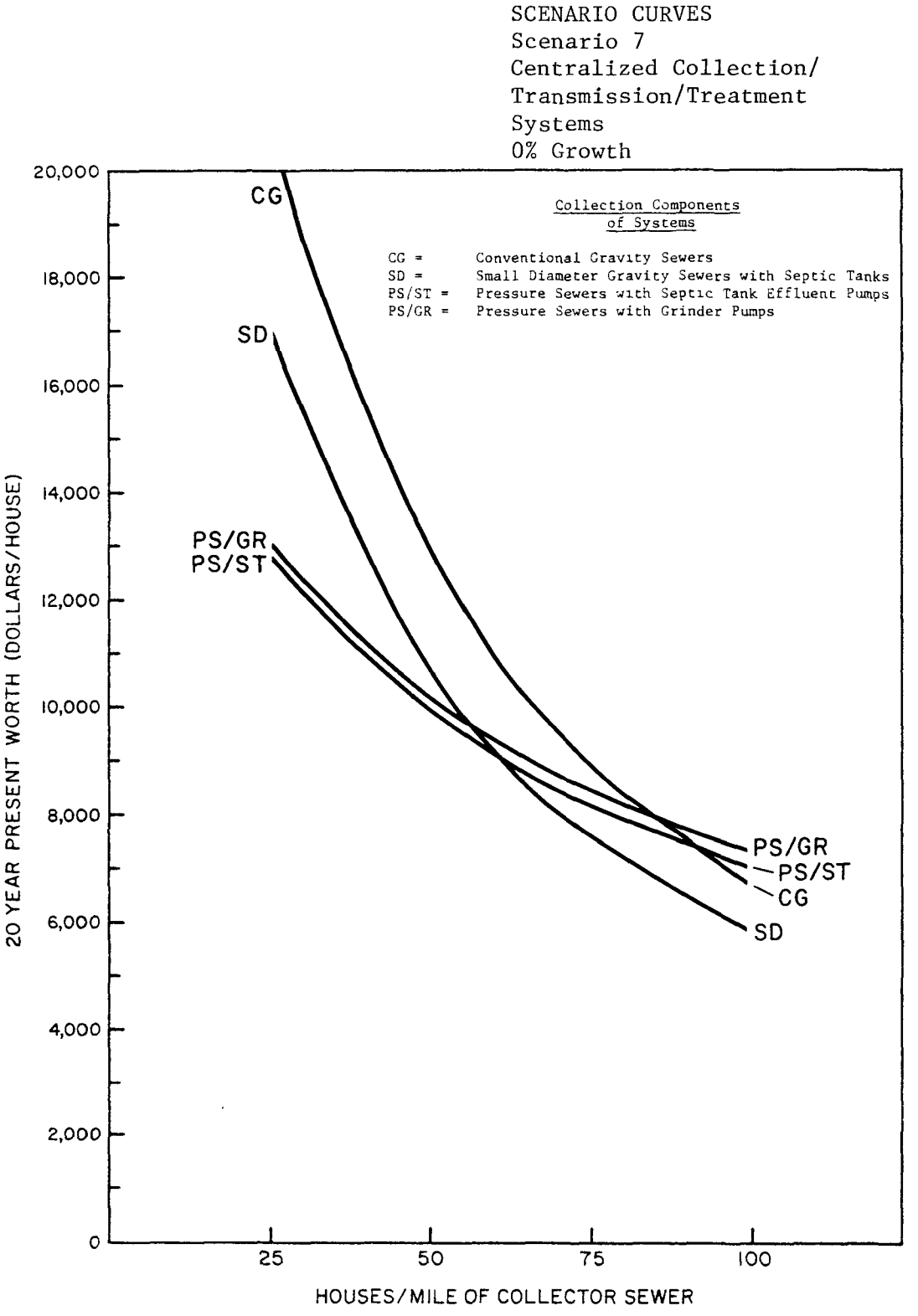

Figure IV-A-4-59. Cost comparison of centralized collection, transmission, and treatment systems for Scenario 7; 0% growth.

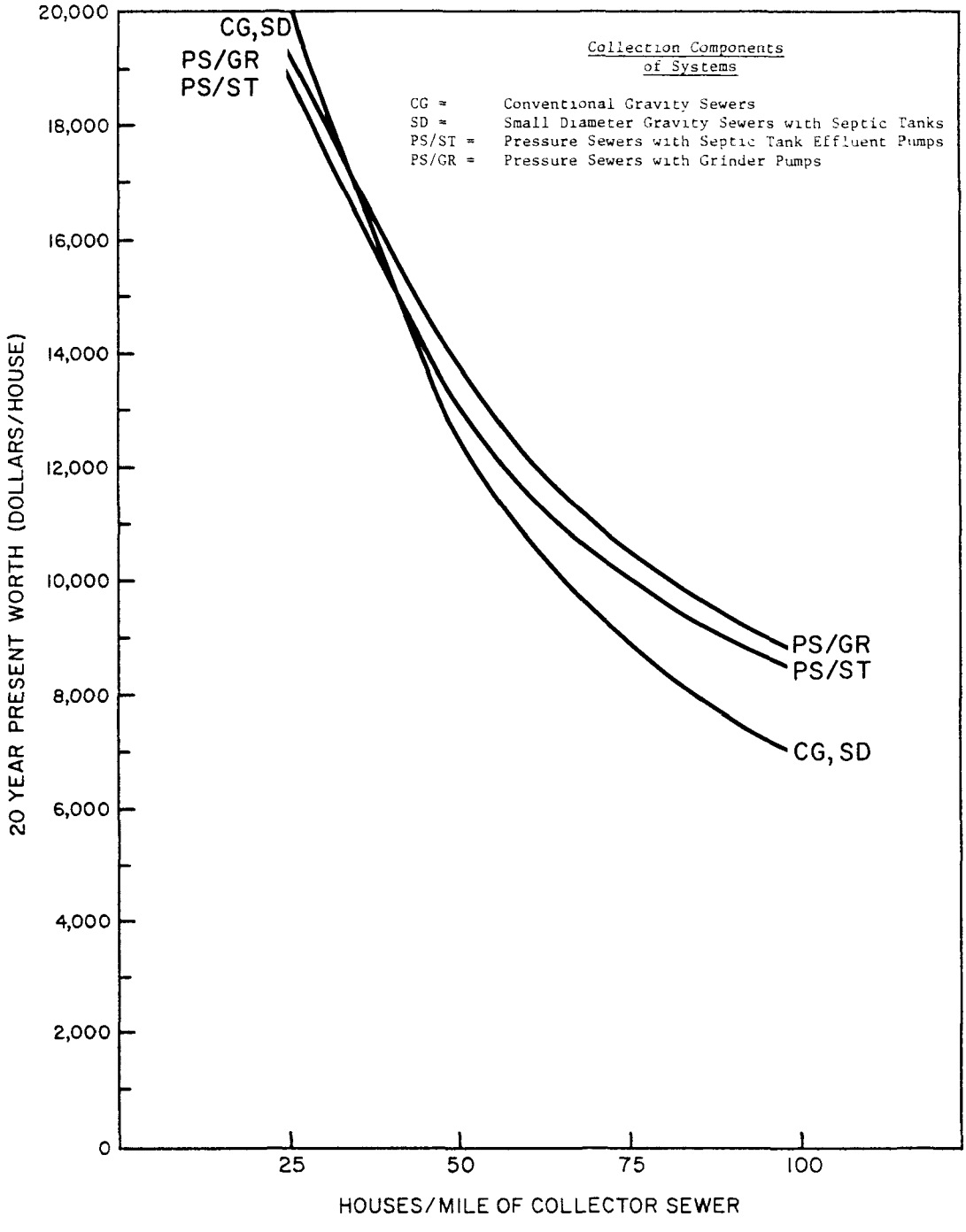

Figure IV-A-4-60. Cost comparison of centralized collection, transmission, and treatment systems for Scenario 8; 0% growth.

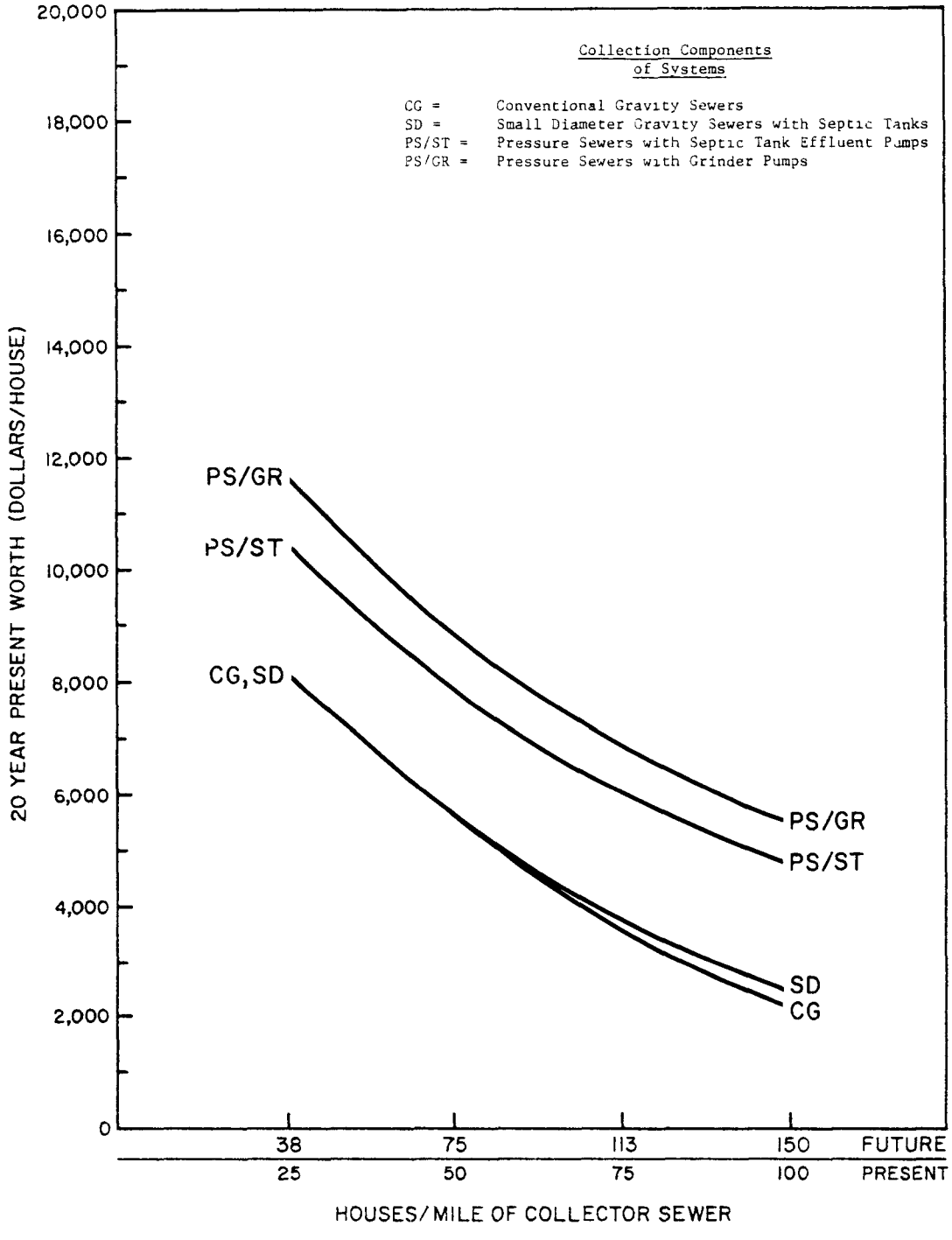

Figure IV-A-4-61. Cost comparison of centralized collection, transmission, and treatment systems for Scenario 1; 50% growth.

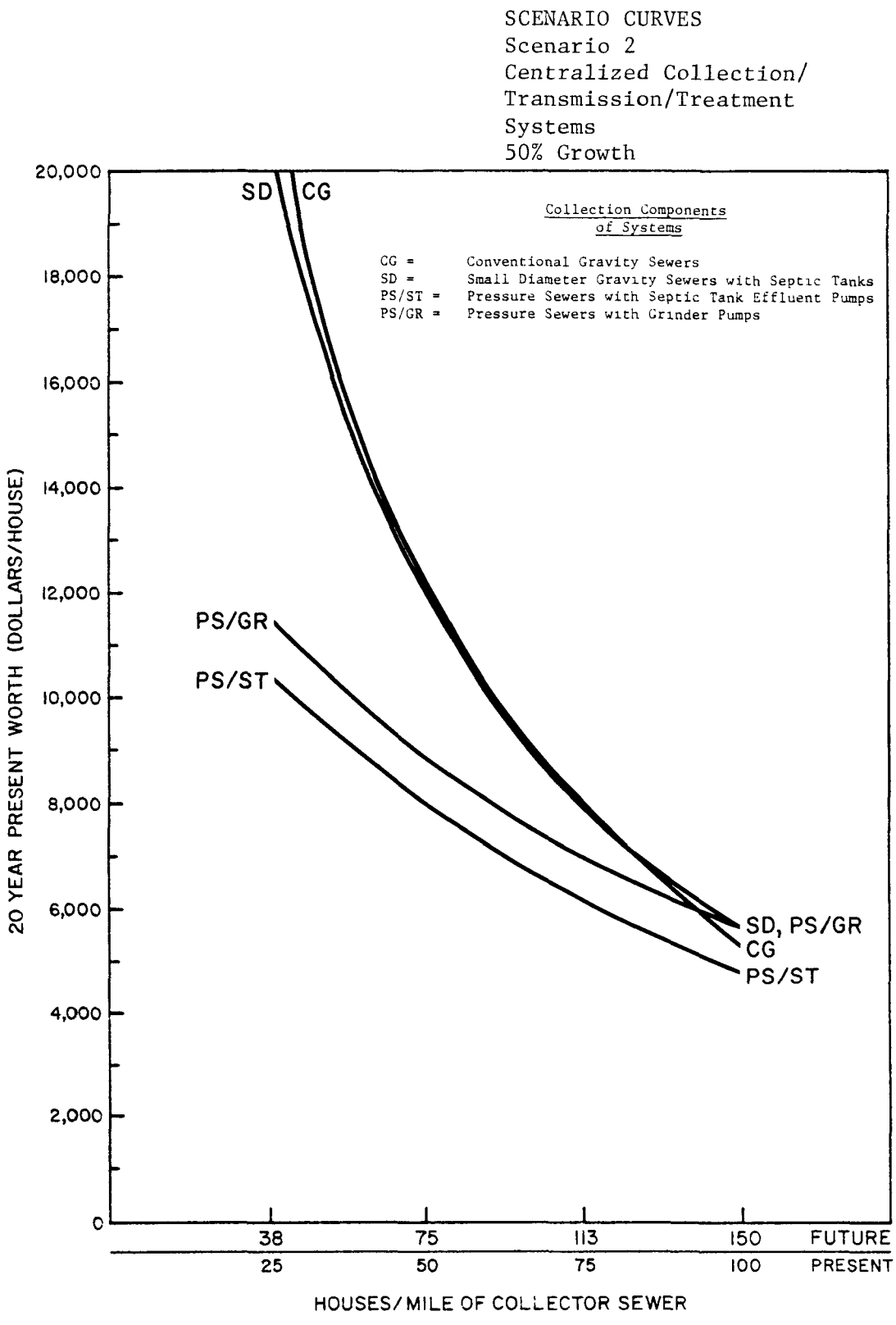

Figure IV-A-4-62. Cost comparison of centralized collection, transmission, and treatment systems for Scenario 2; 50% growth.

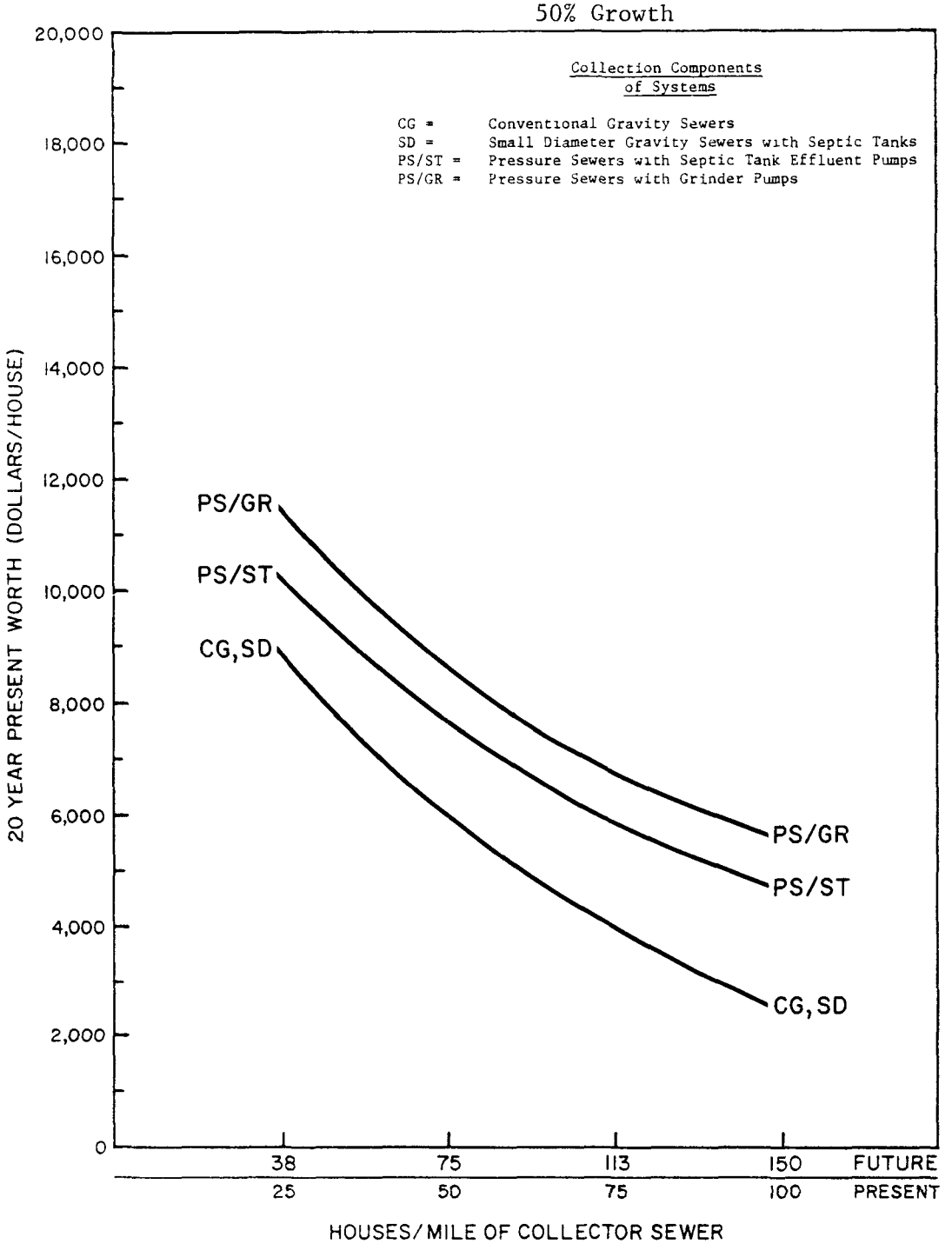

Figure IV-A-4-63. Cost comparison of centralized collection, transmission, and treatment systems for Scenario 3; 50% growth.

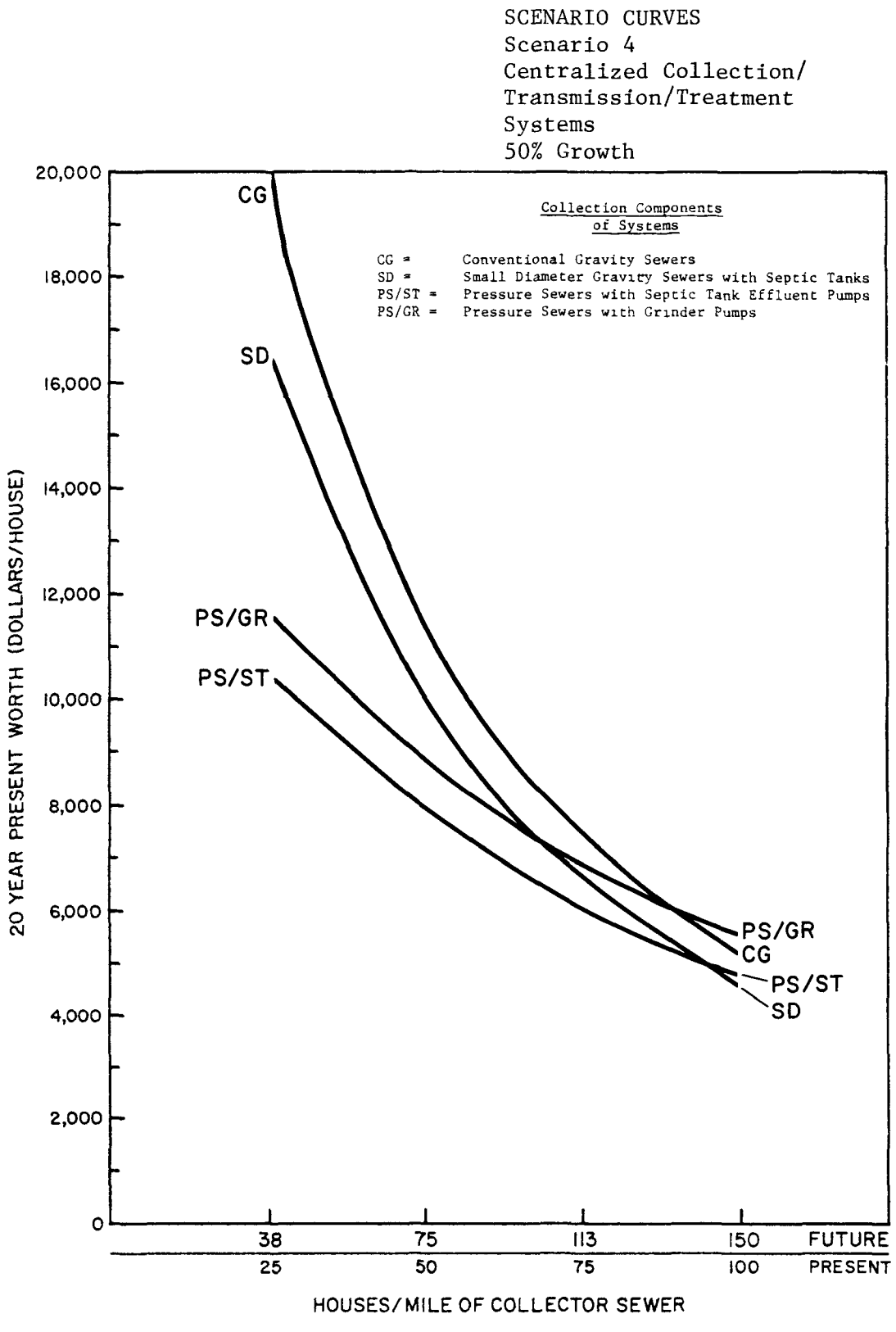

Figure IV-A-4-64. Cost comparison of centralized collection, transmission, and treatment systems for Scenario 4; 50% growth.

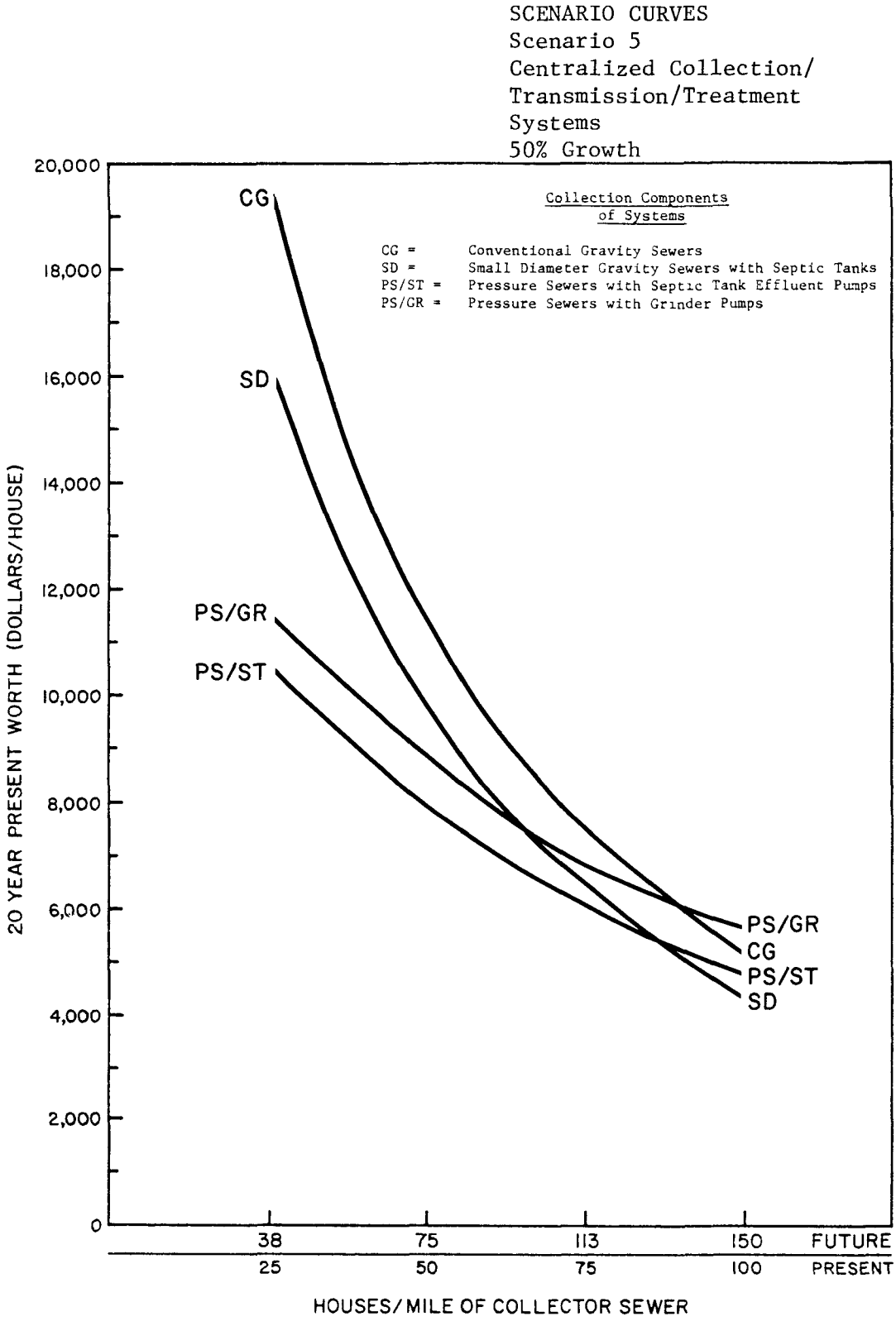

Figure IV-A-4-65. Cost comparison of centralized collection, transmission, and treatment systems for Scenario 5; 50% growth.

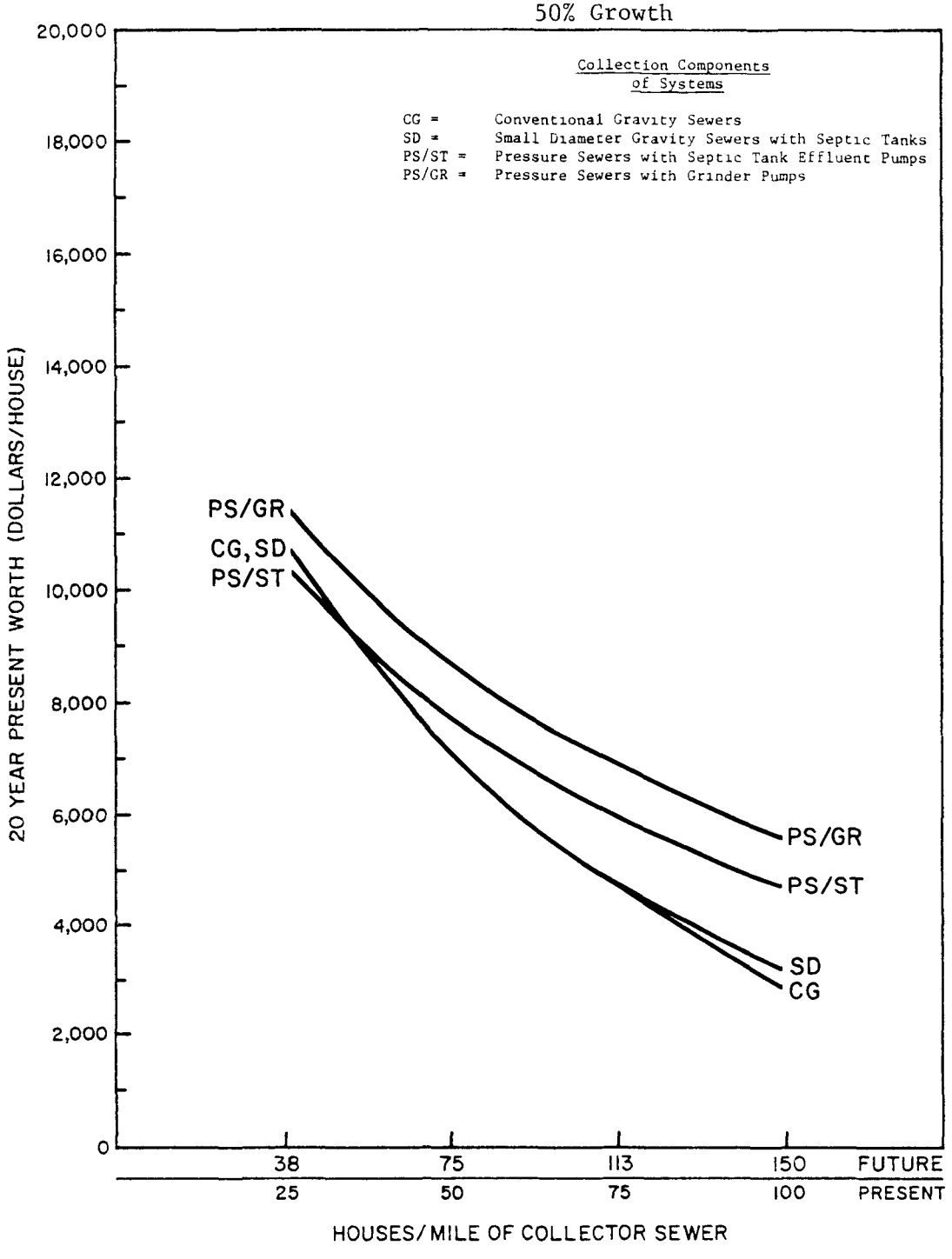

Figure IV-A-4-66. Cost comparison of centralized collection, transmission, and treatment systems for Scenario 6; 50% growth.

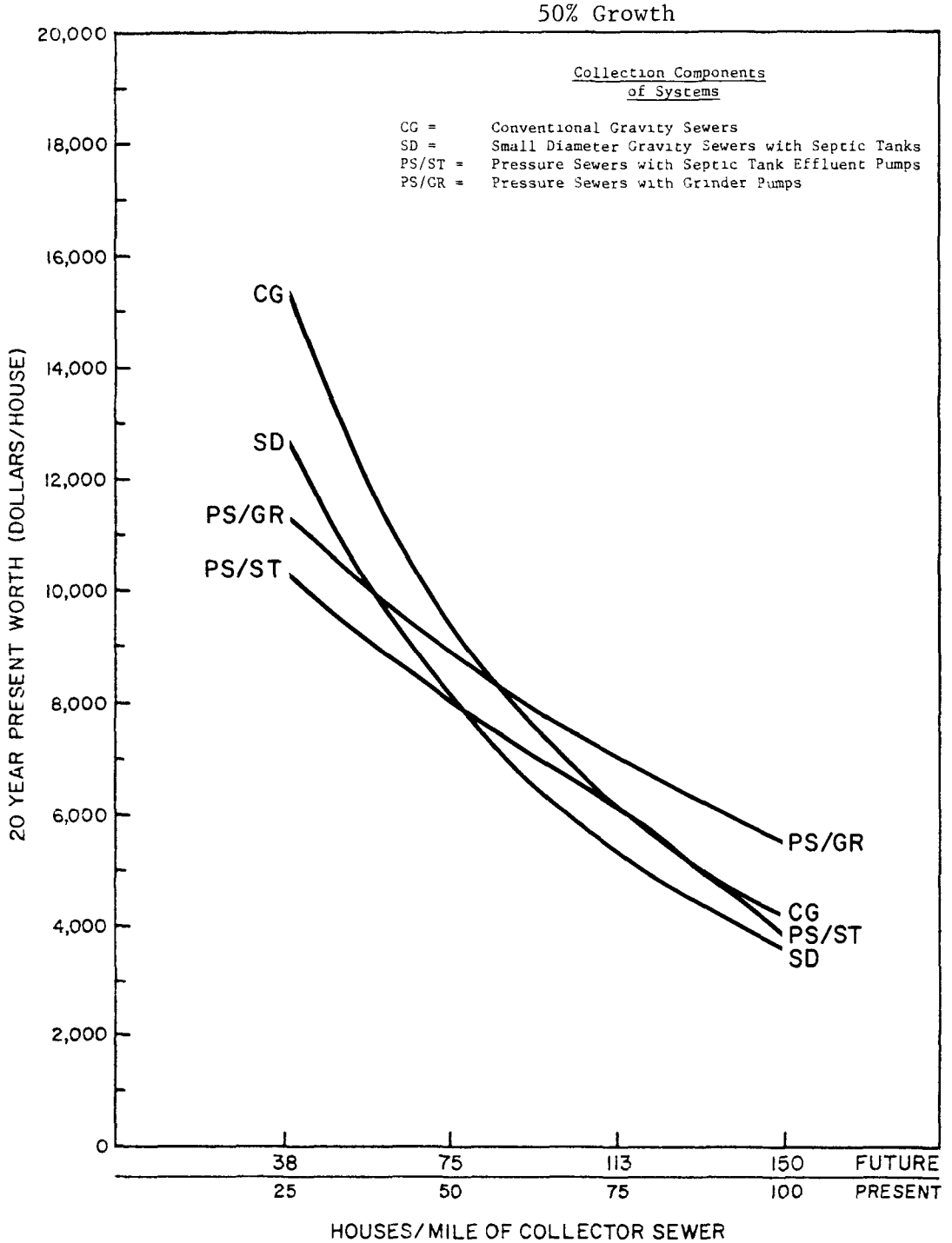

Figure IV-A-4-67. Cost comparison of centralized collection, transmission, and treatment systems for Scenario 7; 50% growth.

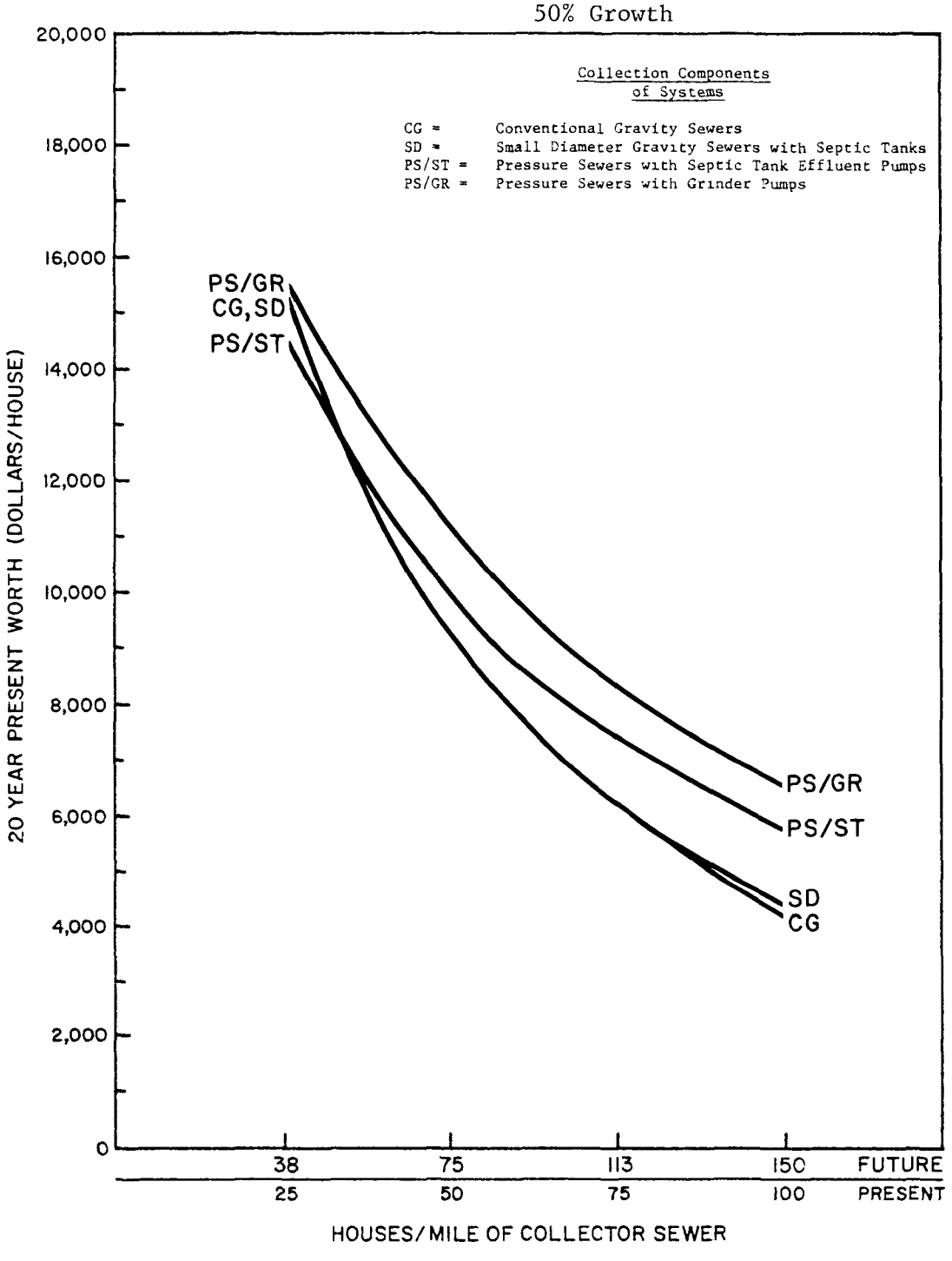

Figure IV-A-4-68. Cost comparison of centralized collection, transmission, and treatment systems for Scenario 8; 50% growth.

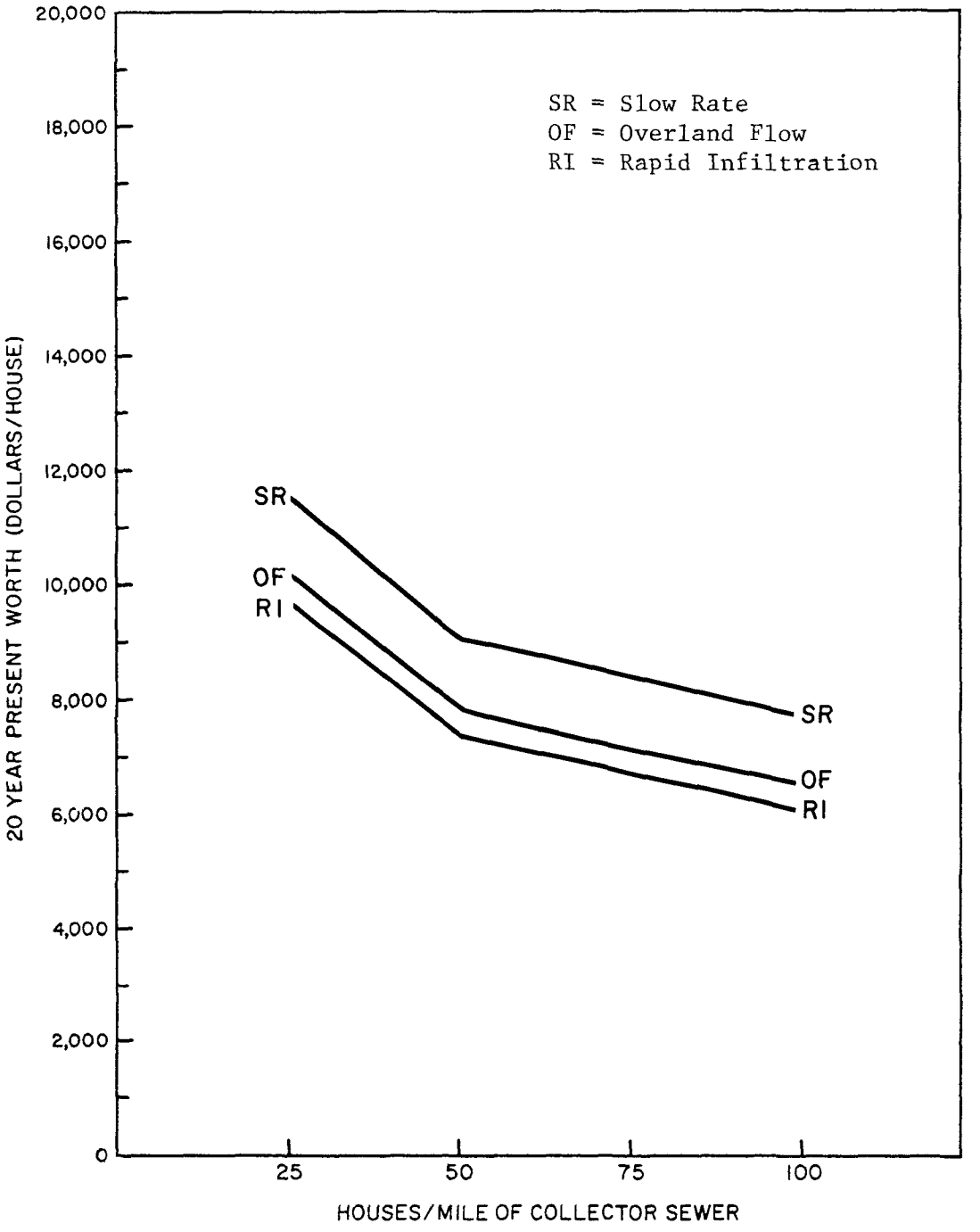

Figure IV-A-4-69. Cost comparison of collection, transmission, and land application systems for Scenario 1; 0% growth.

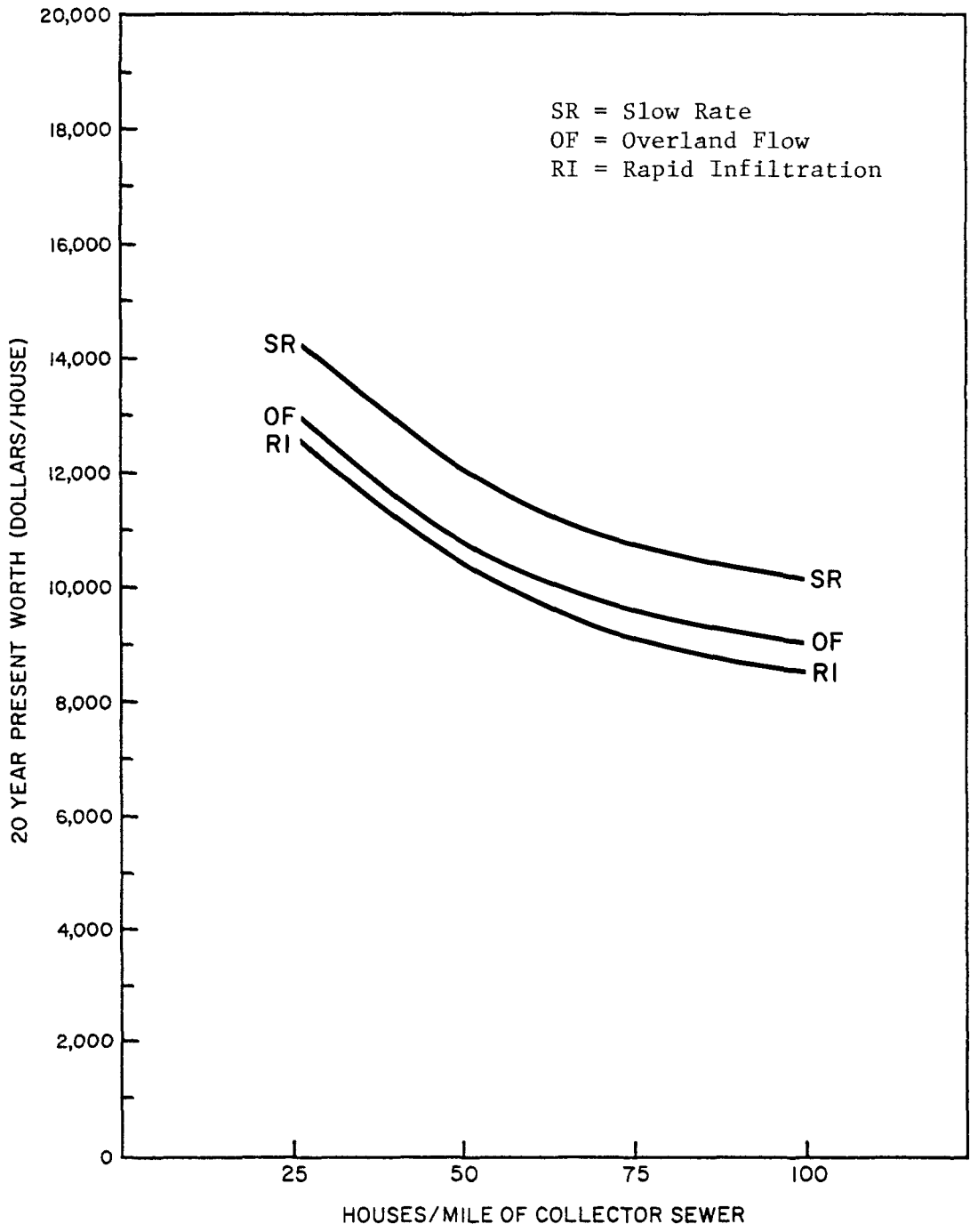

Figure IV-A-4-70. Cost comparison of collection, transmission, and land application systems for Scenario 2; 0% growth.

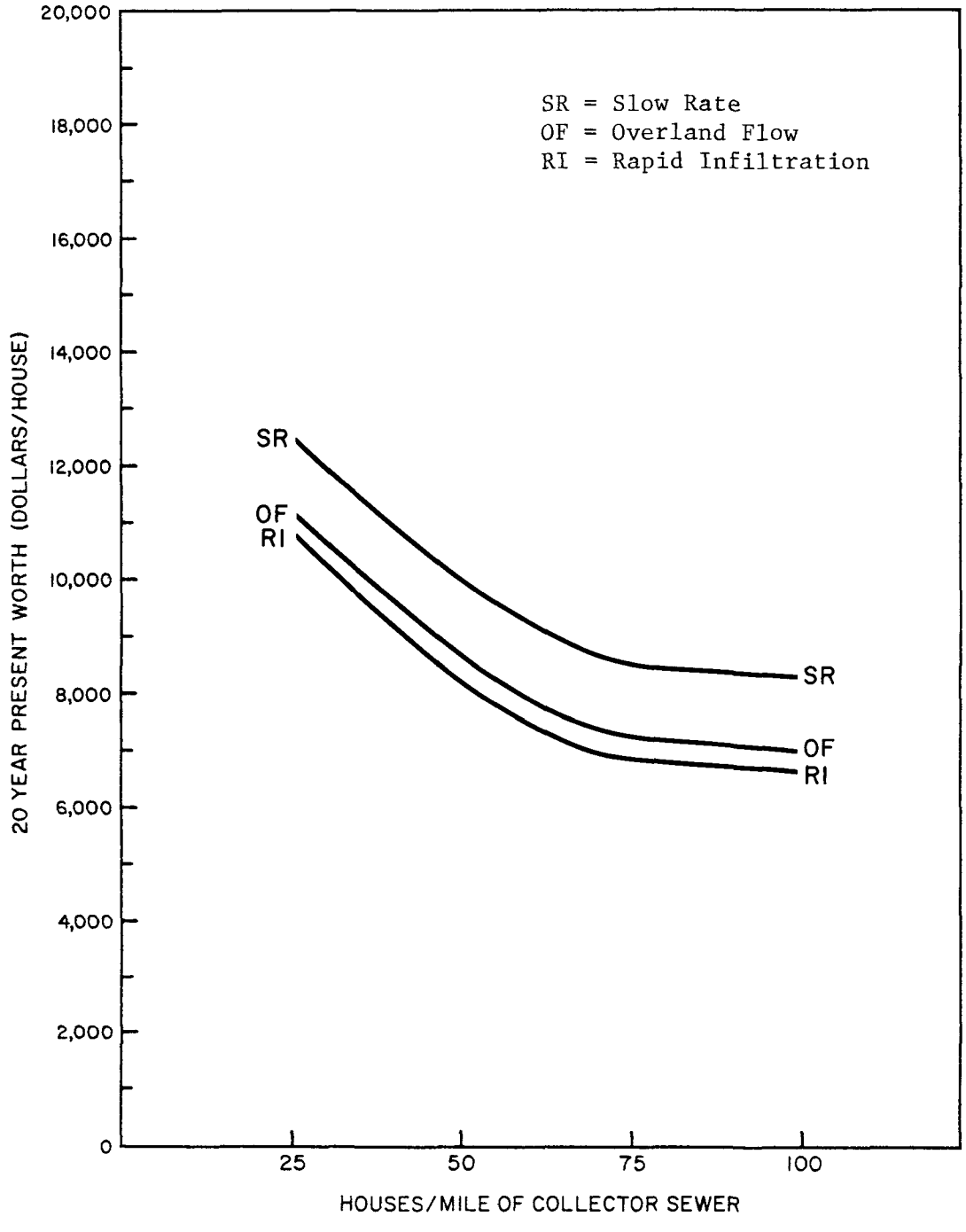

Figure IV-A-4-71. Cost comparison of collection, transmission, and land application systems for Scenario 3; 0% growth.

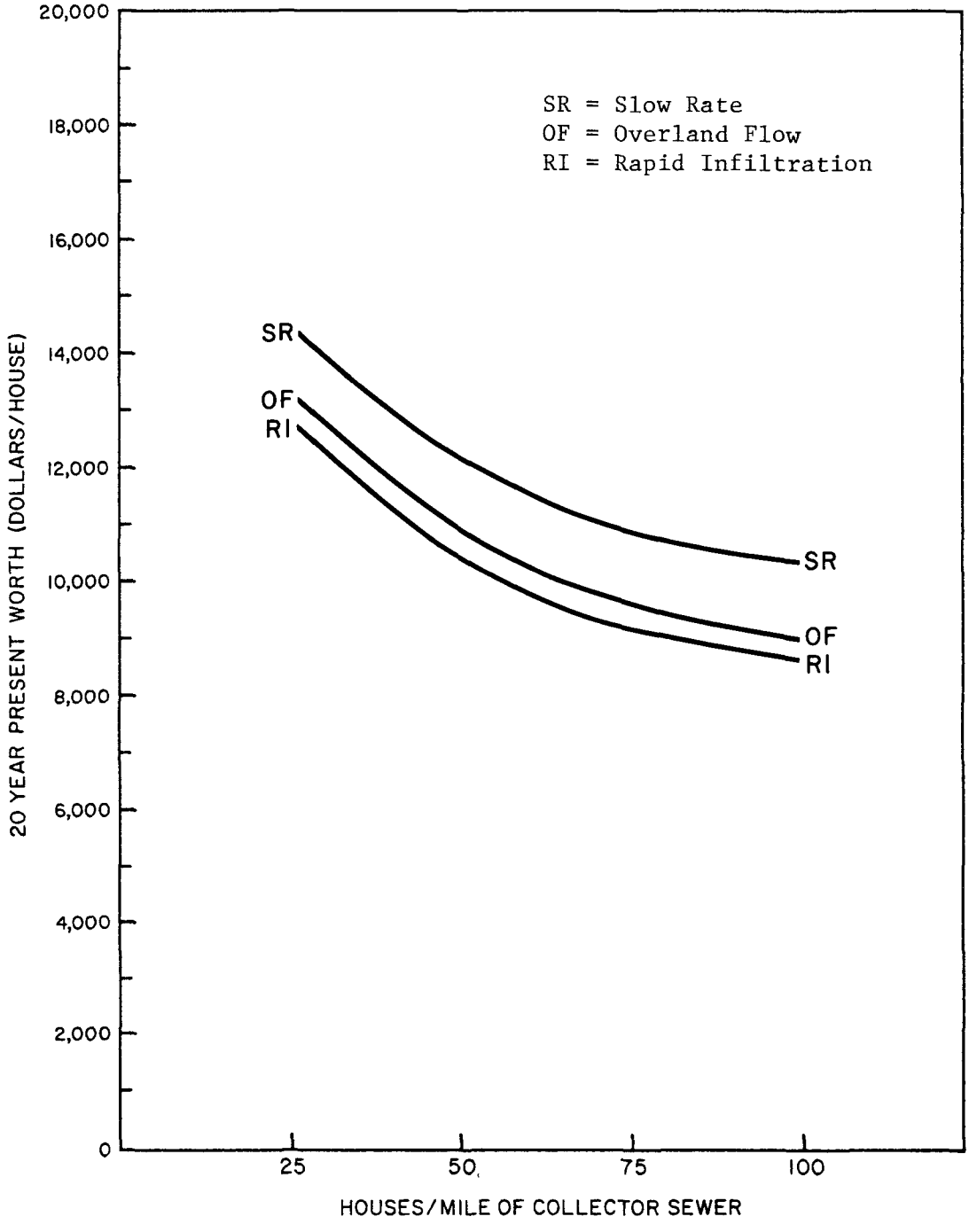

Figure IV-A-4-72. Cost comparison of collection, transmission, and land application systems for Scenario 4; 0% growth.

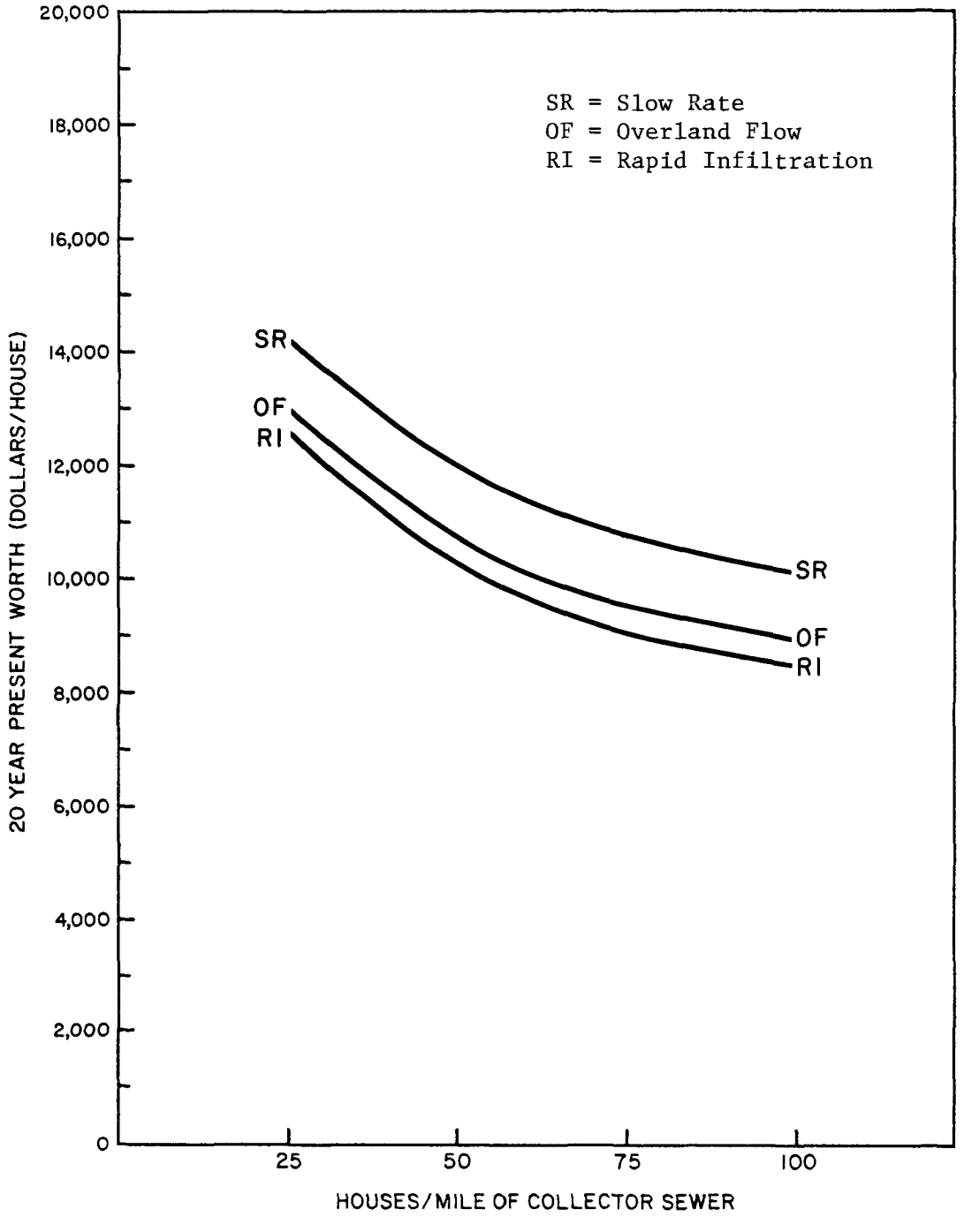

Figure IV-A-4-73. Cost comparison of collection, transmission, and land application systems for Scenario 5; 0% growth.

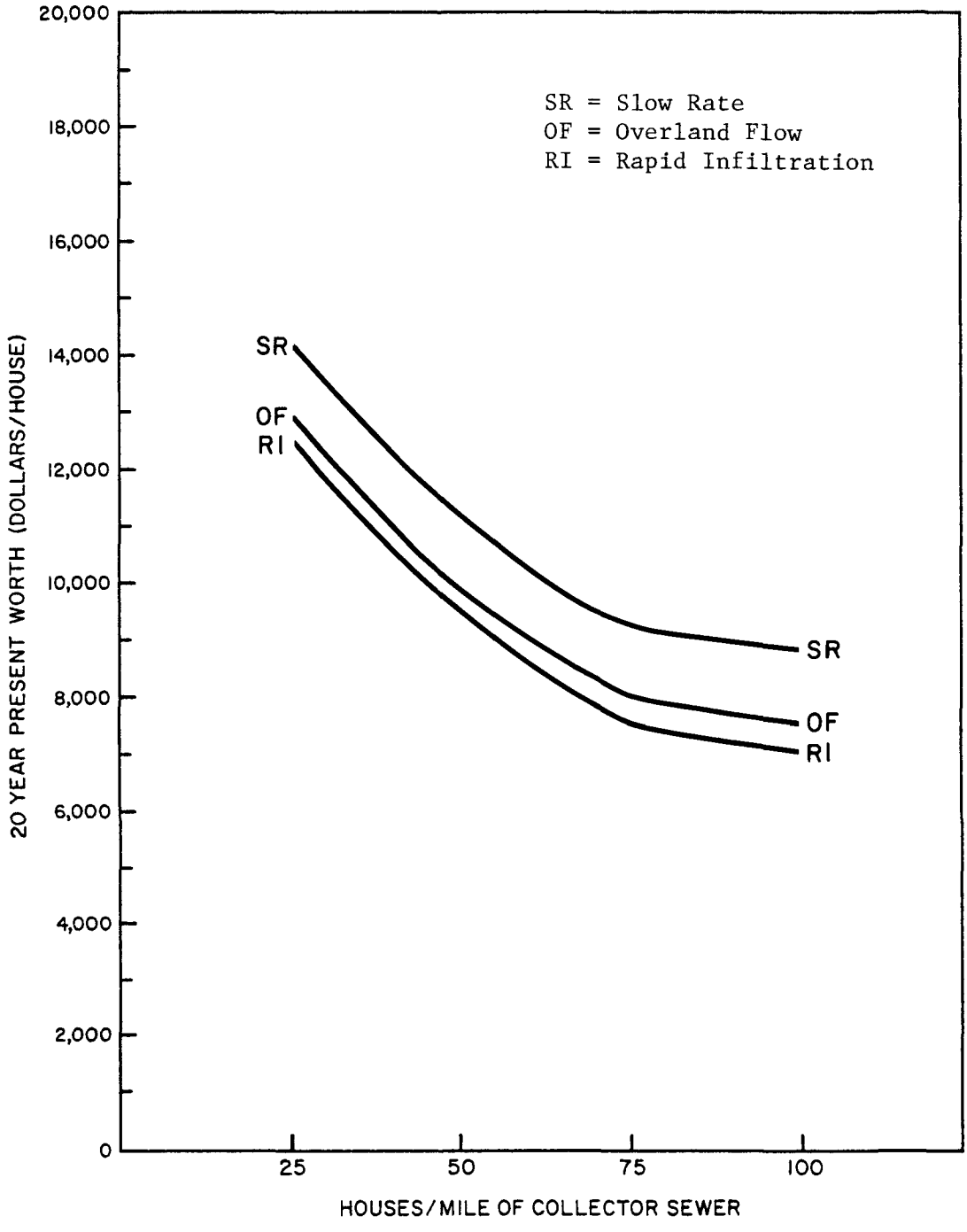

Figure IV-A-4-74. Cost comparison of collection, transmission, and land application systems for Scenario 6; 0% growth.

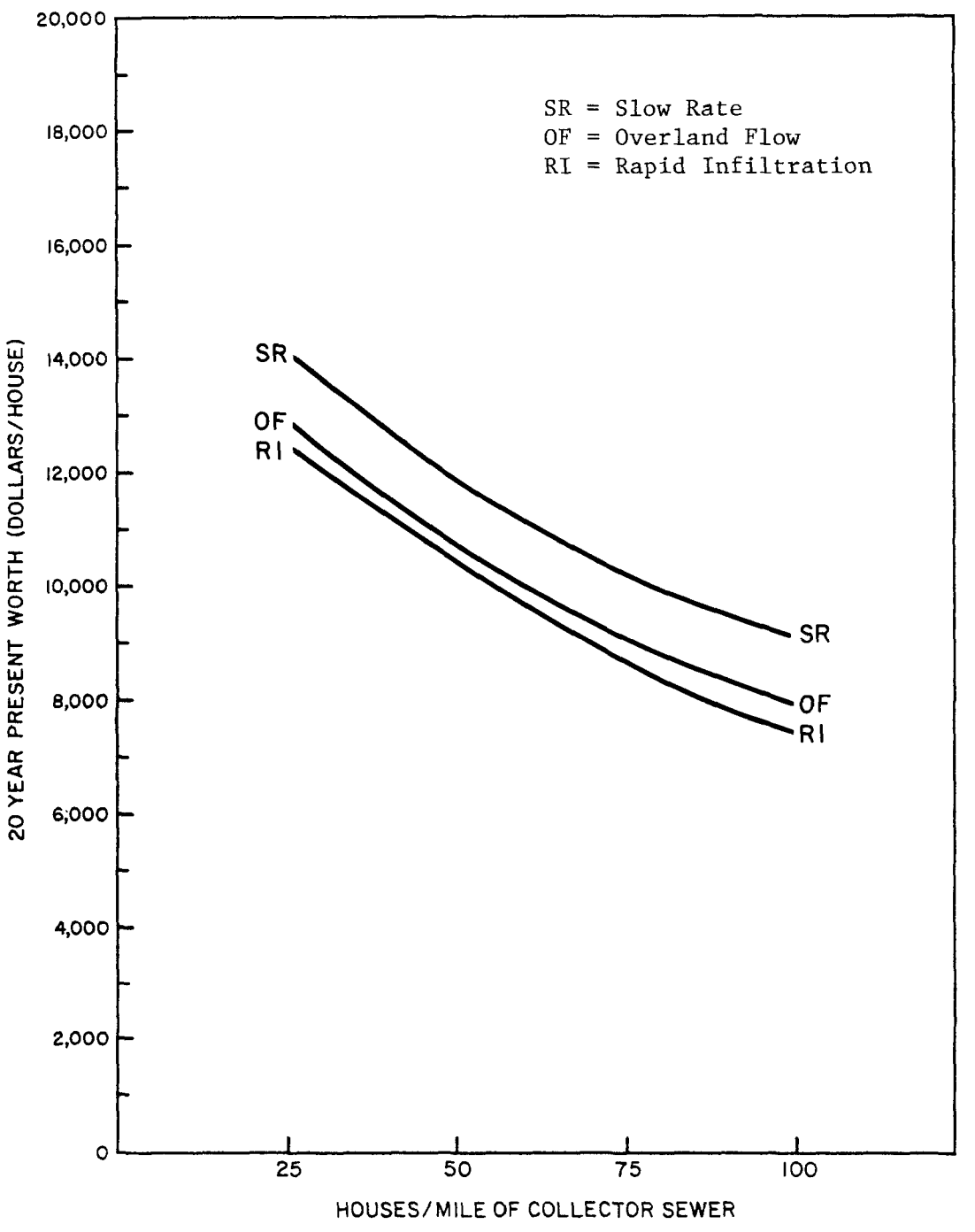

Figure IV-A-4-75. Cost comparison of collection, transmission, and land application systems for Scenario 7; 0% growth.

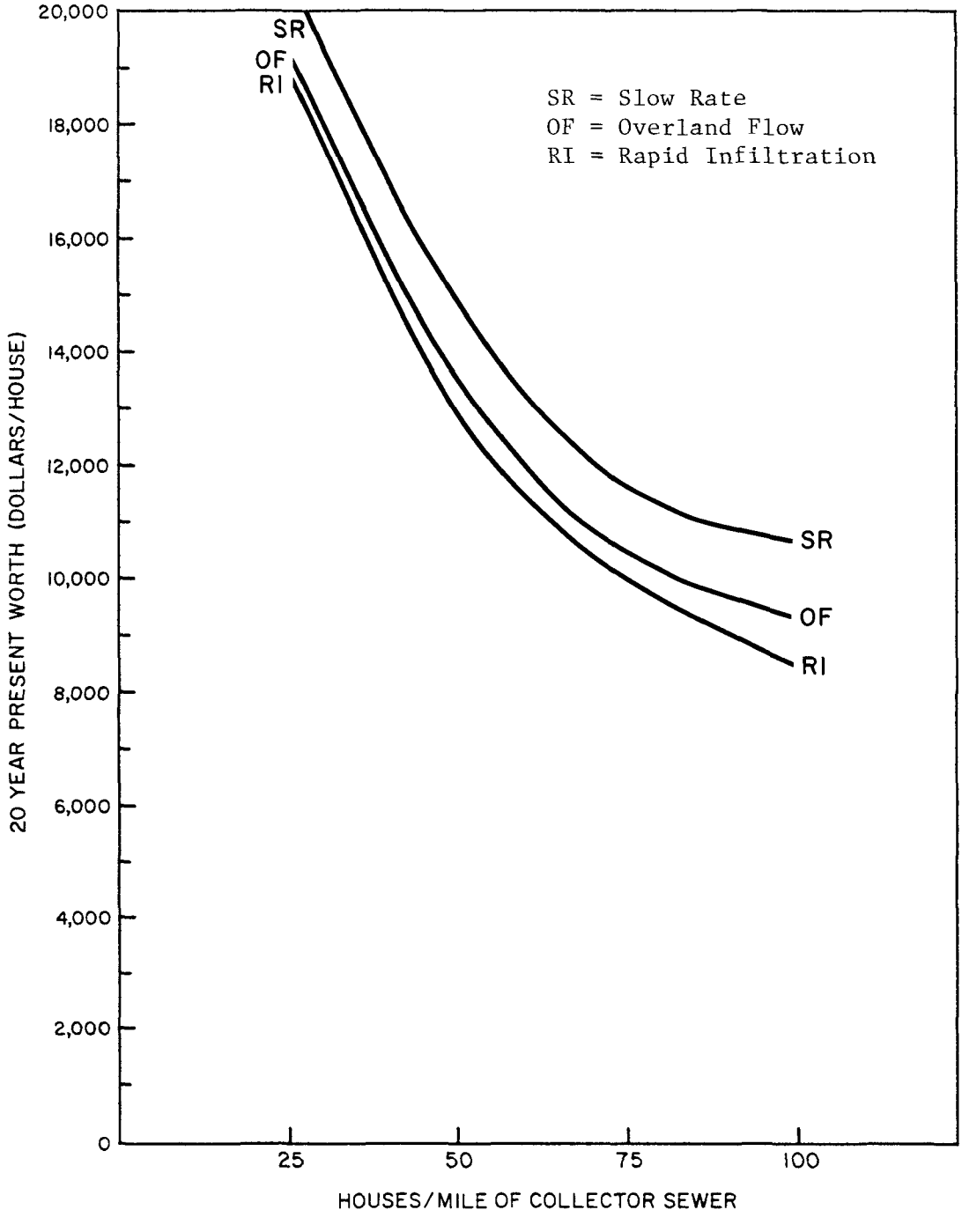

Figure IV-A-4-76. Cost comparison of collection, transmission, and land application systems for Scenario 8; 0% growth.

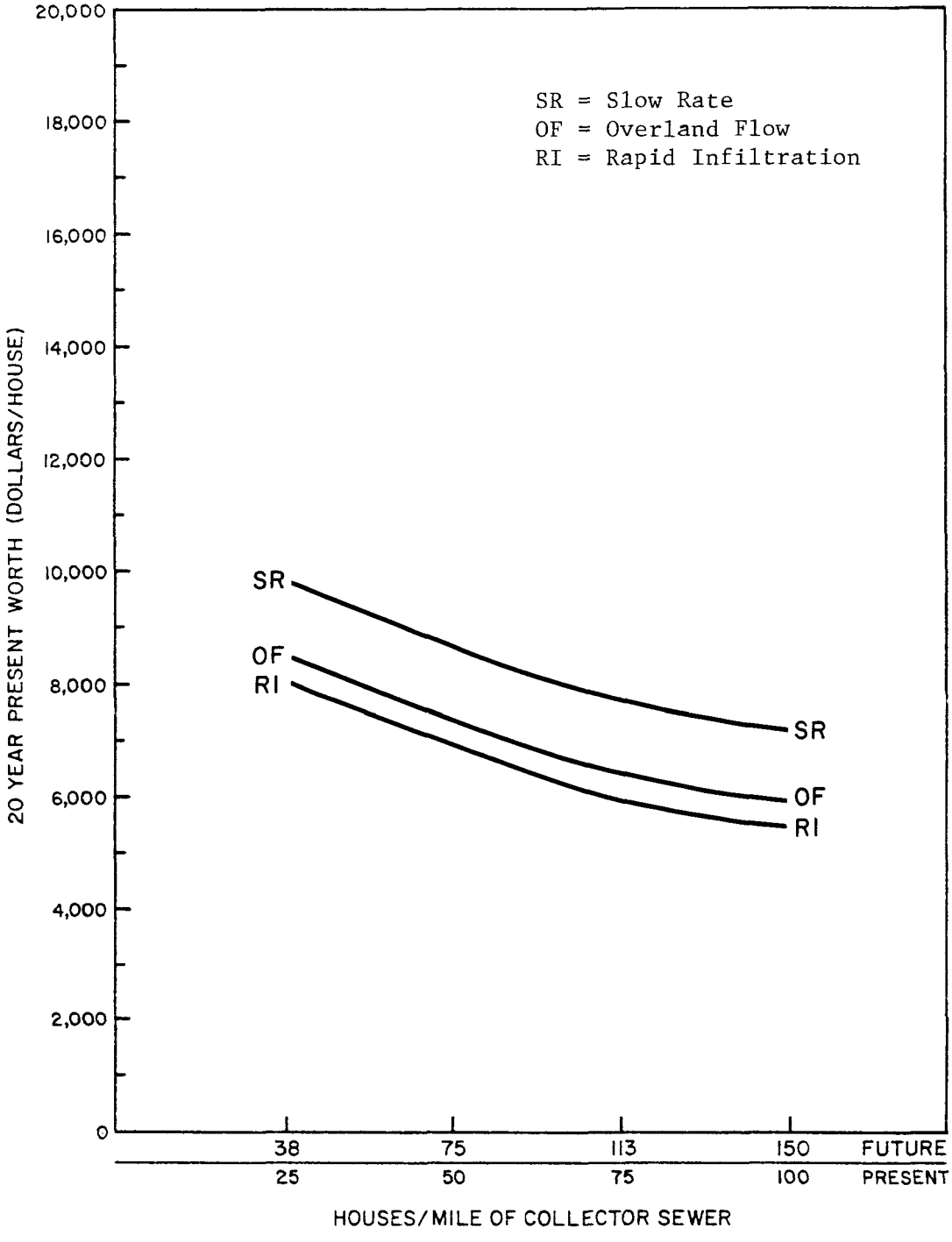

Figure IV-A-4-77. Cost comparison of collection, transmission, and land application systems for Scenario 1; 50% growth.

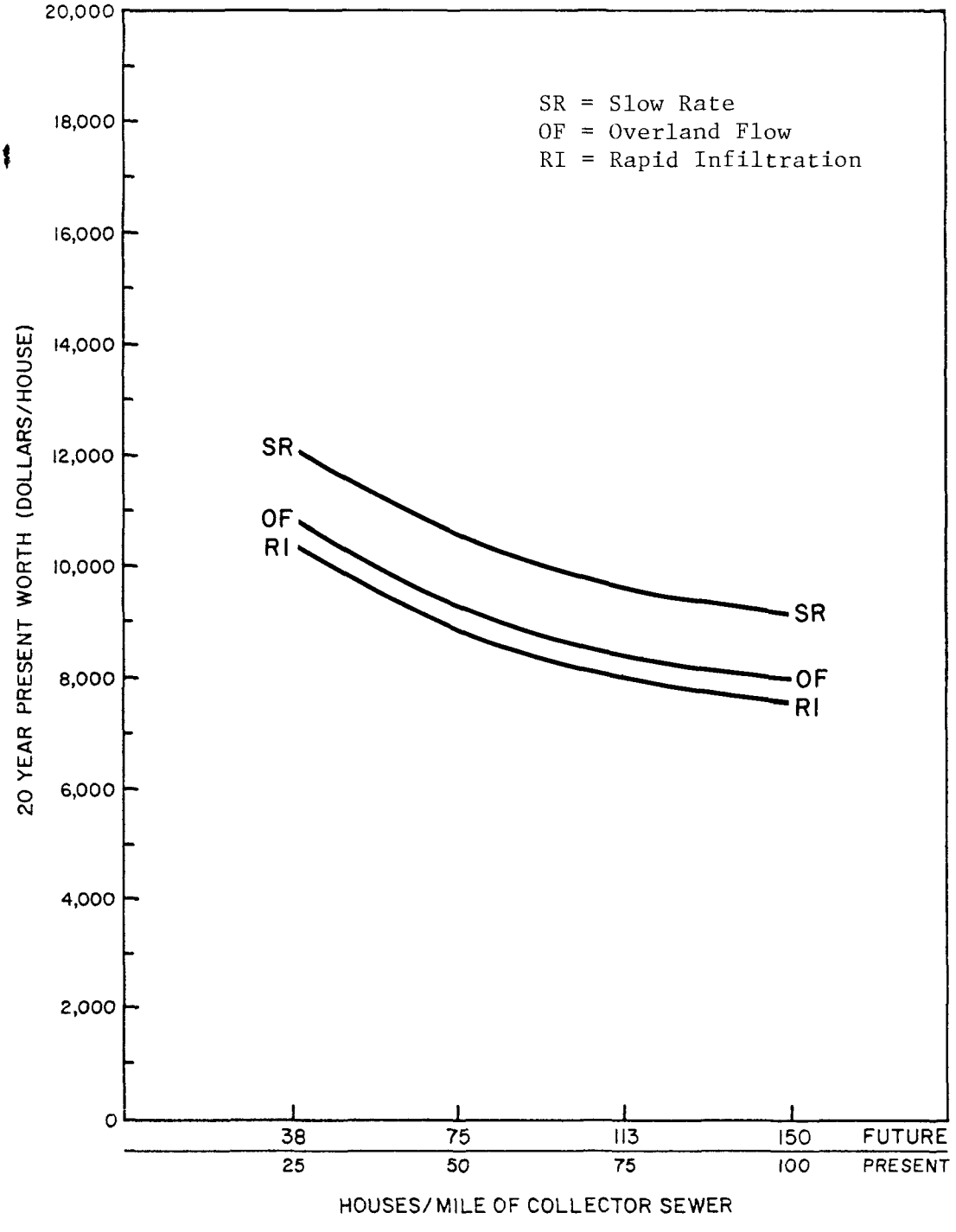

Figure IV-A-4-78. Cost comparison of collection, transmission, and land application systems for Scenario 2; 50% growth.

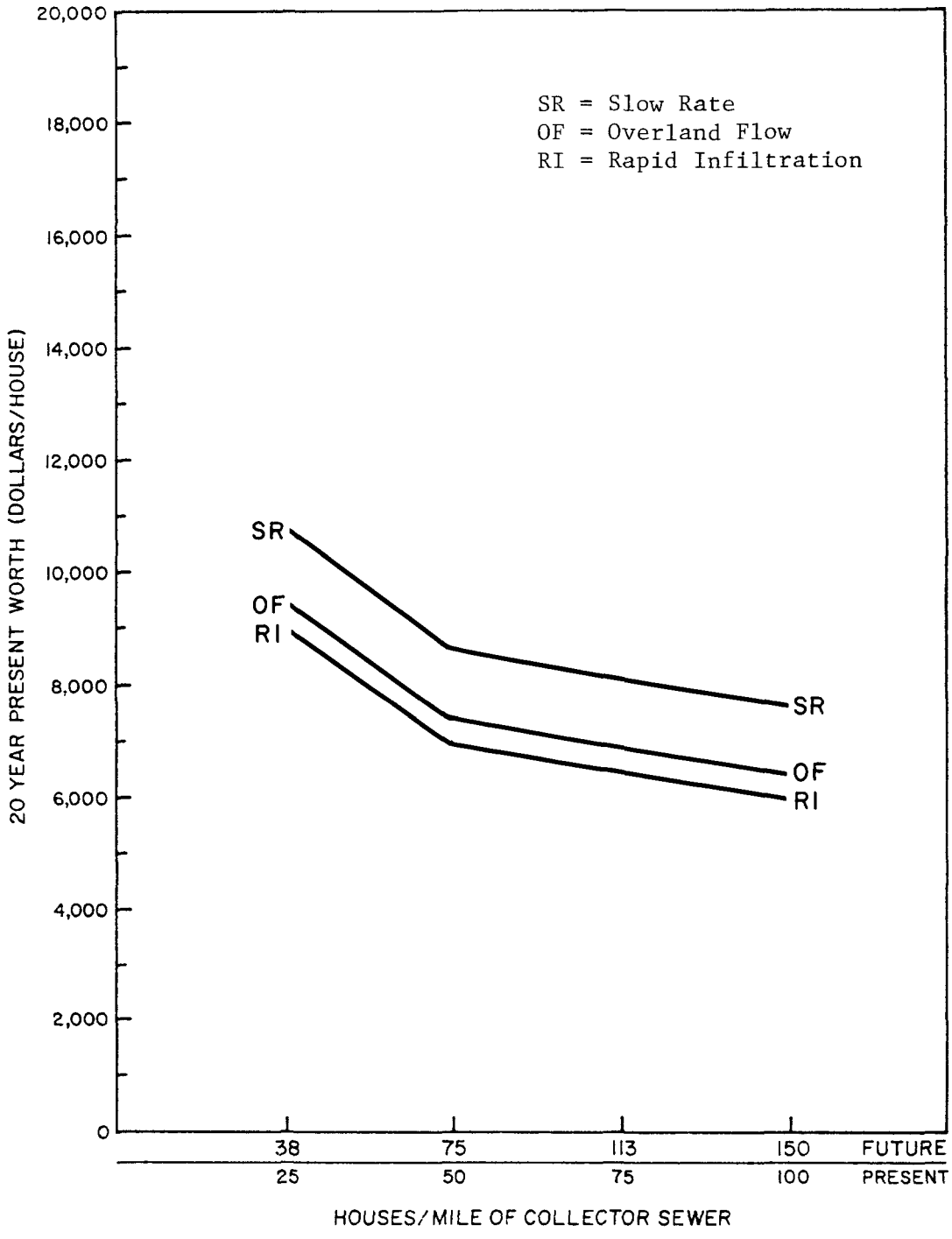

Figure IV-A-4-79. Cost comparison of collection, transmission, and land application systems for Scenario 3; 50% growth.

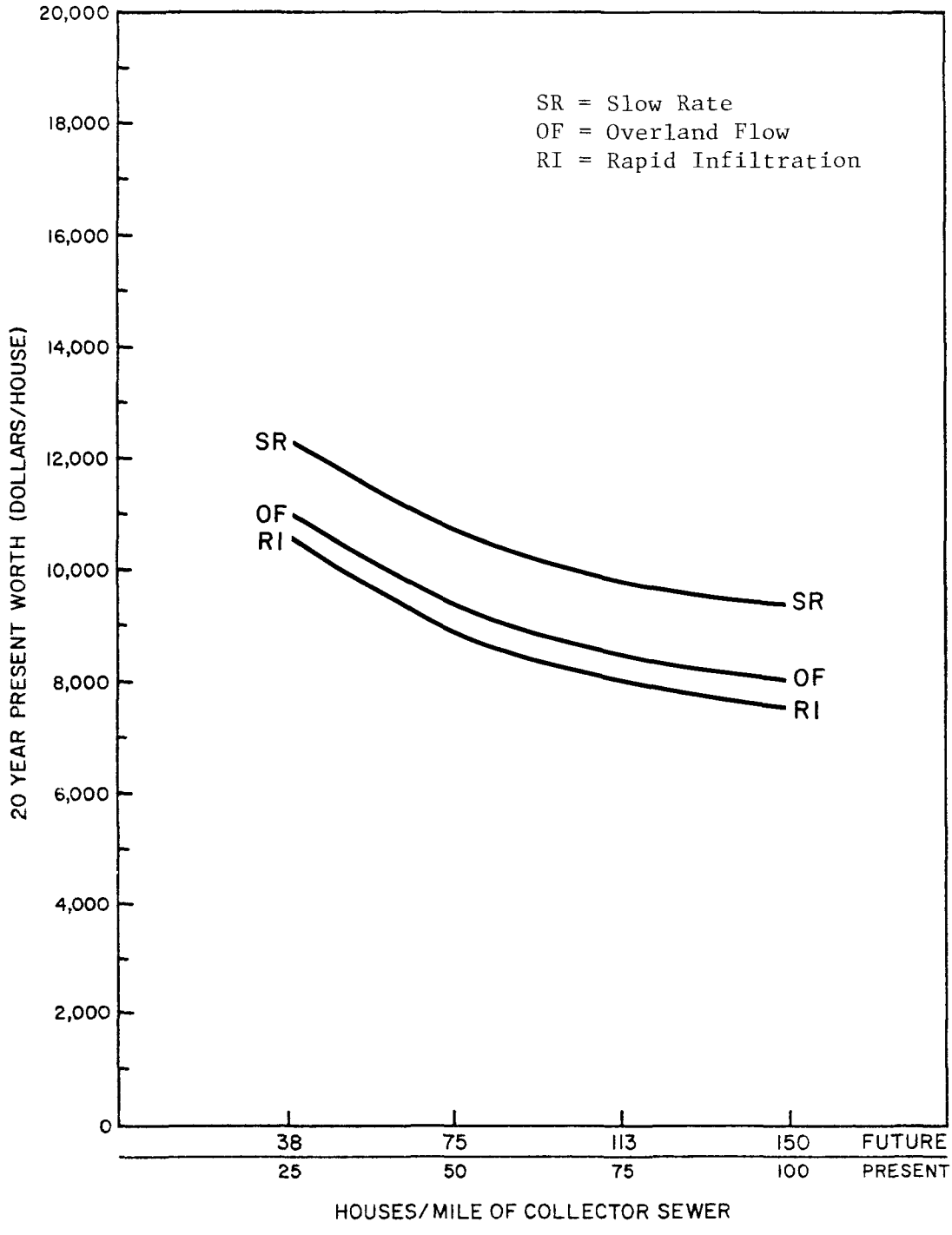

Figure IV-A-4-80. Cost comparison of collection, transmission, and land application systems for Scenario 4; 50% growth.

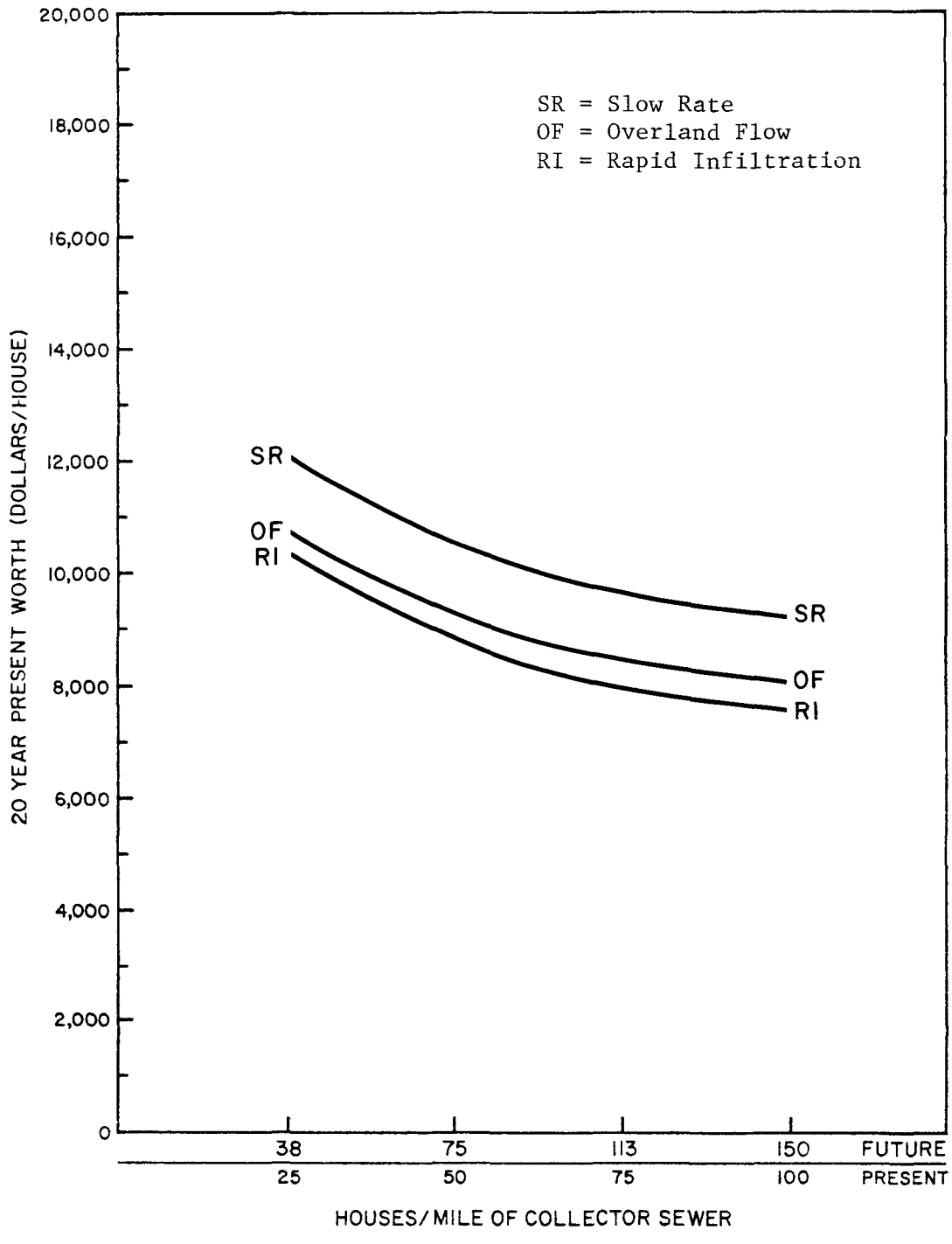

Figure IV-A-4-81. Cost comparison of collection, transmission, and land application systems for Scenario 5; 50% growth.

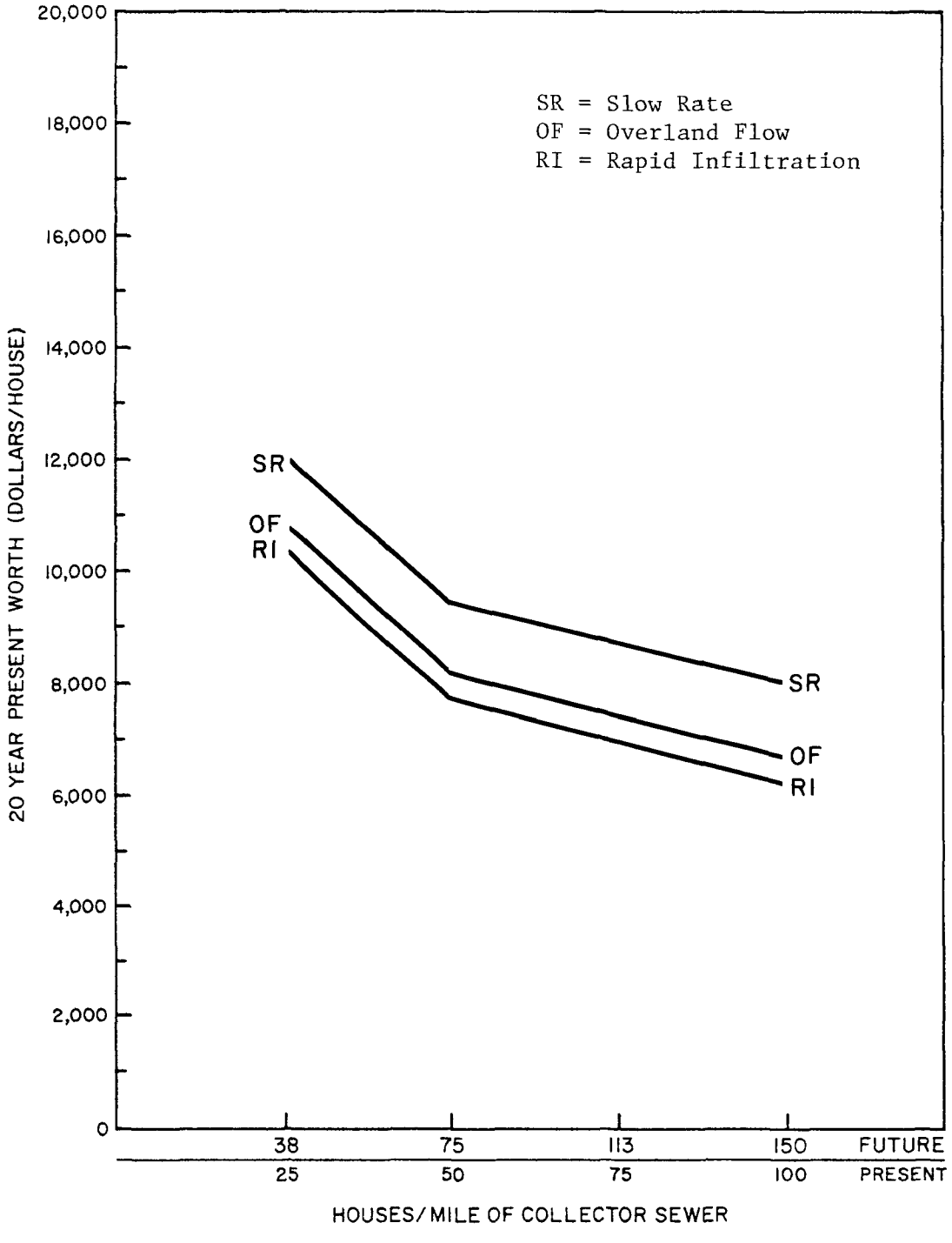

Figure IV-A-4-82. Cost comparison of collection, transmission, and land application systems for Scenario 6; 50% growth.

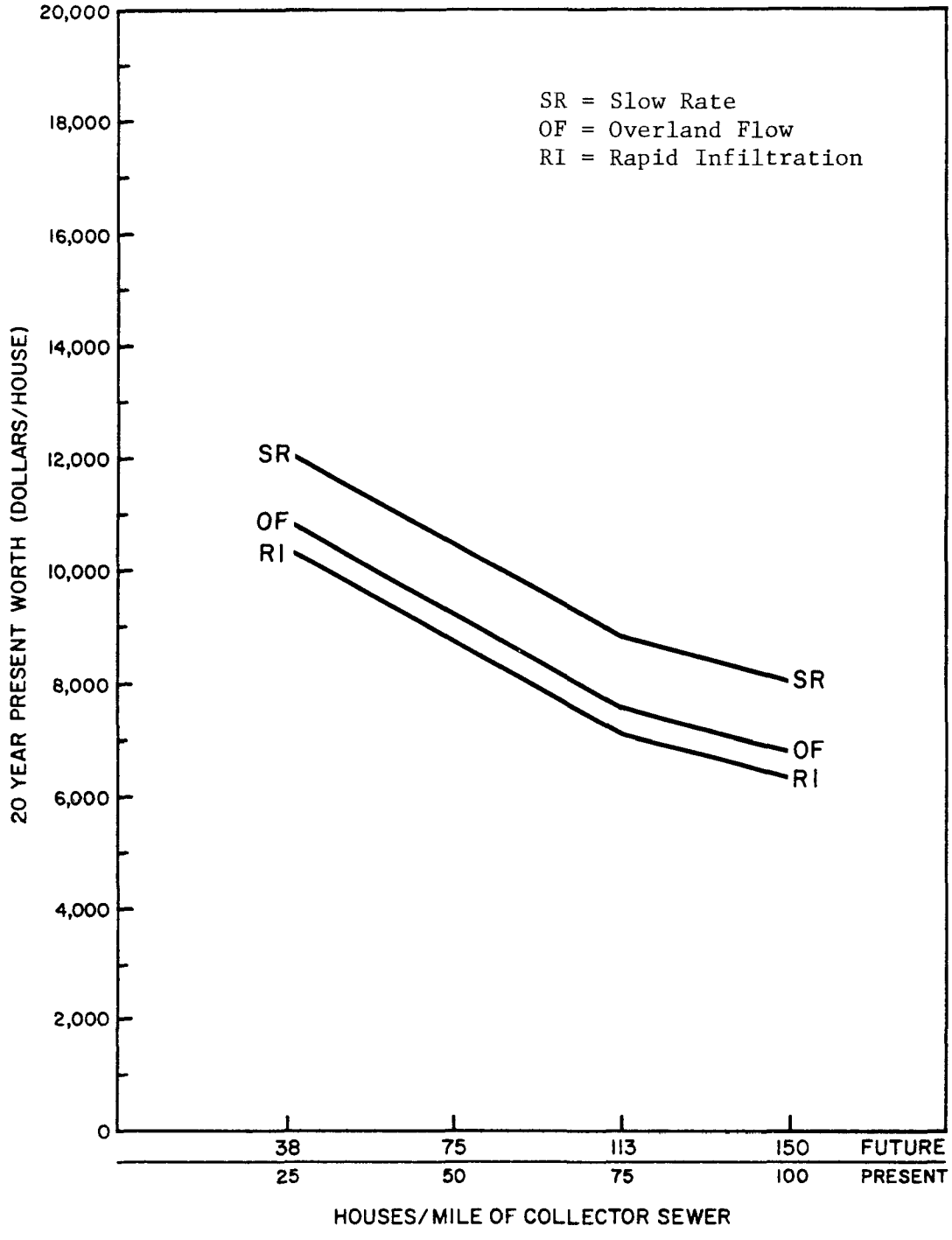

Figure IV-A-4-83. Cost comparison of collection, transmission, and land application systems for Scenario 7; 50% growth.

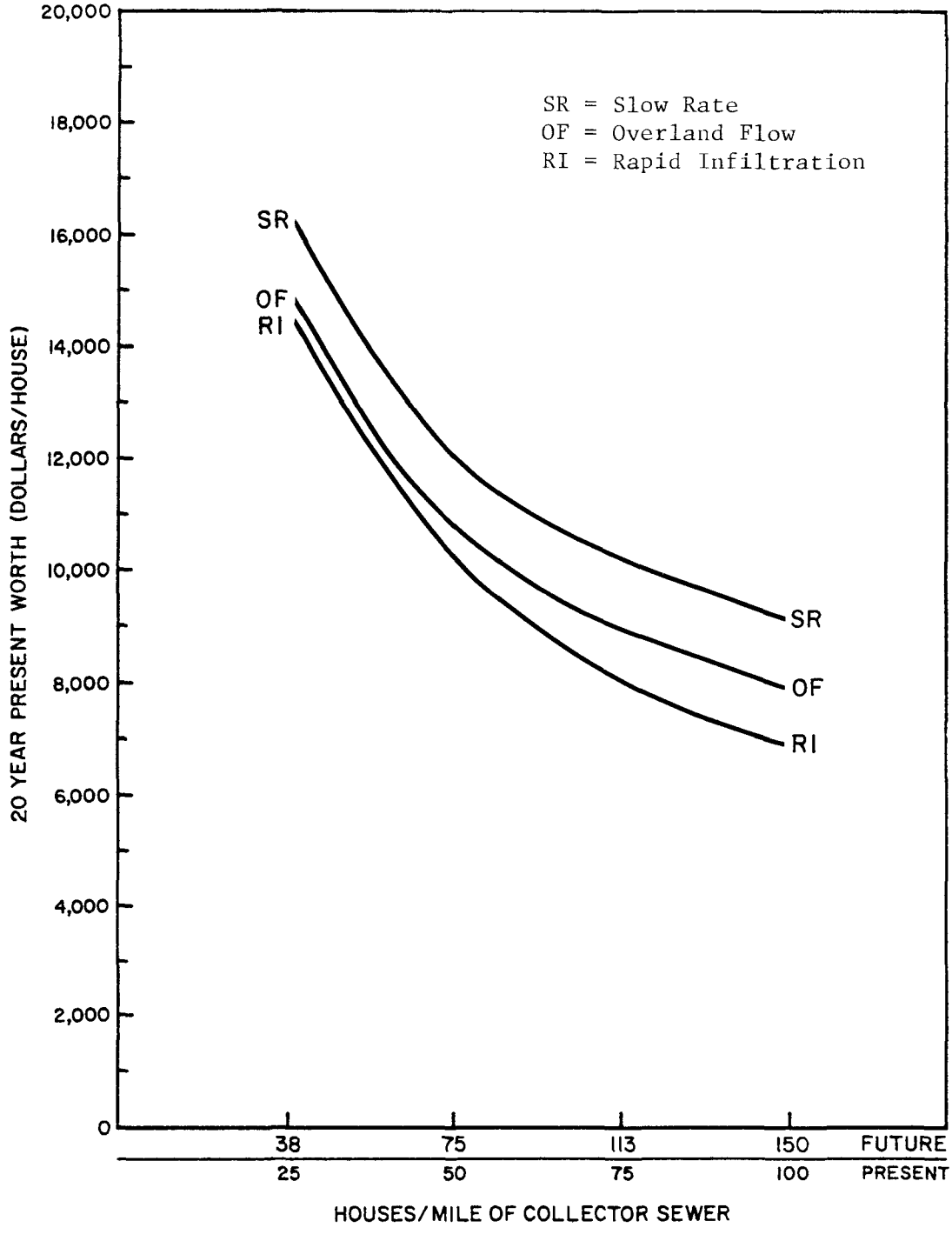

Figure IV-A-4-84. Cost comparison of collection, transmission, and land application systems for Scenario 8; 50% growth.

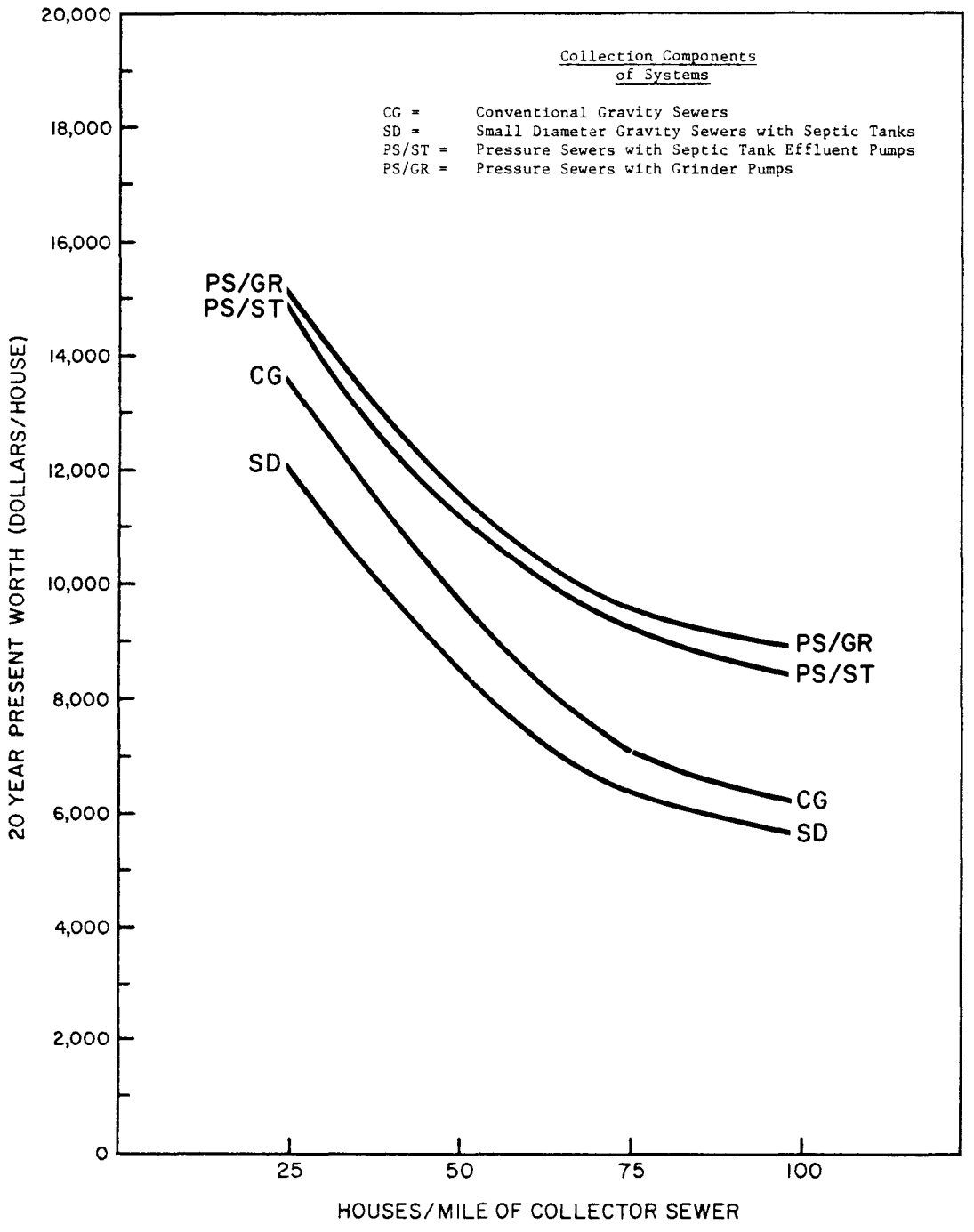

Figure IV-A-4-85. Cost comparison of collection, transmission, and cluster systems for Scenario 1; 0% growth.

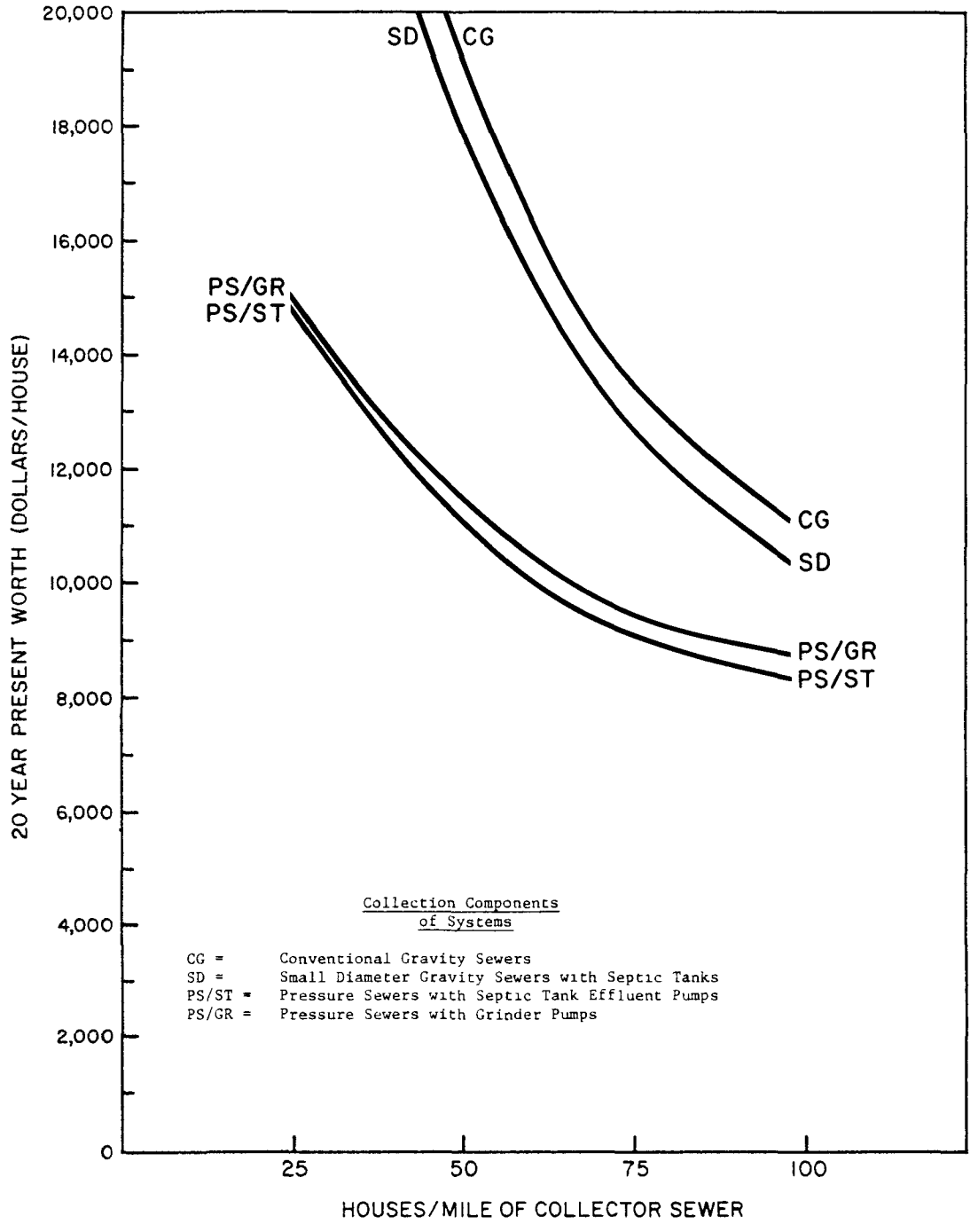

Figure IV-A-4-86. Cost comparison of collection, transmission, and cluster systems for Scenario 2; 0% growth.

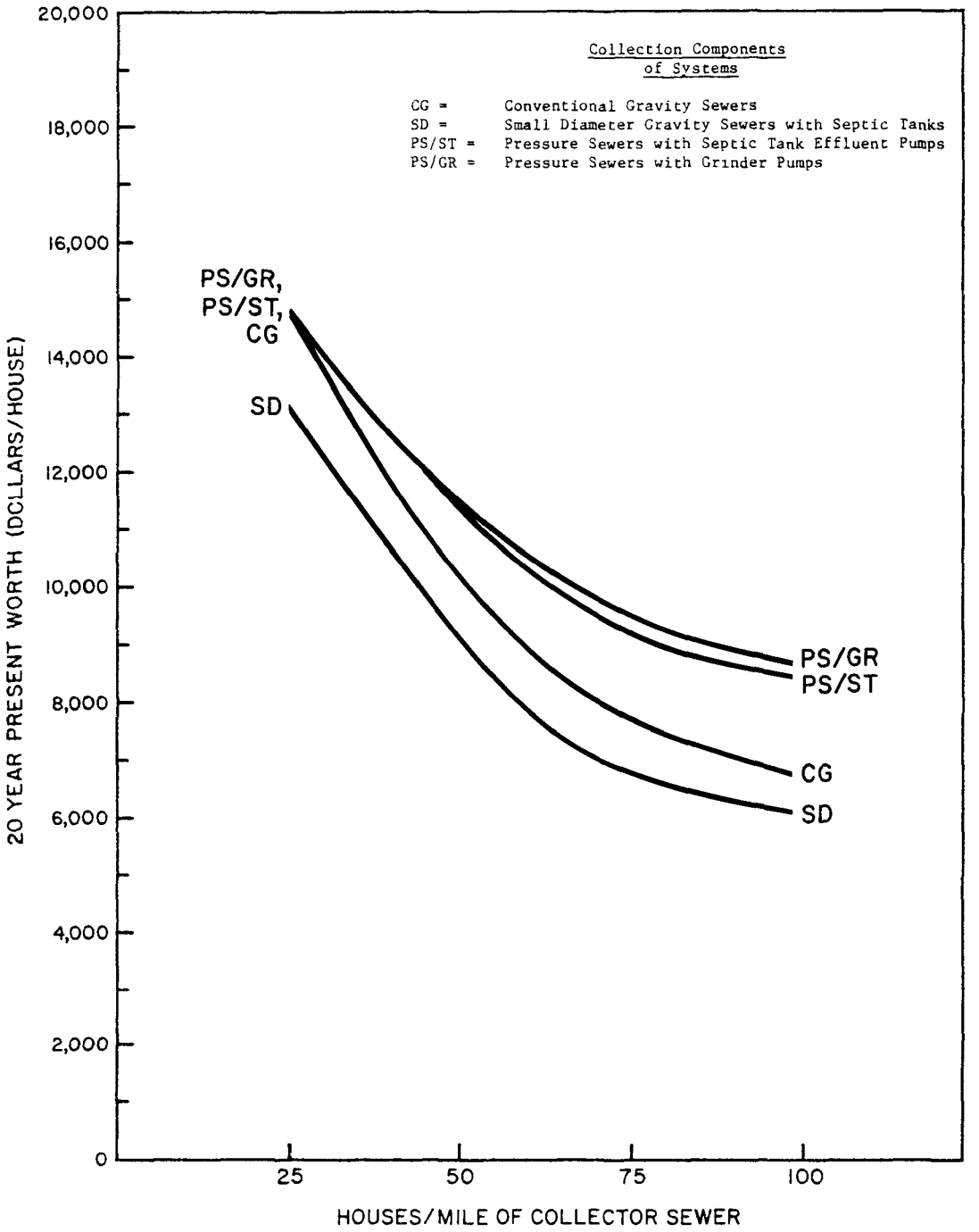

Figure IV-A-4-87. Cost comparison of collection, transmission, and cluster systems for Scenario 3; 0% growth.

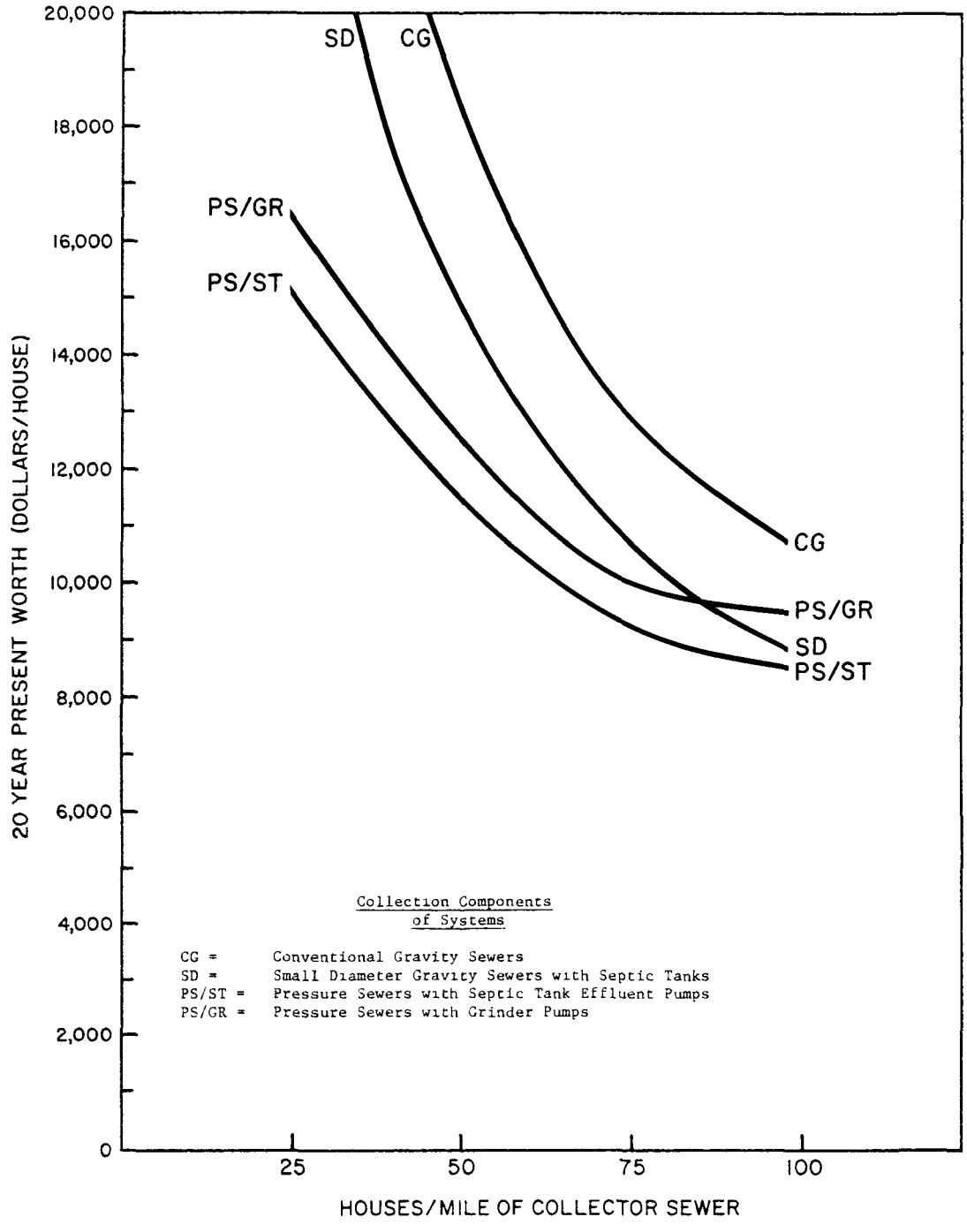

Figure IV-A-4-88. Cost comparison of collection, transmission, and cluster systems for Scenario 4; 0% growth.

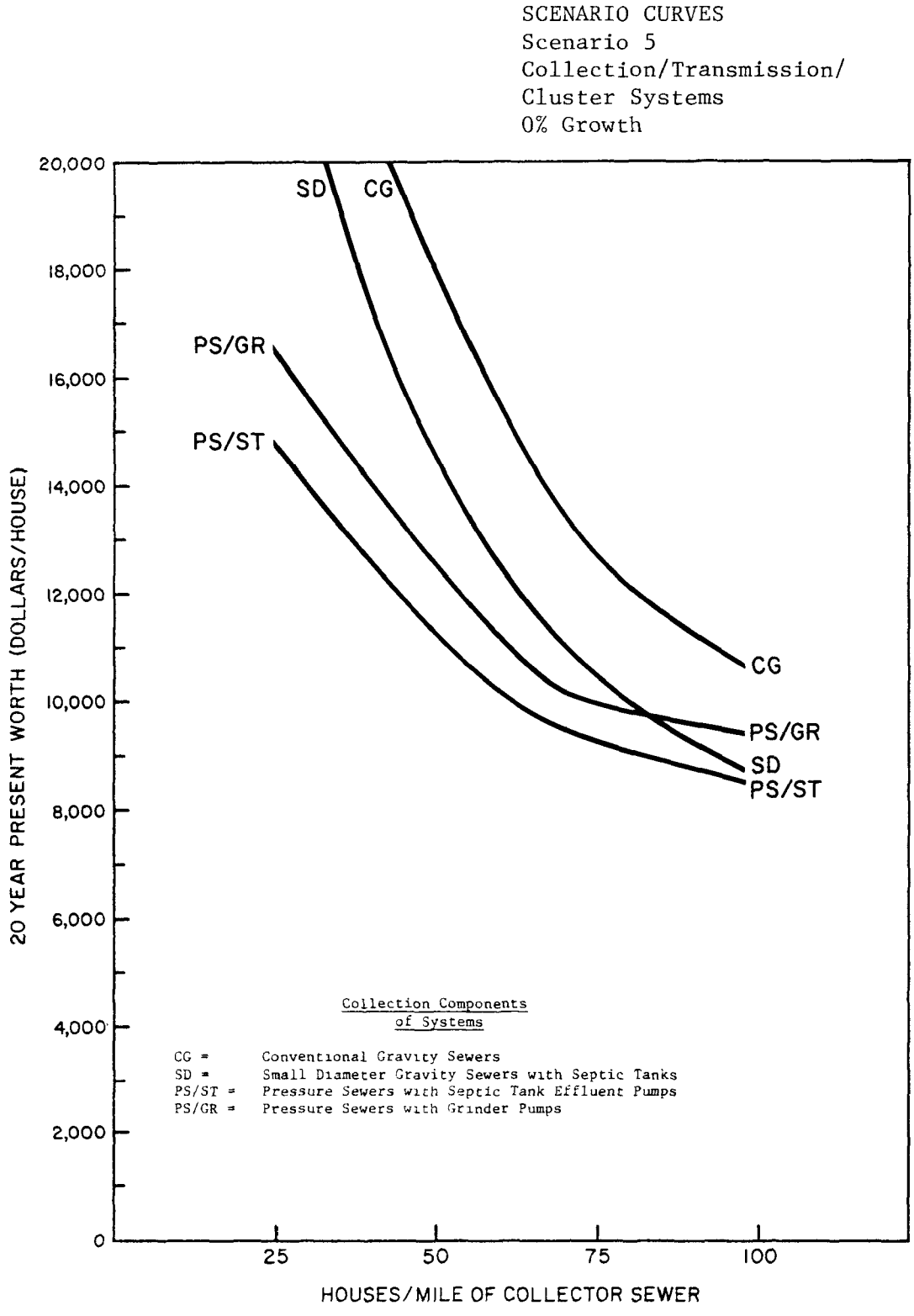

Figure IV-A-4-89. Cost comparison of collection, transmission, and cluster systems for Scenario 5; 0% growth.

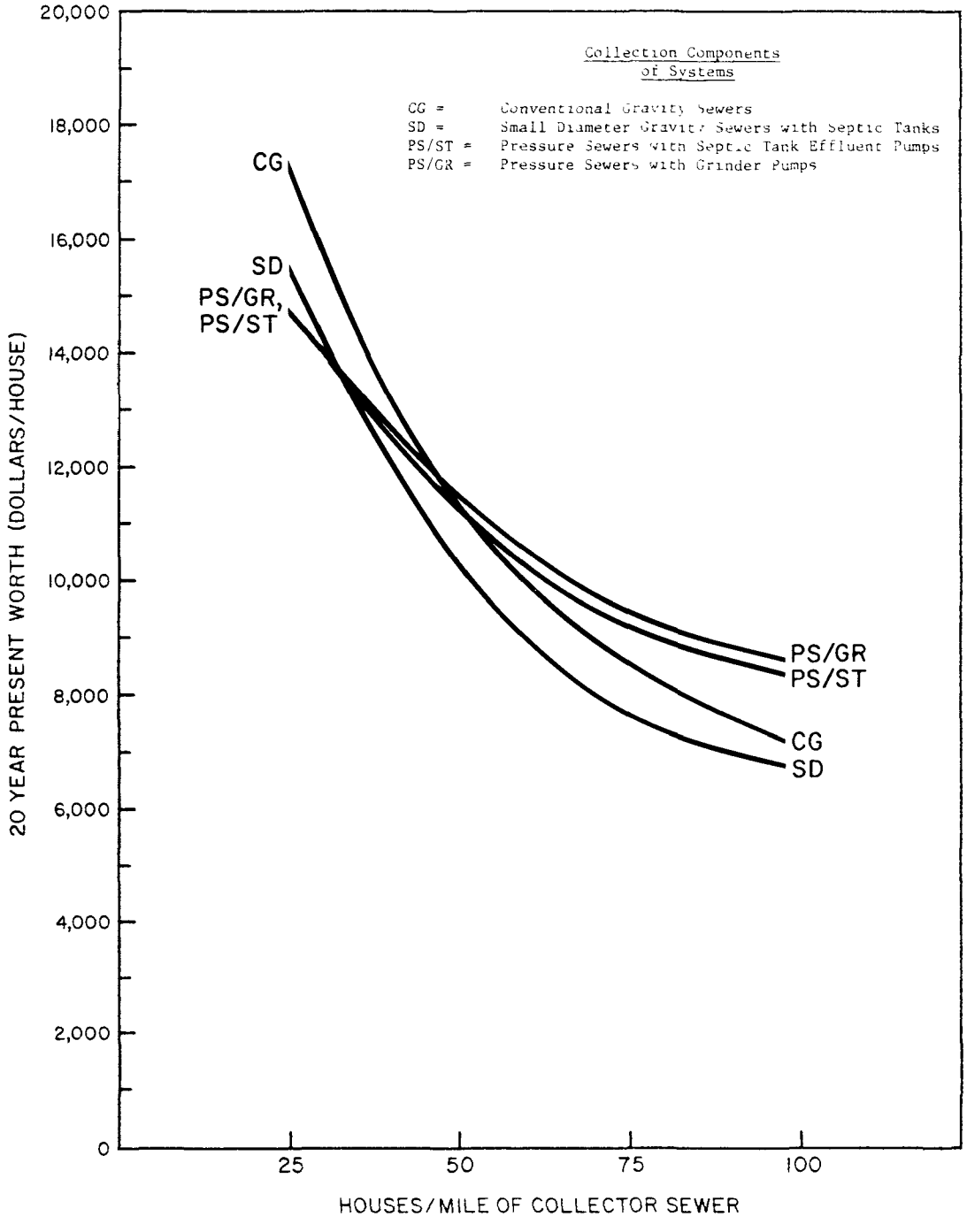

Figure IV-A-4-90. Cost comparison of collection, transmission, and cluster systems for Scenario 6; 0% growth.

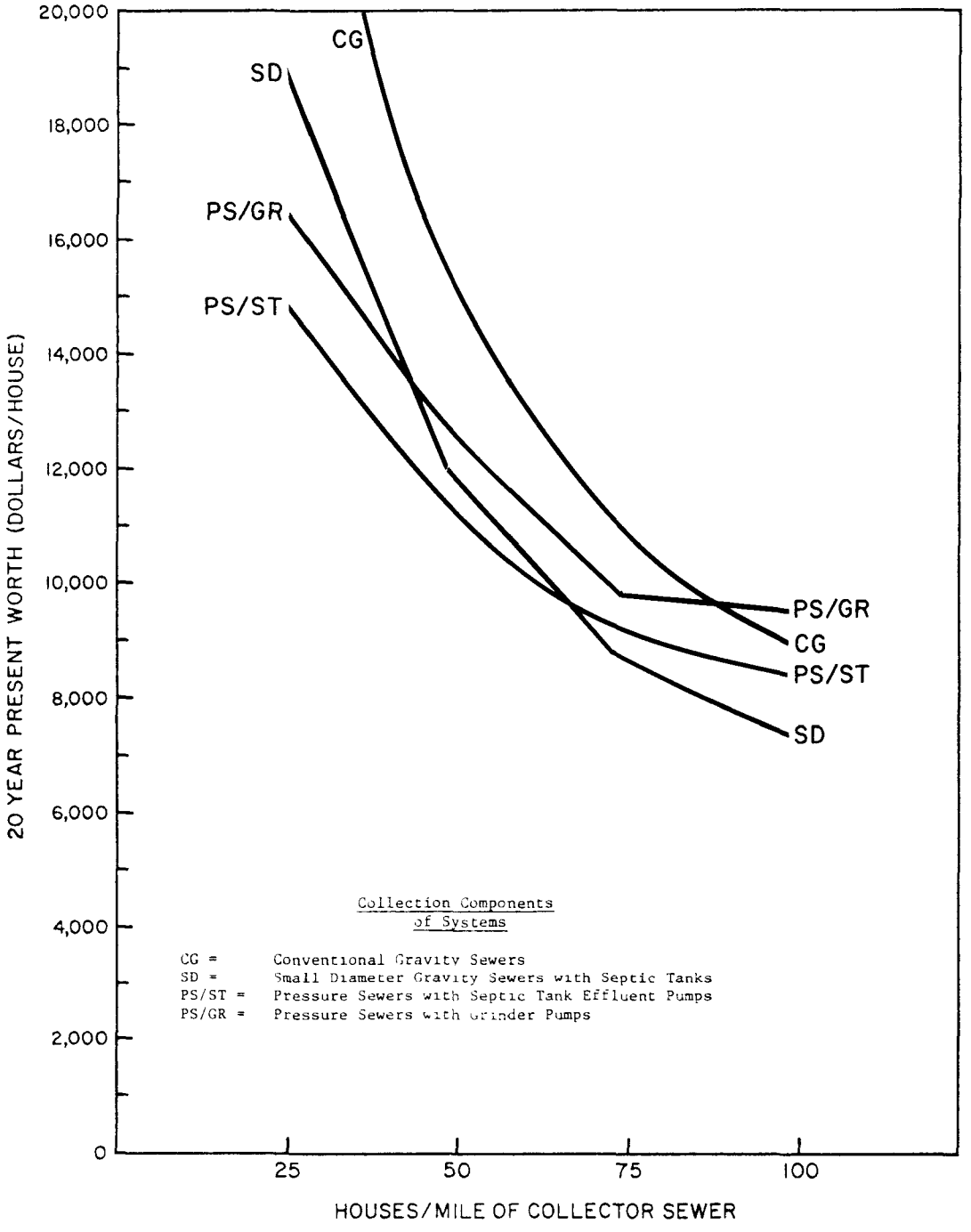

Figure IV-A-4-91. Cost comparison of collection, transmission, and cluster systems for Scenario 7; 0% growth.

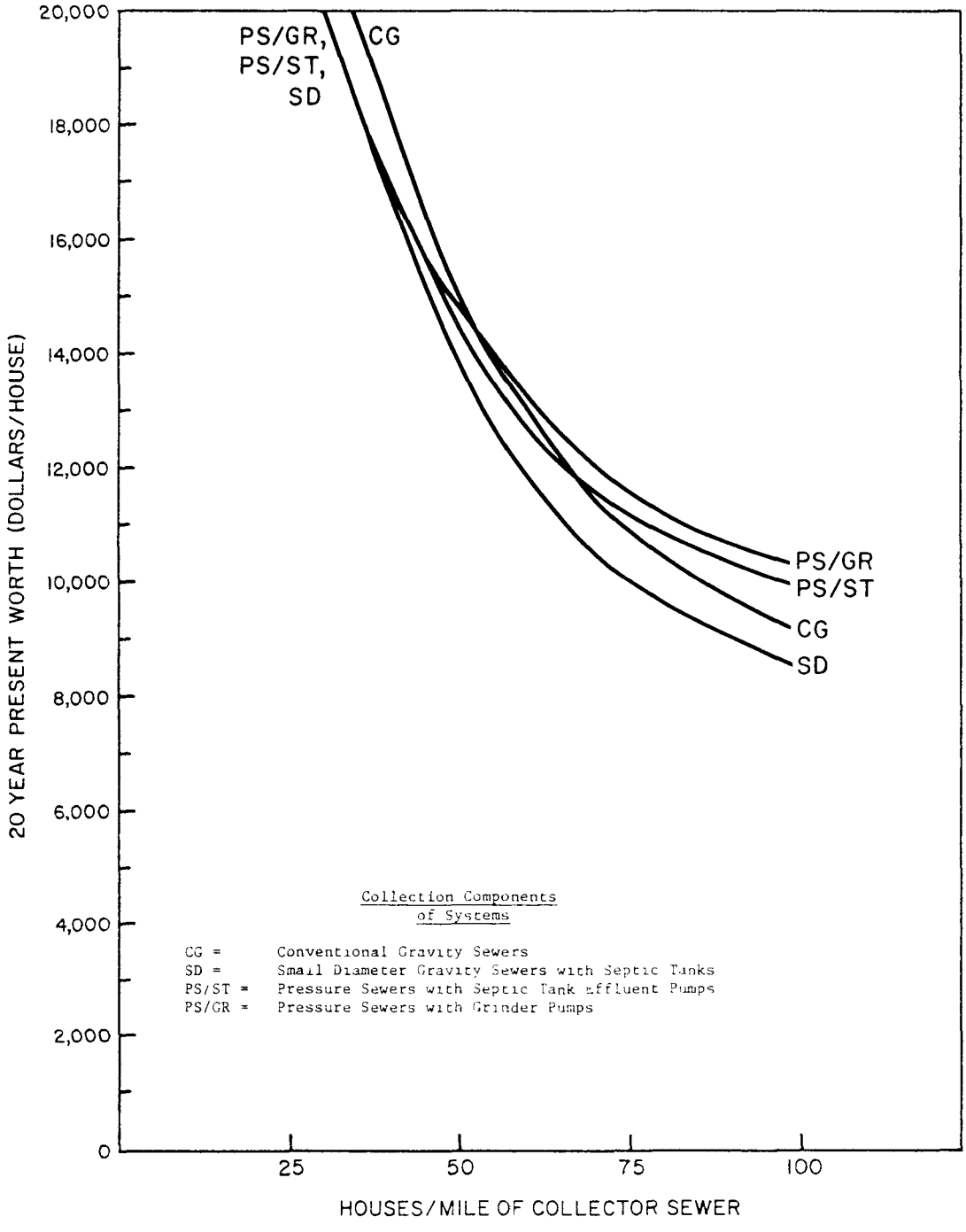

Figure IV-A-4-92. Cost comparison of collection, transmission, and cluster systems for Scenario 8: 0% growth.

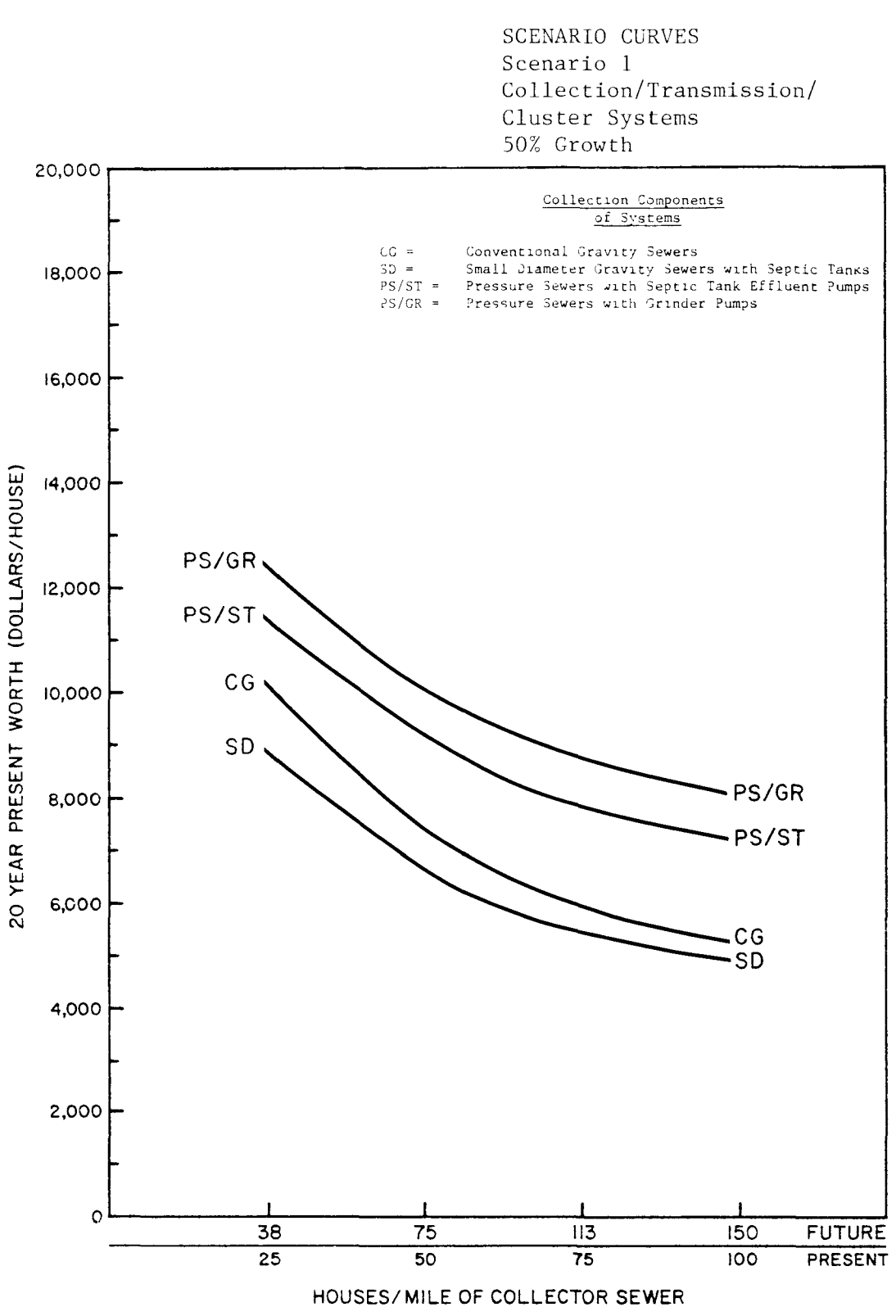

Figure IV-A-4-93. Cost comparison of collection, transmission, and cluster systems for Scenario 1; 50% growth.

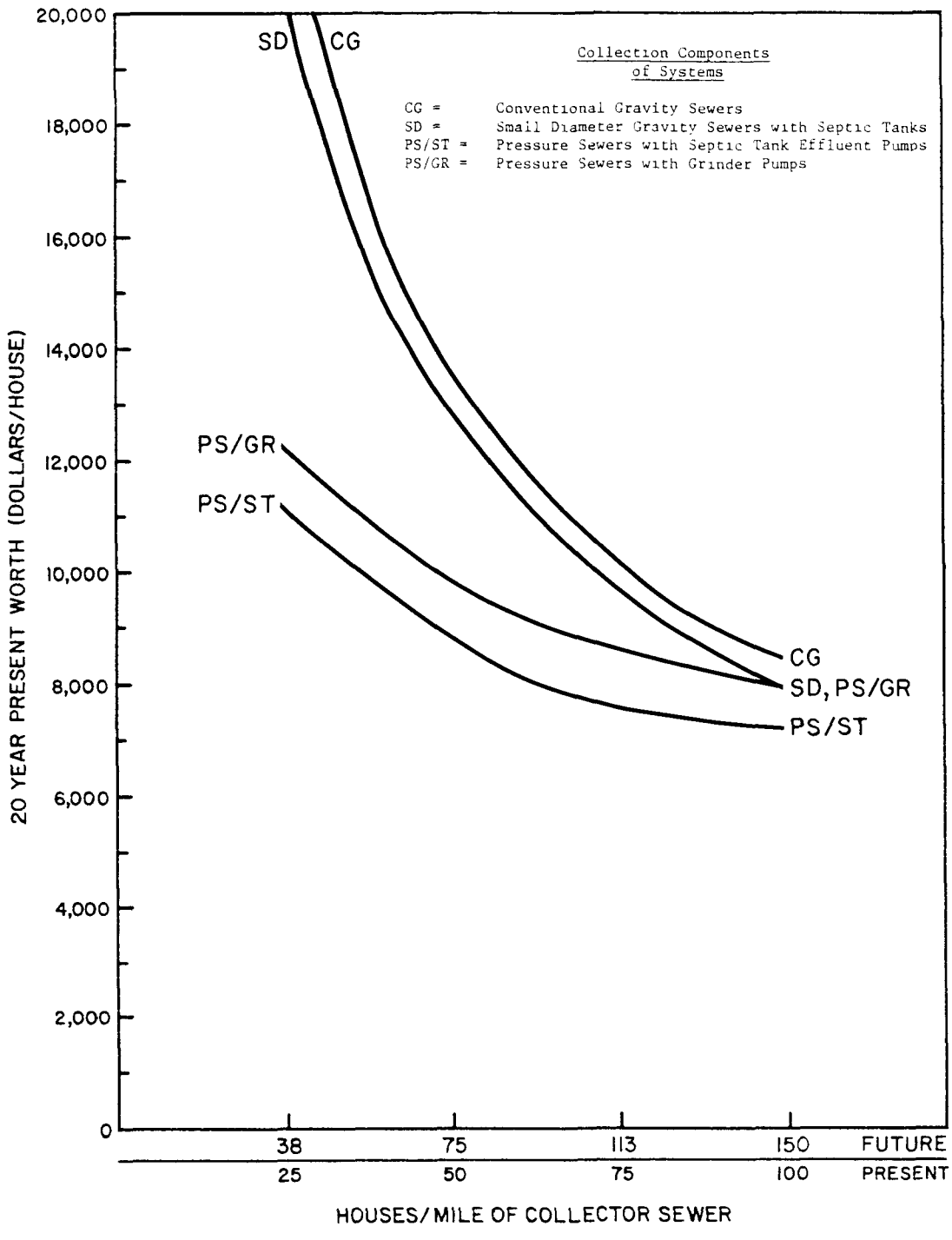

Figure IV-A-4-94. Cost comparison of collection, transmission, and cluster systems for Scenario 2; 50% growth.

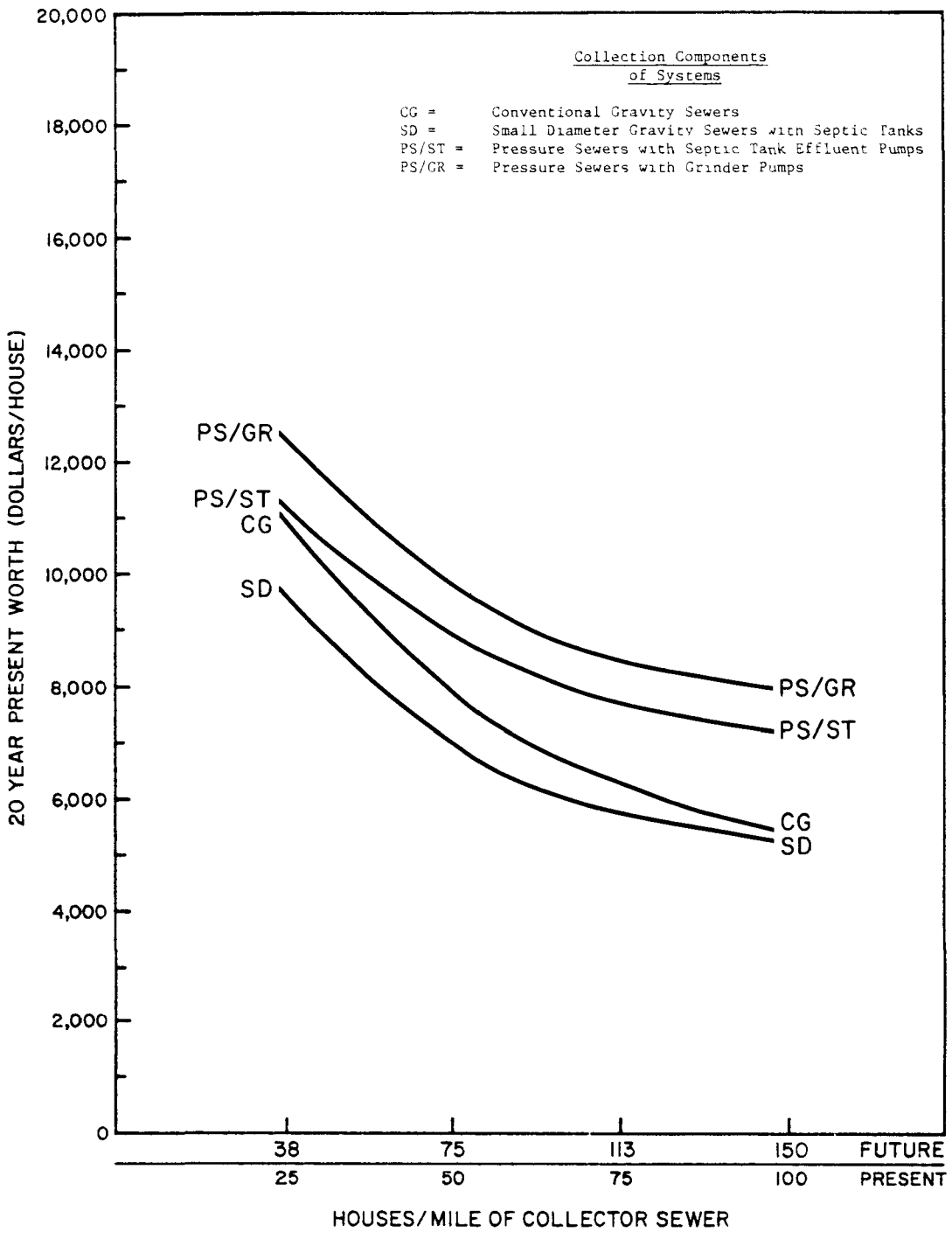

Figure IV-A-4-95. Cost comparison of collection, transmission, and cluster systems for Scenario 3; 50% growth.

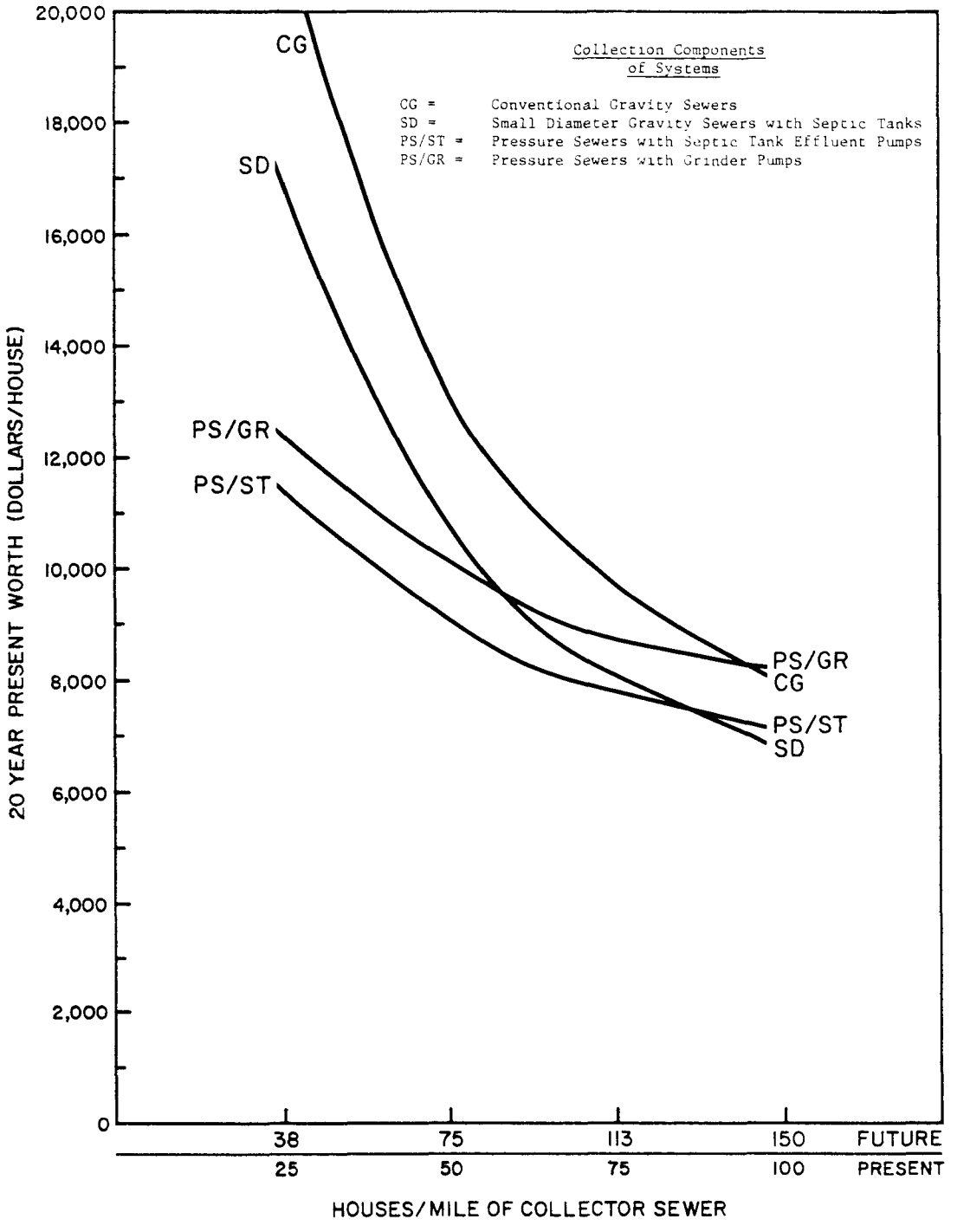

Figure IV-A-4-96. Cost comparison of collection, transmission, and cluster systems for Scenario 4; 50% growth.

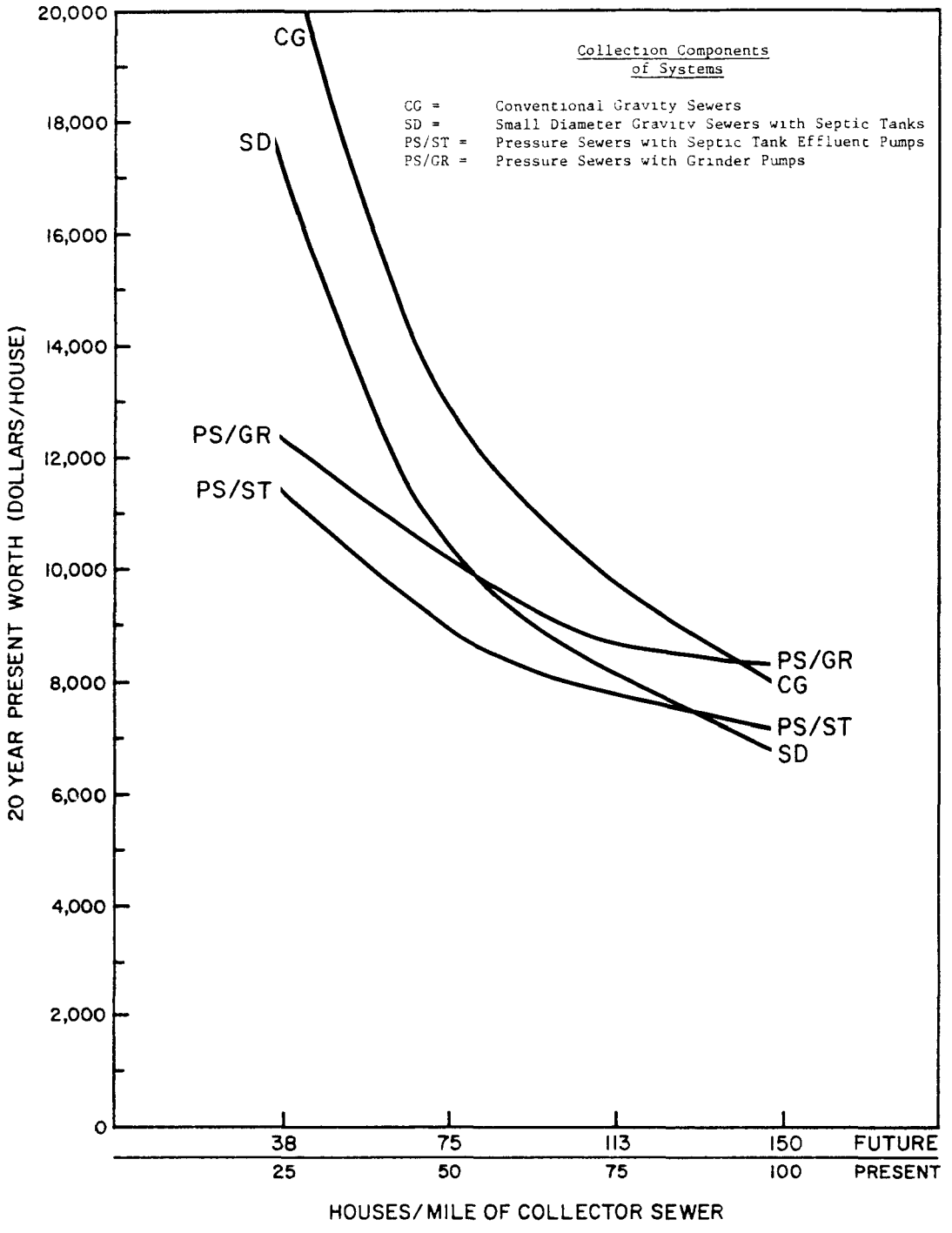

Figure IV-A-4-97. Cost comparison of collection, transmission, and cluster systems for Scenario 5: 50% growth.

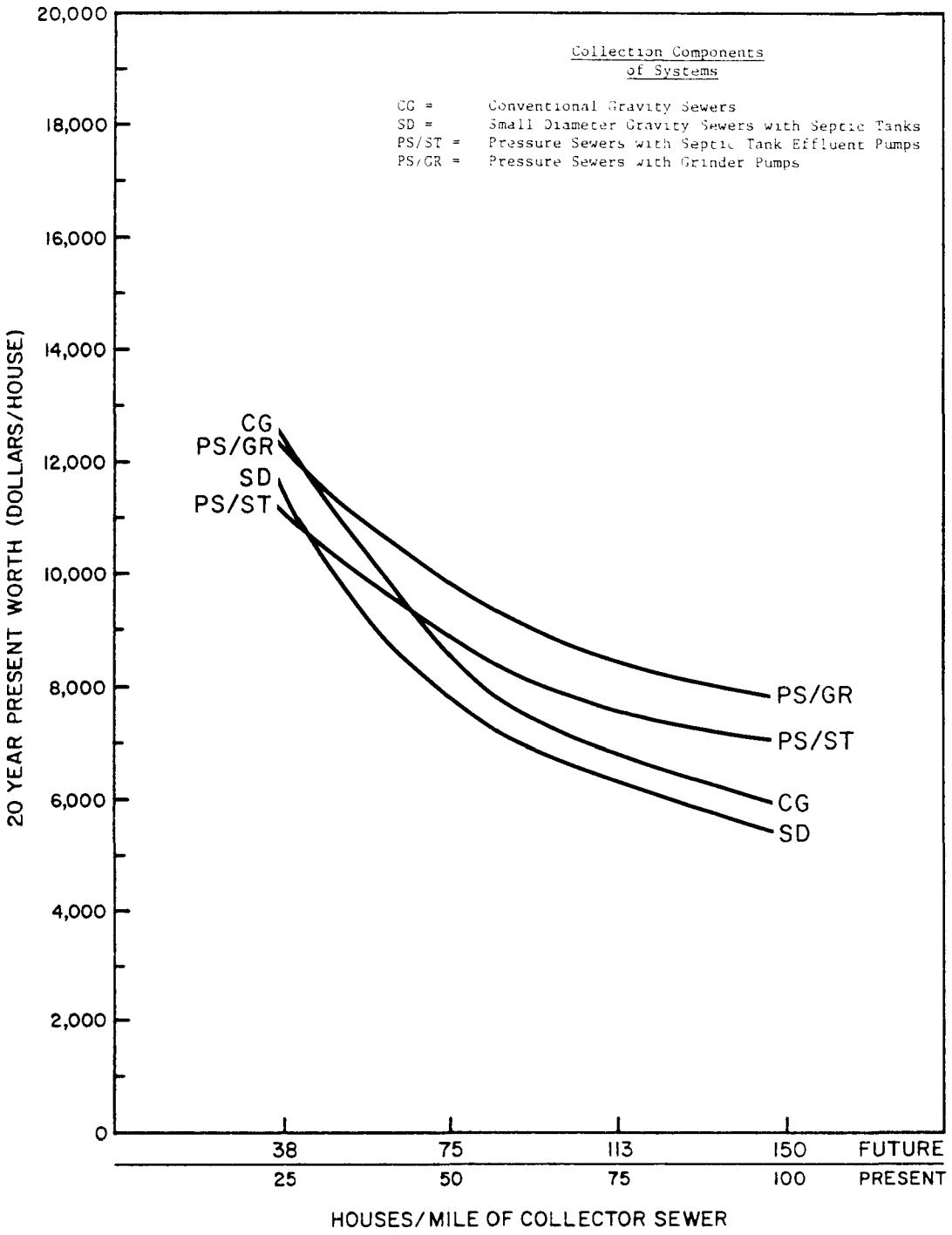

Figure IV-A-4-98. Cost comparison of collection, transmission, and cluster systems for Scenario 6; 50% growth.

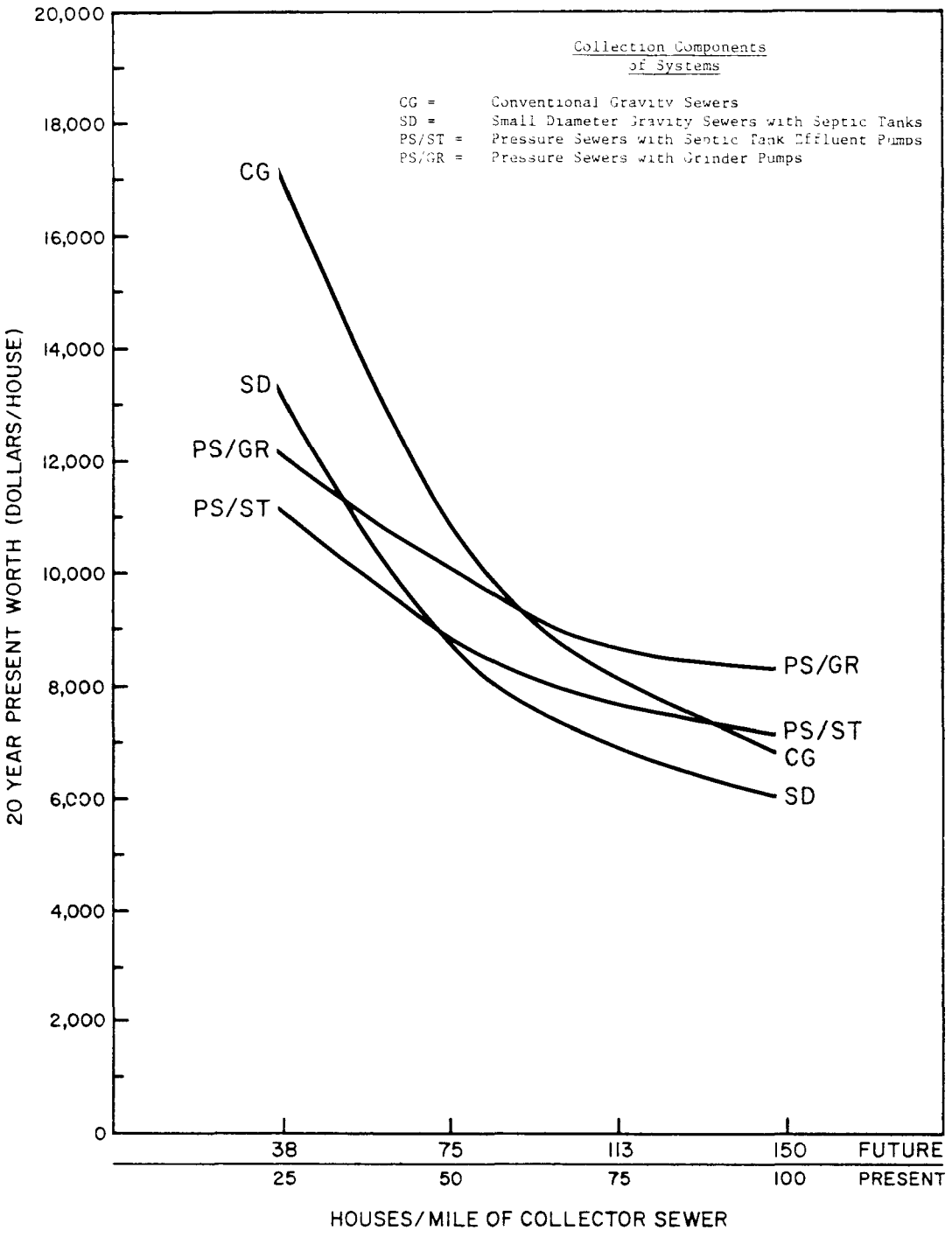

Figure IV-A-4-99. Cost comparison of collection, transmission, and cluster systems for Scenario 7; 50% growth.

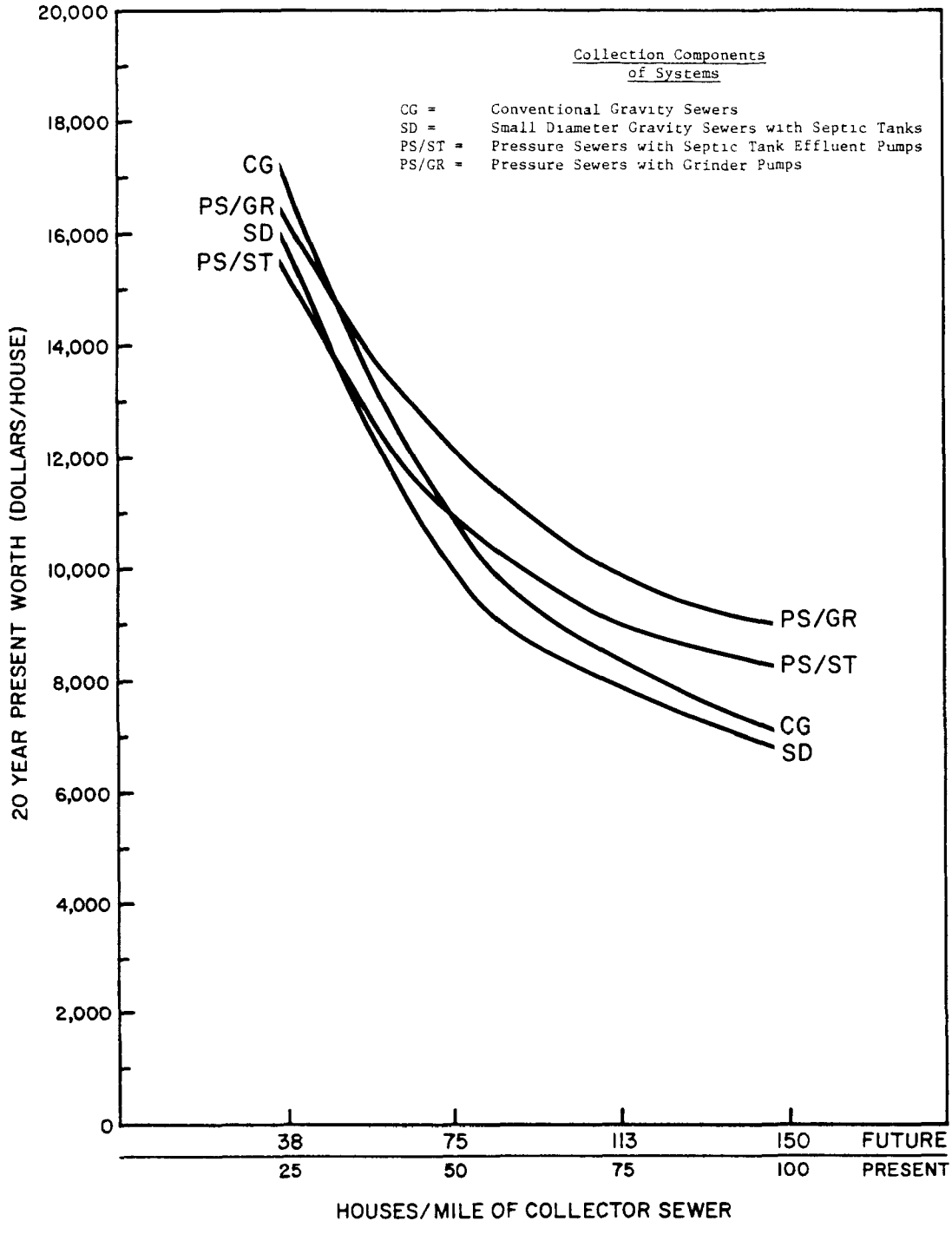

Figure IV-A-4-100. Cost comparison of collection, transmission, and cluster systems for Scenario 8; 50% growth.

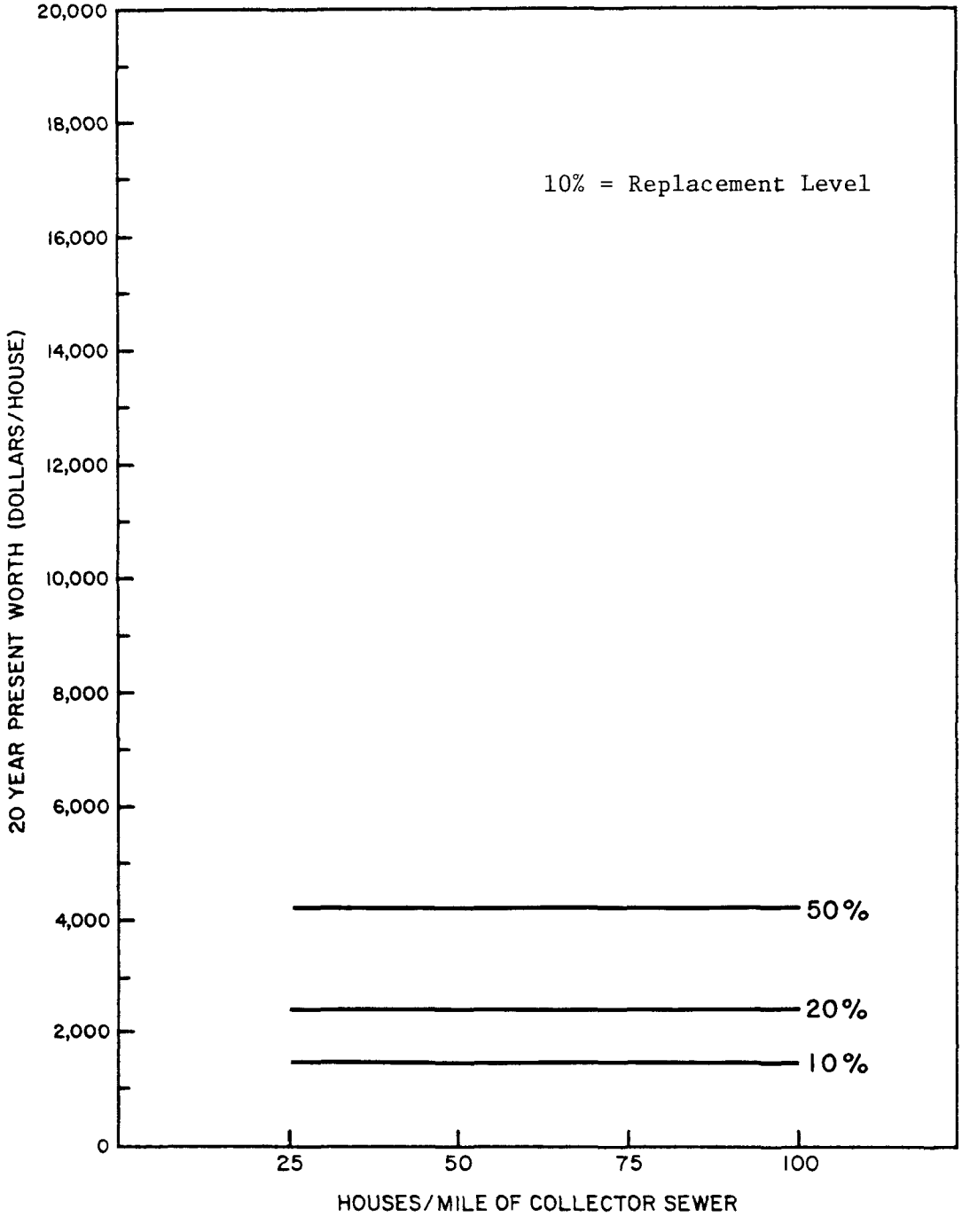

Figure IV-A-4-101. Cost comparison of on-site systems at 3 replacement levels for Scenario 1; 0% growth.

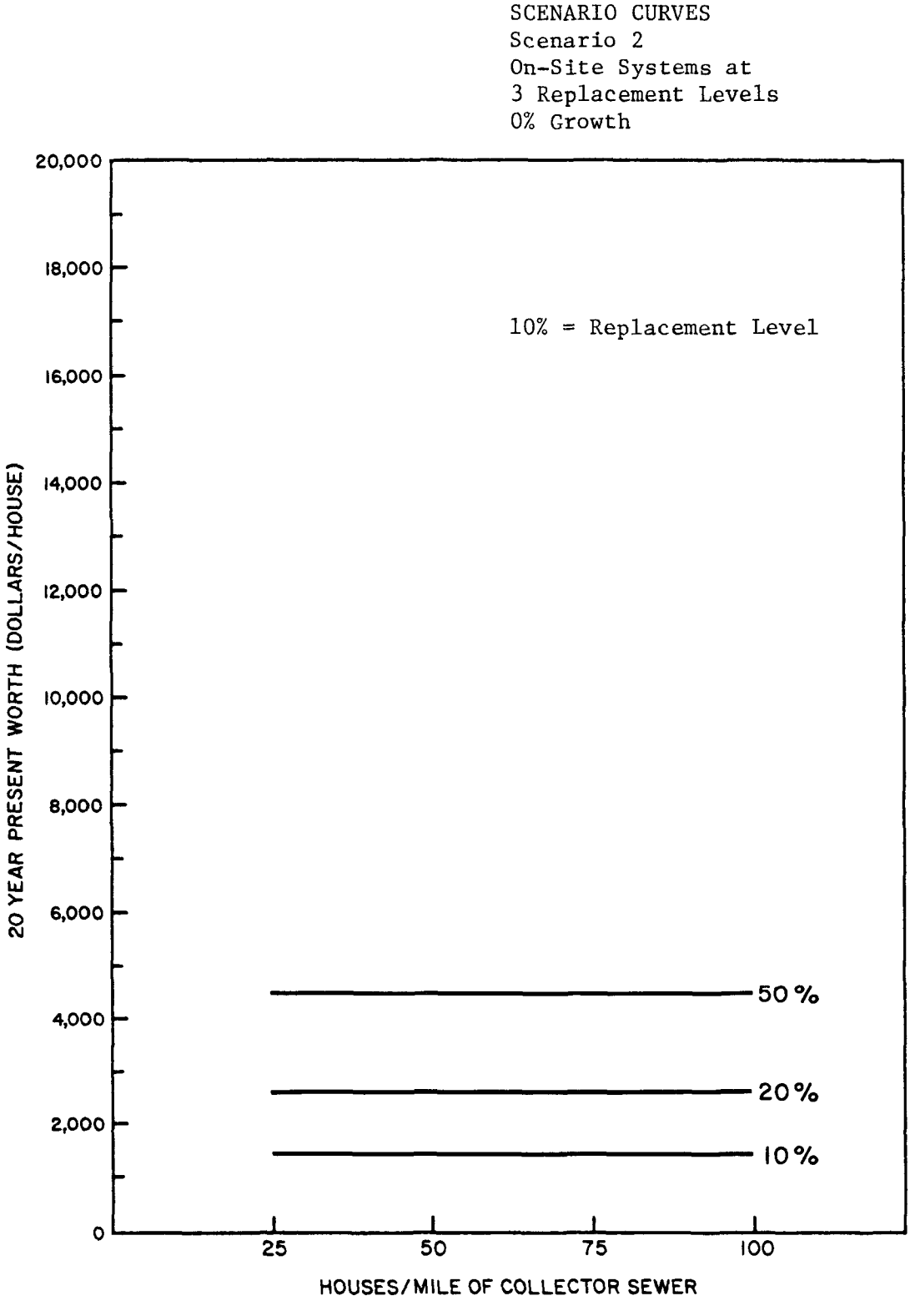

Figure IV-A-4-102. Cost comparison of on-site systems at 3 replacement levels for Scenario 2; 0% growth.

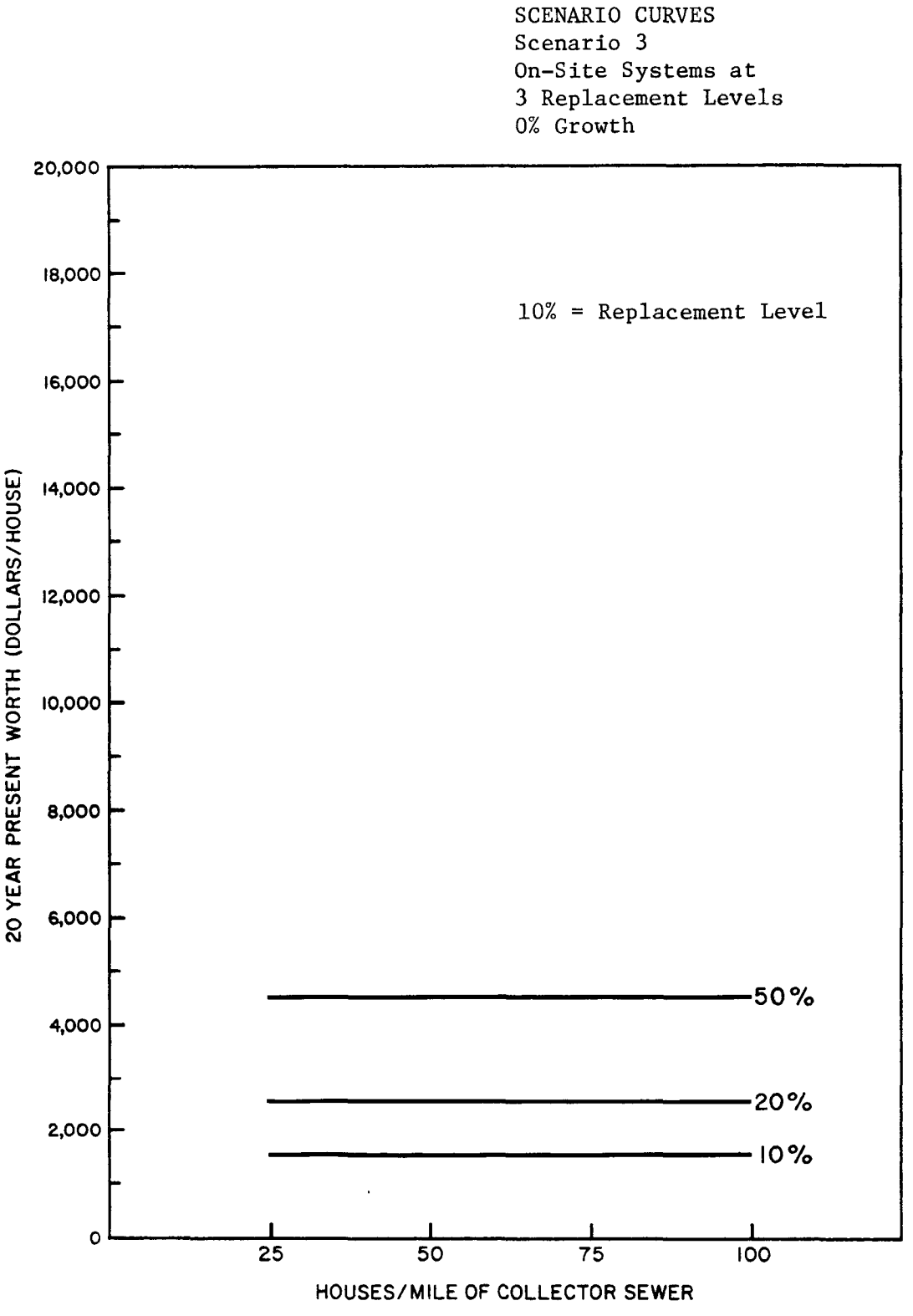

Figure IV-A-103. Cost comparison of on-site systems at 3 replacement levels for Scenario 3; 0% growth.

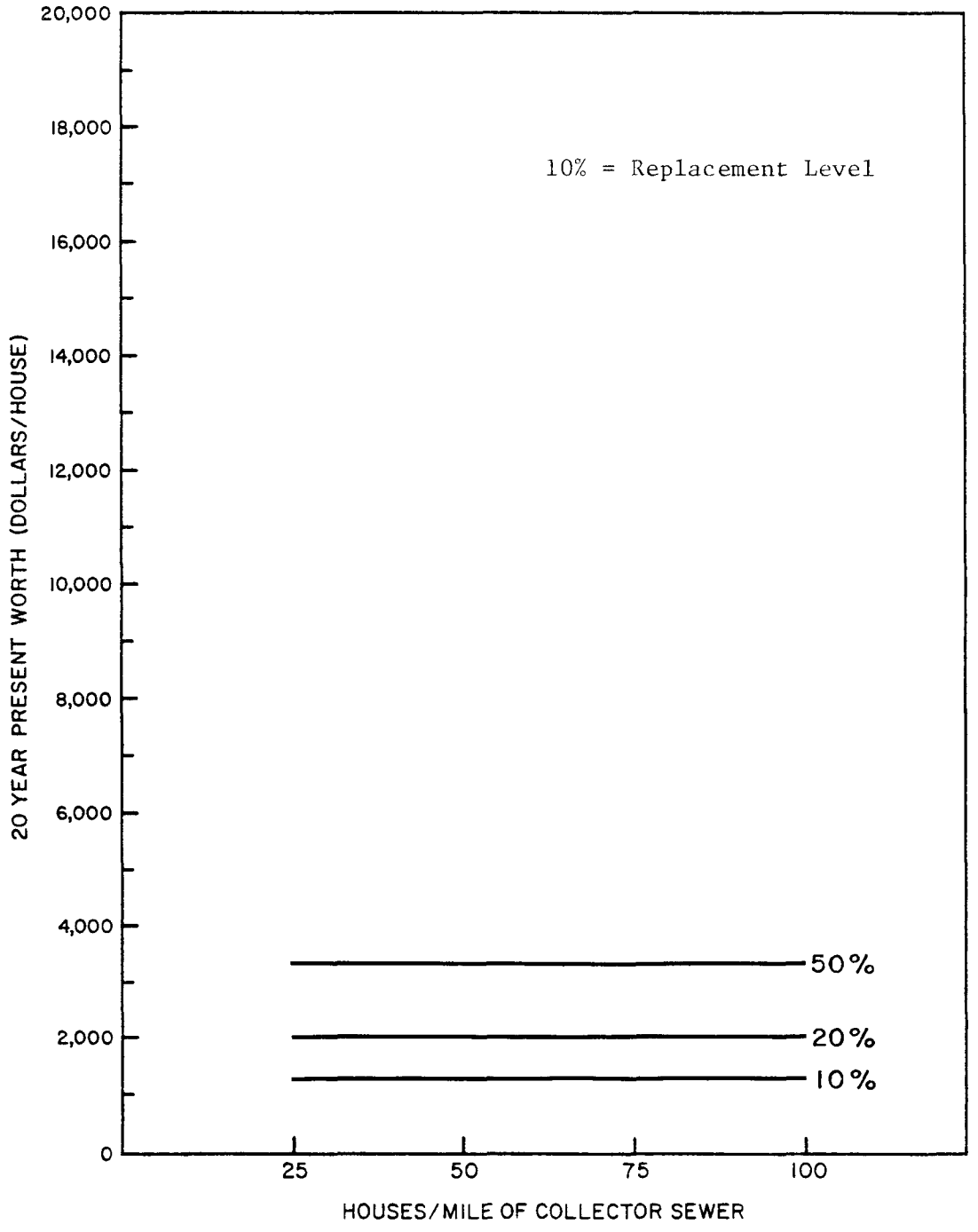

Figure IV-A-4-104. Cost comparison of on-site systems at 3 replacement levels for Scenario 4; 0% growth.

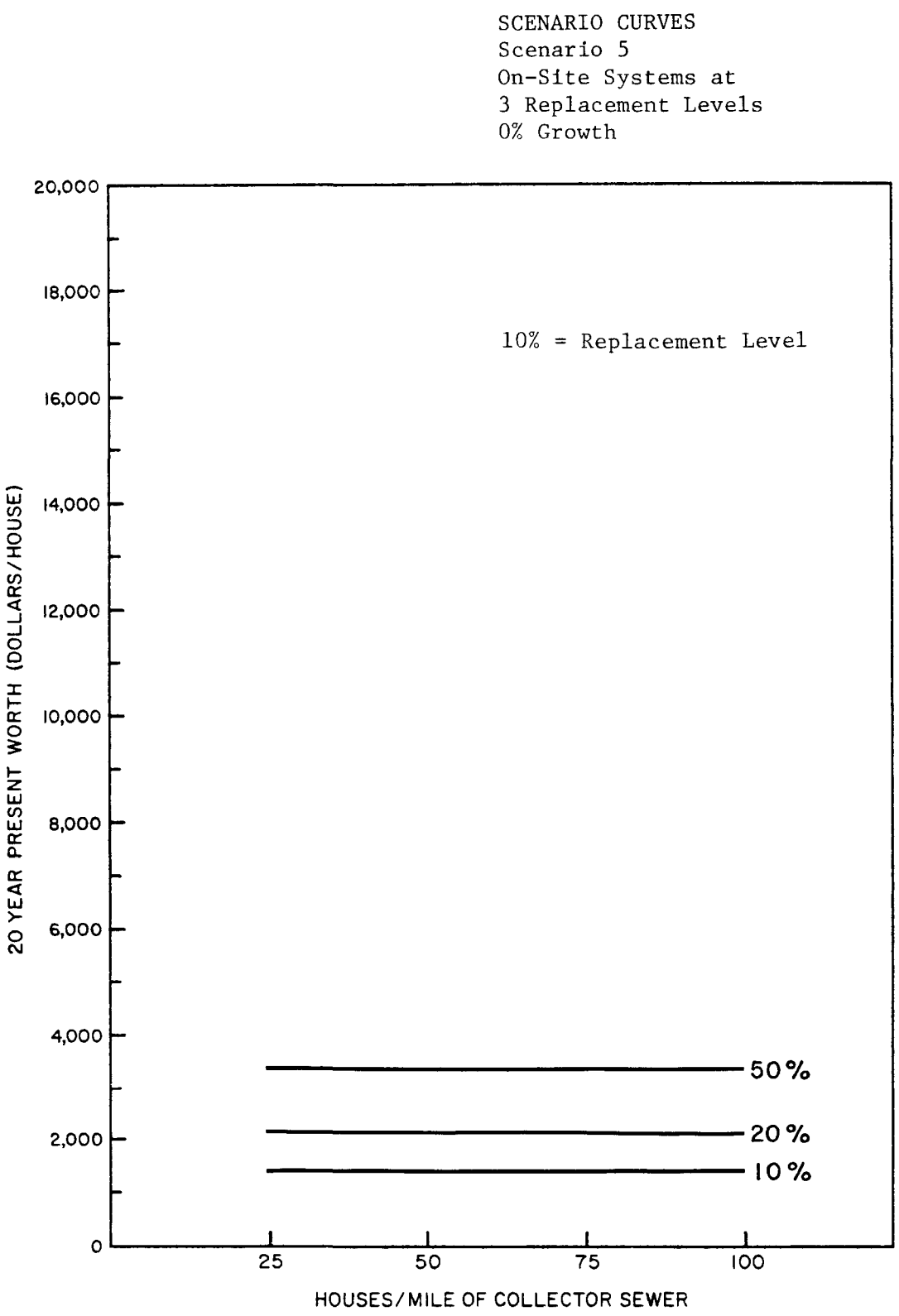

Figure IV-A-4-105. Cost comparison of on-site systems at 3 replacement levels for Scenario 5; 0% growth.

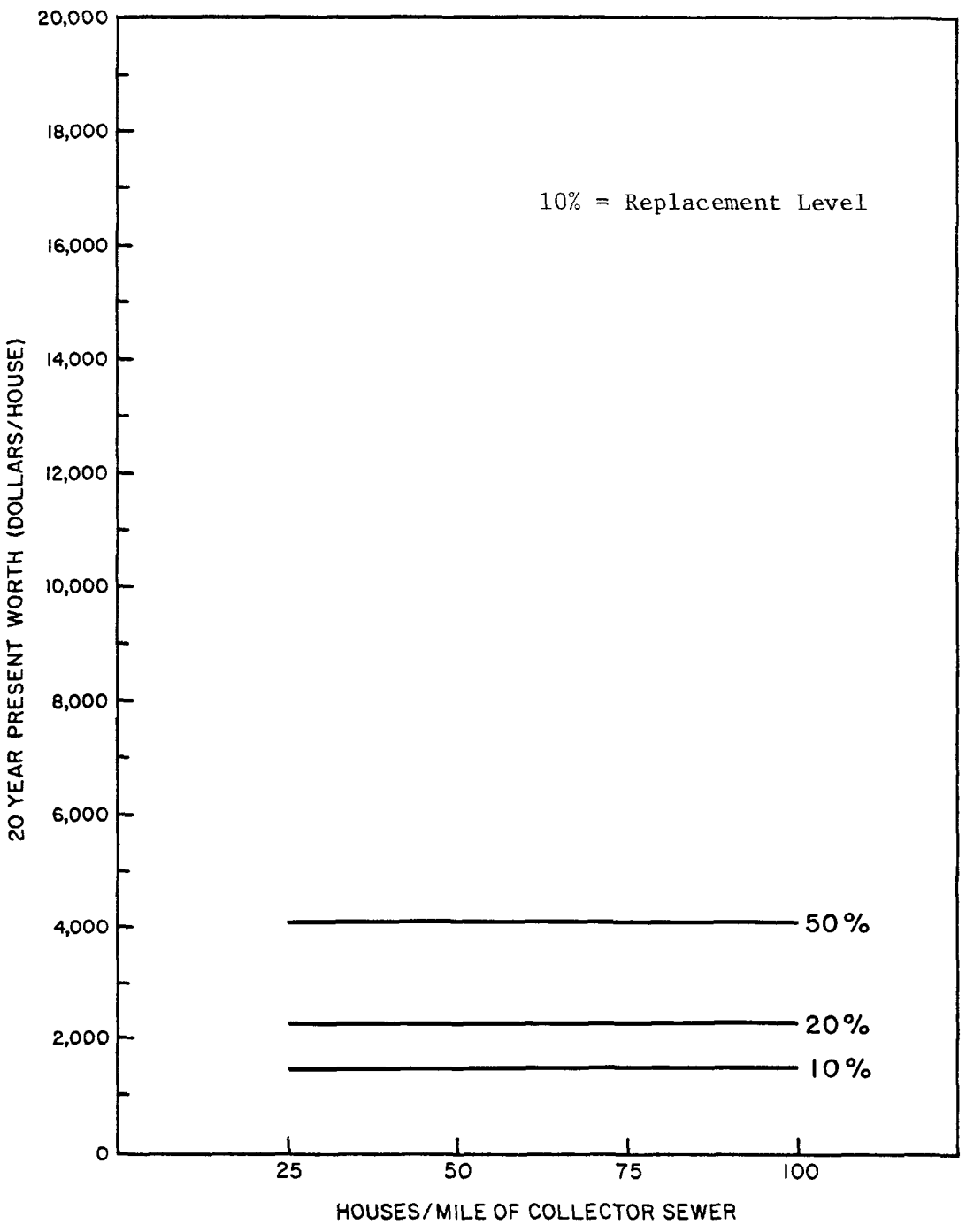

Figure IV-A-4-106. Cost comparison of on-site systems at 3 replacement levels for Scenario 6; 0% growth.

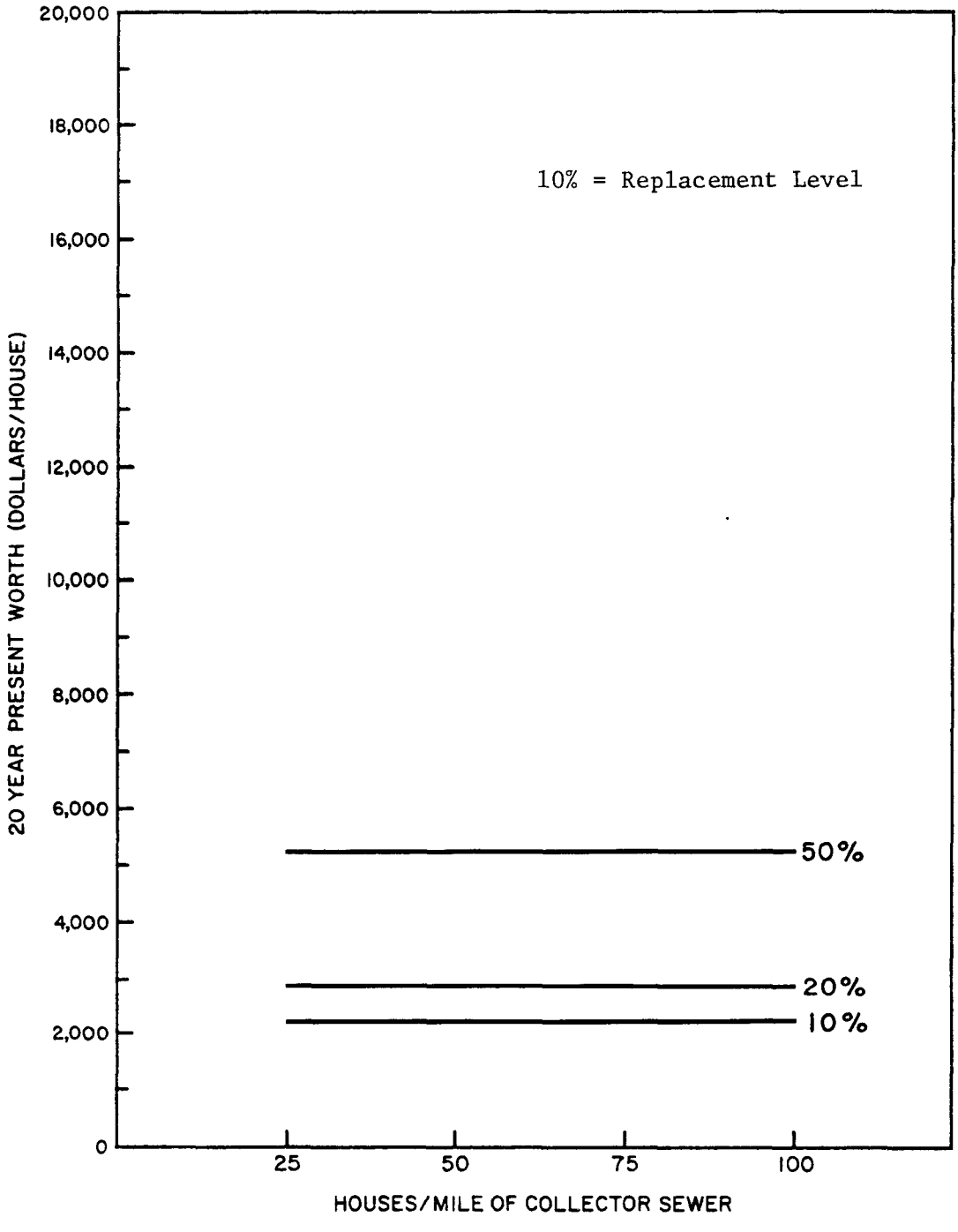

Figure IV-A-4-107. Cost comparison of on-site systems at 3 replacement levels for Scenario 7; 0% Growth

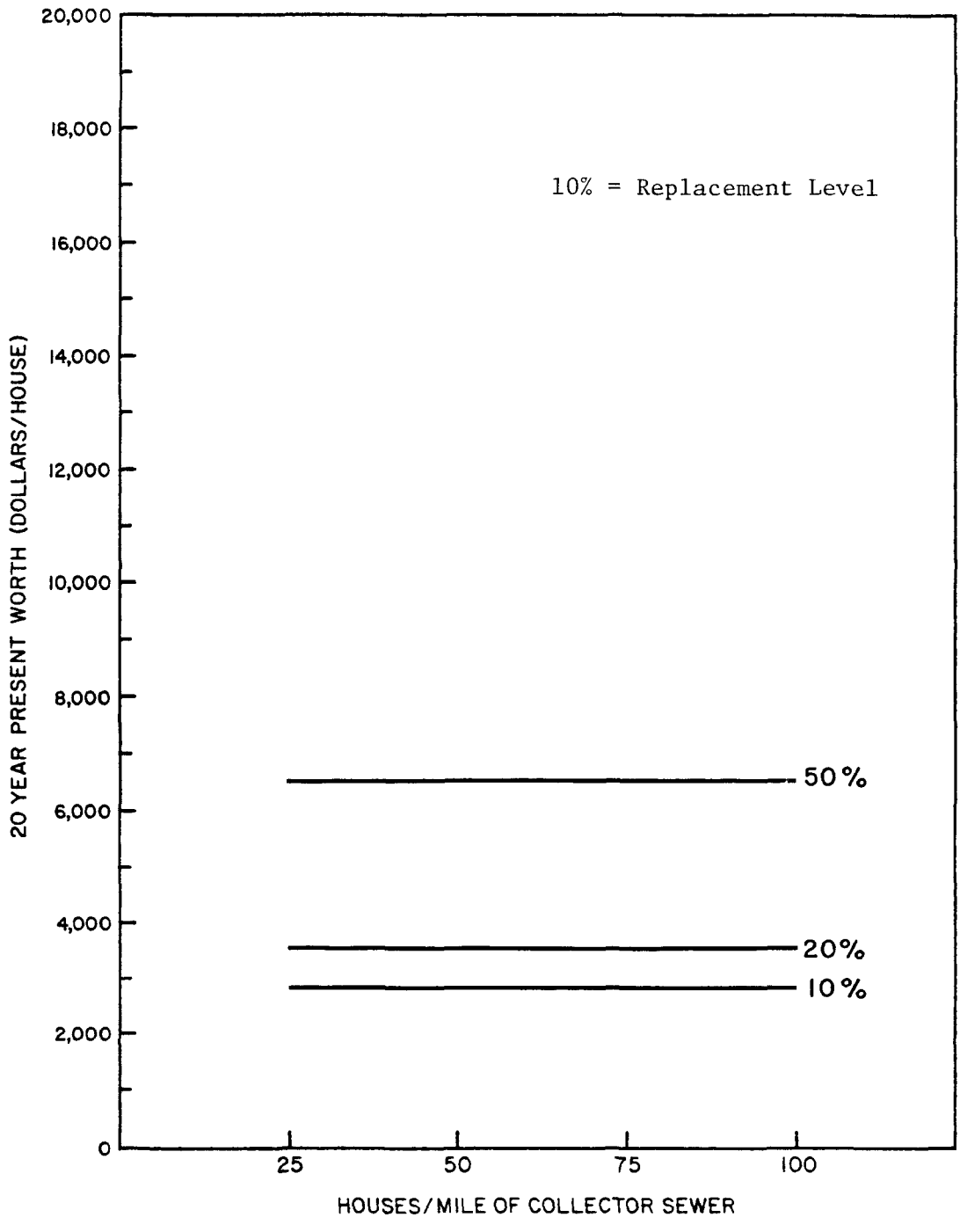

Figure IV-A-4-108. Cost comparison of on-site systems at 3 replacement levels for Scenario 8; 0% growth.

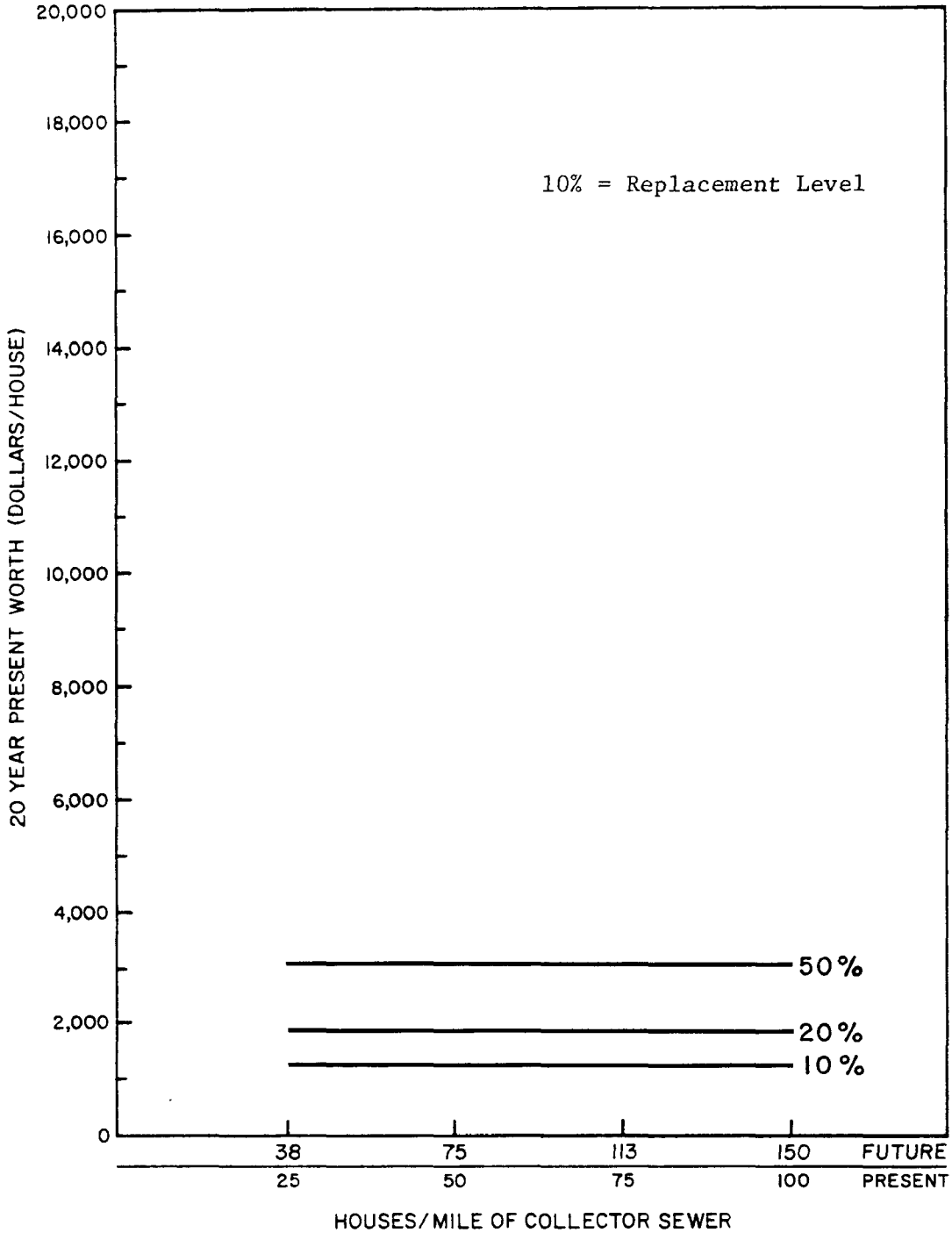

Figure IV-A-4-109. Cost comparison of on-site systems at 3 replacement levels for Scenario 1; 50% growth.

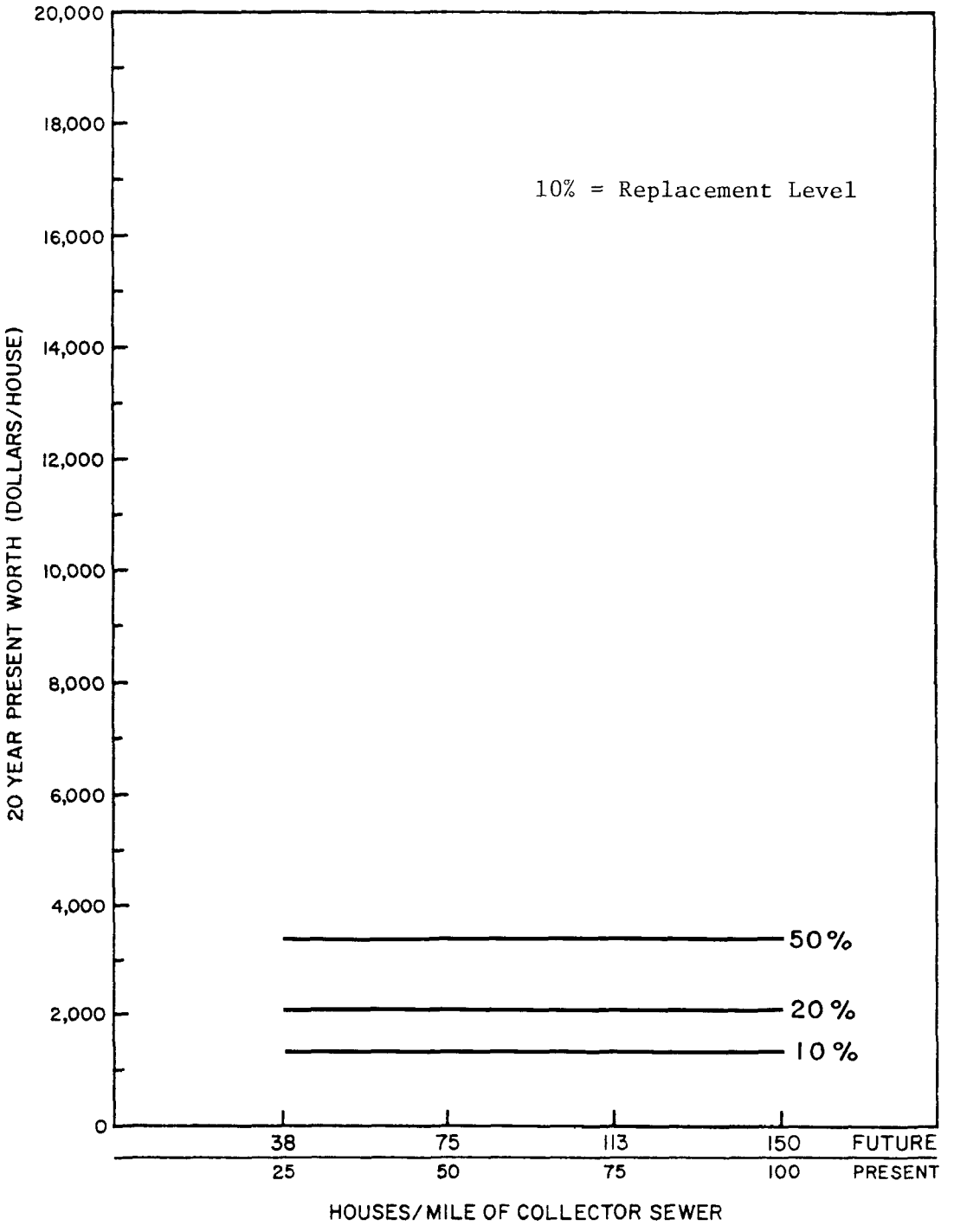

Figure IV-A-4-110. Cost comparison of on-site systems at 3 replacement levels for Scenario 2; 50% growth.

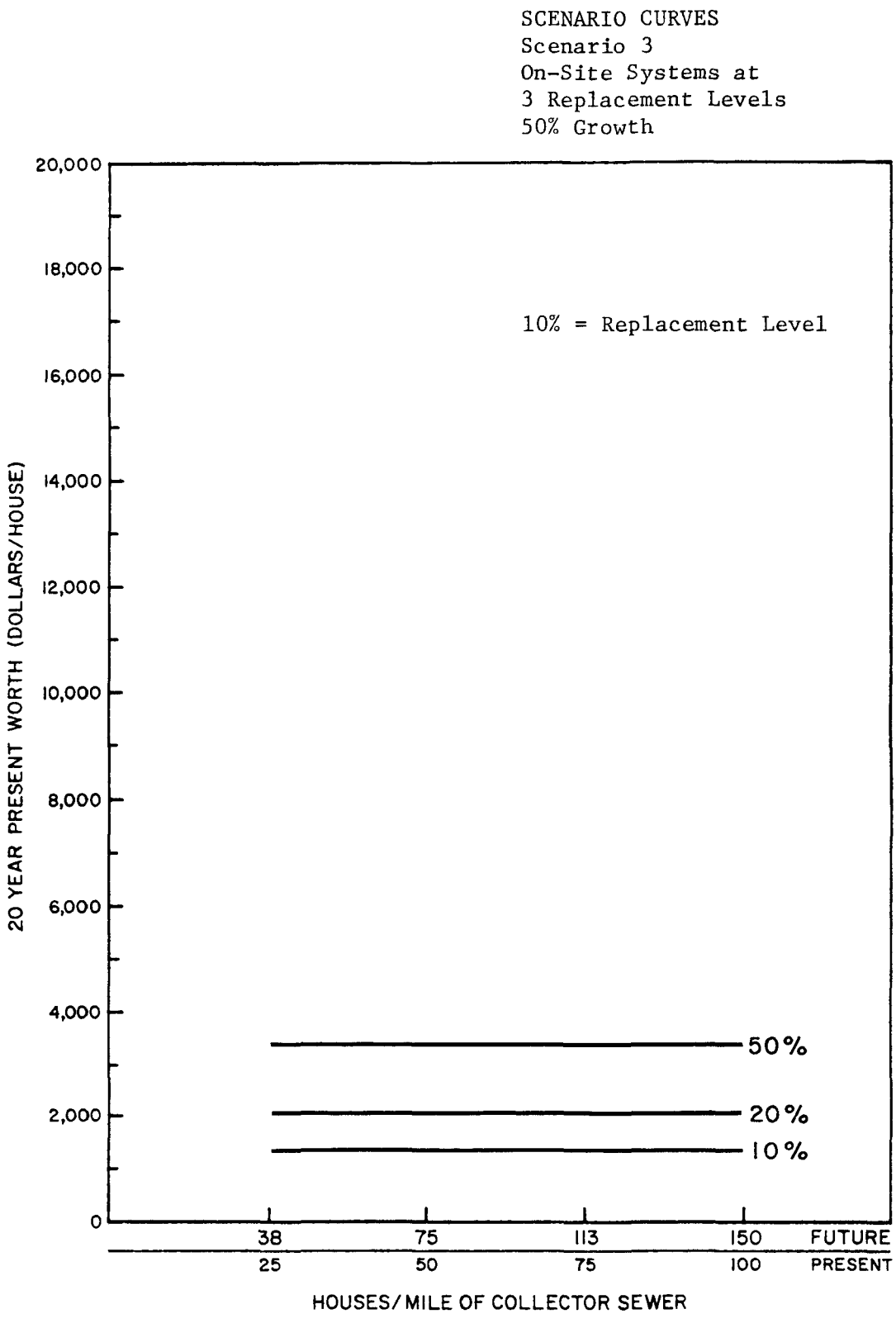

Figure IV-A-4-111. Cost comparison of on-site systems at 3 replacement levels for Scenario 3; 50% growth.

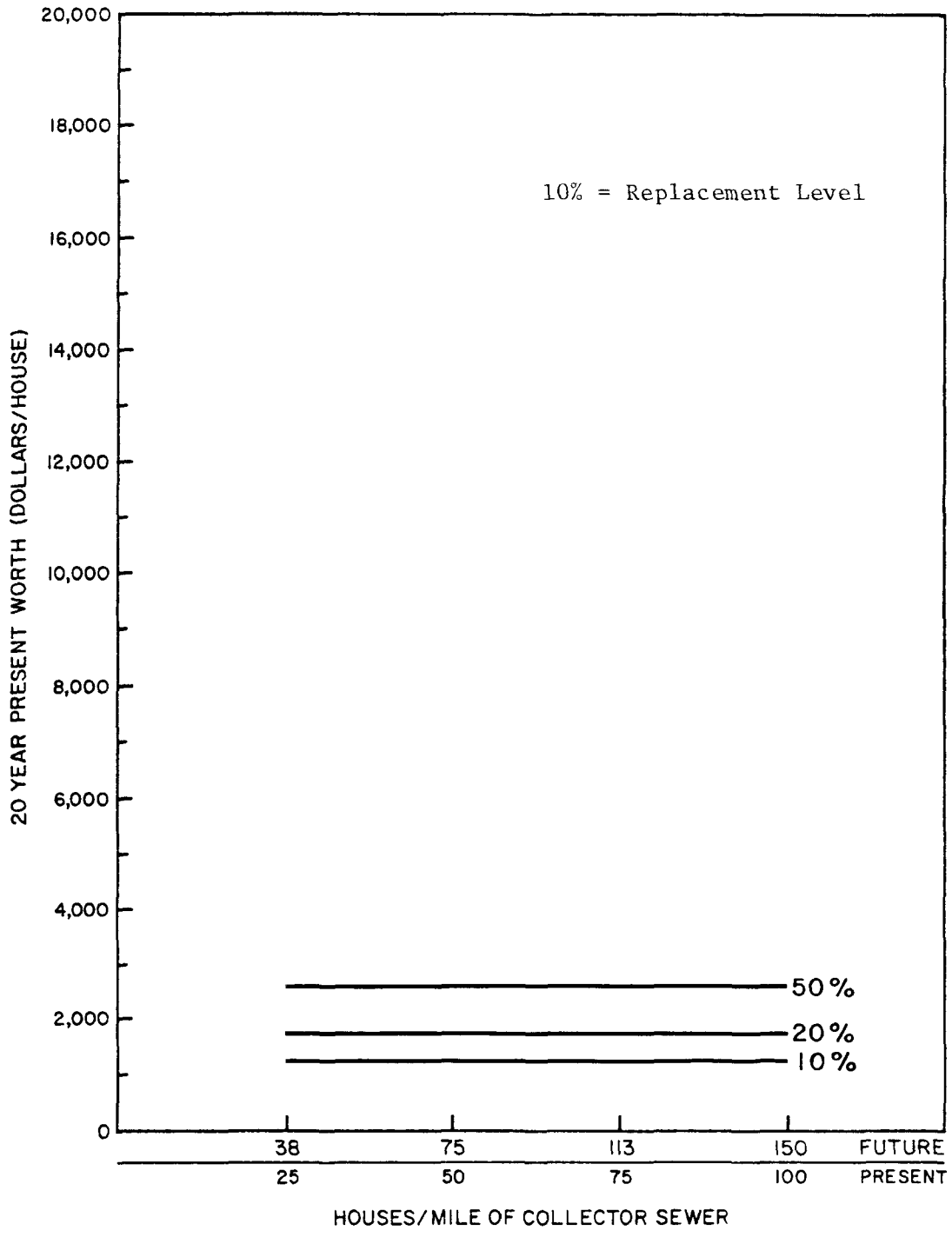

Figure IV-A-4-112. Cost comparison of on-site systems at 3 replacement levels for Scenario 4; 50% growth.

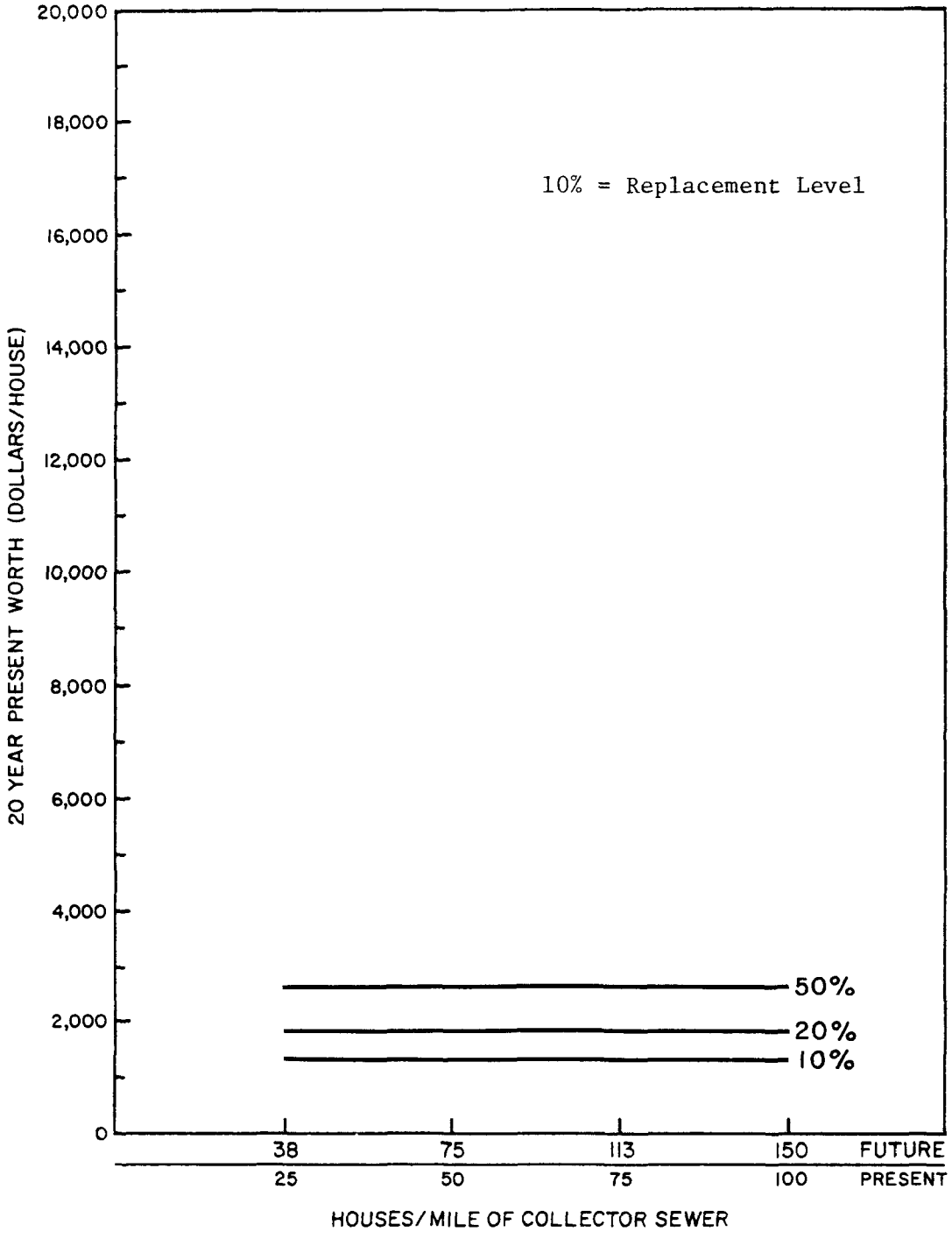

Figure IV-A-4-113. Cost comparison of on-site systems at 3 replacement levels for Scenario 5; 50% growth.

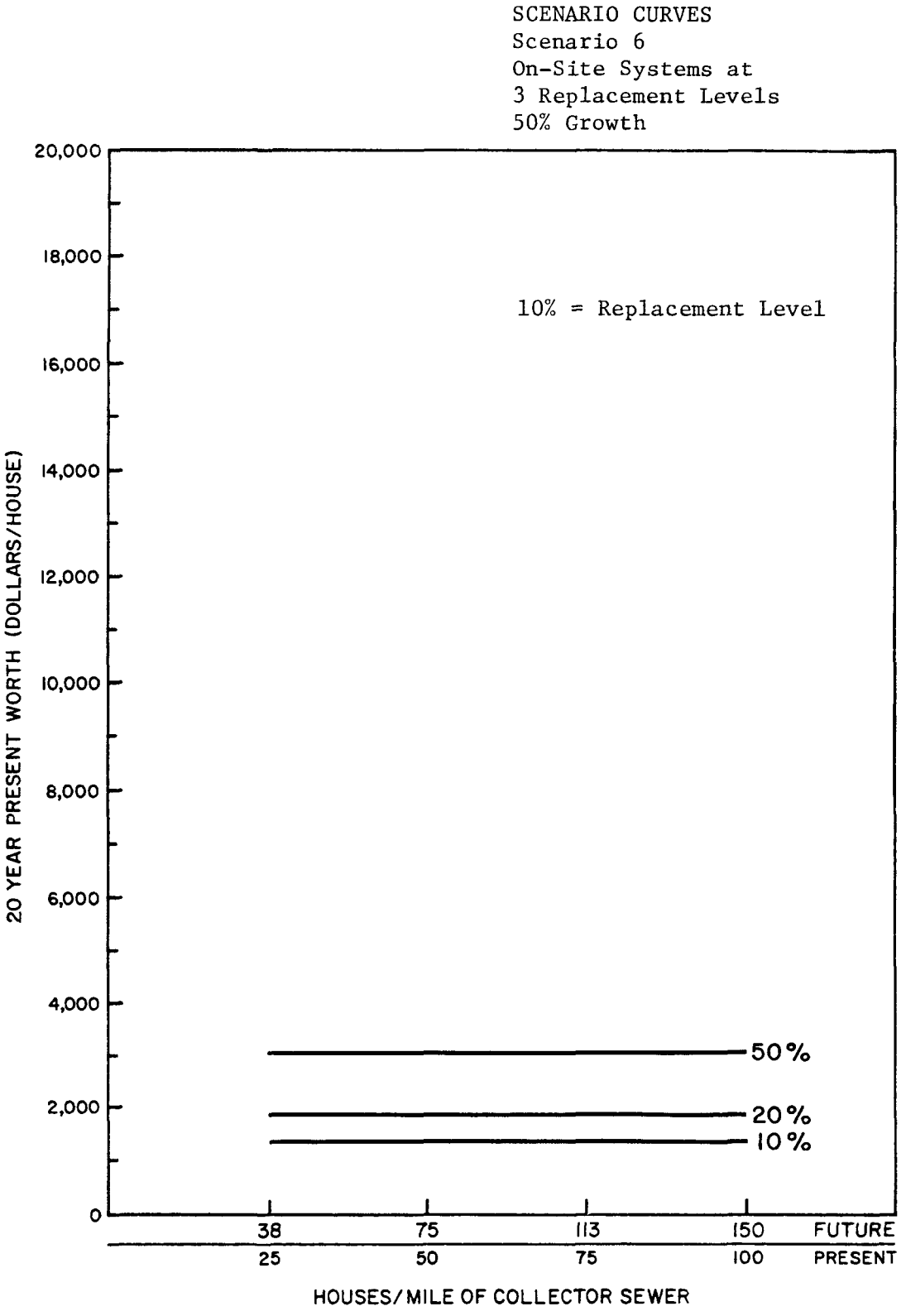

Figure IV-A-4-114. Cost comparison of on-site systems at 3 replacement levels for Scenario 6; 50% growth.

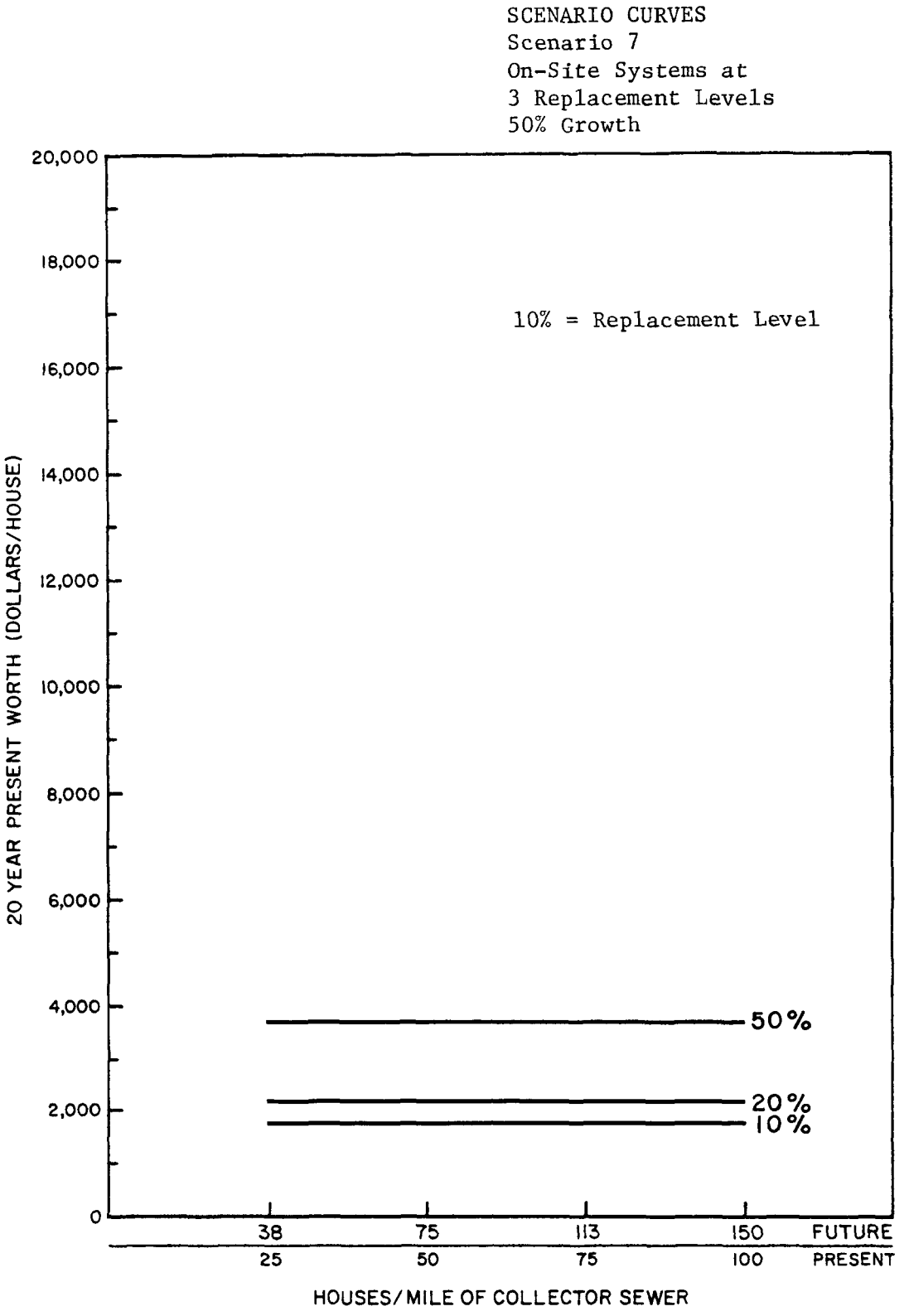

Figure IV-A-4-115. Cost comparison of on-site systems at 3 replacement levels for Scenario 7; 50% growth.

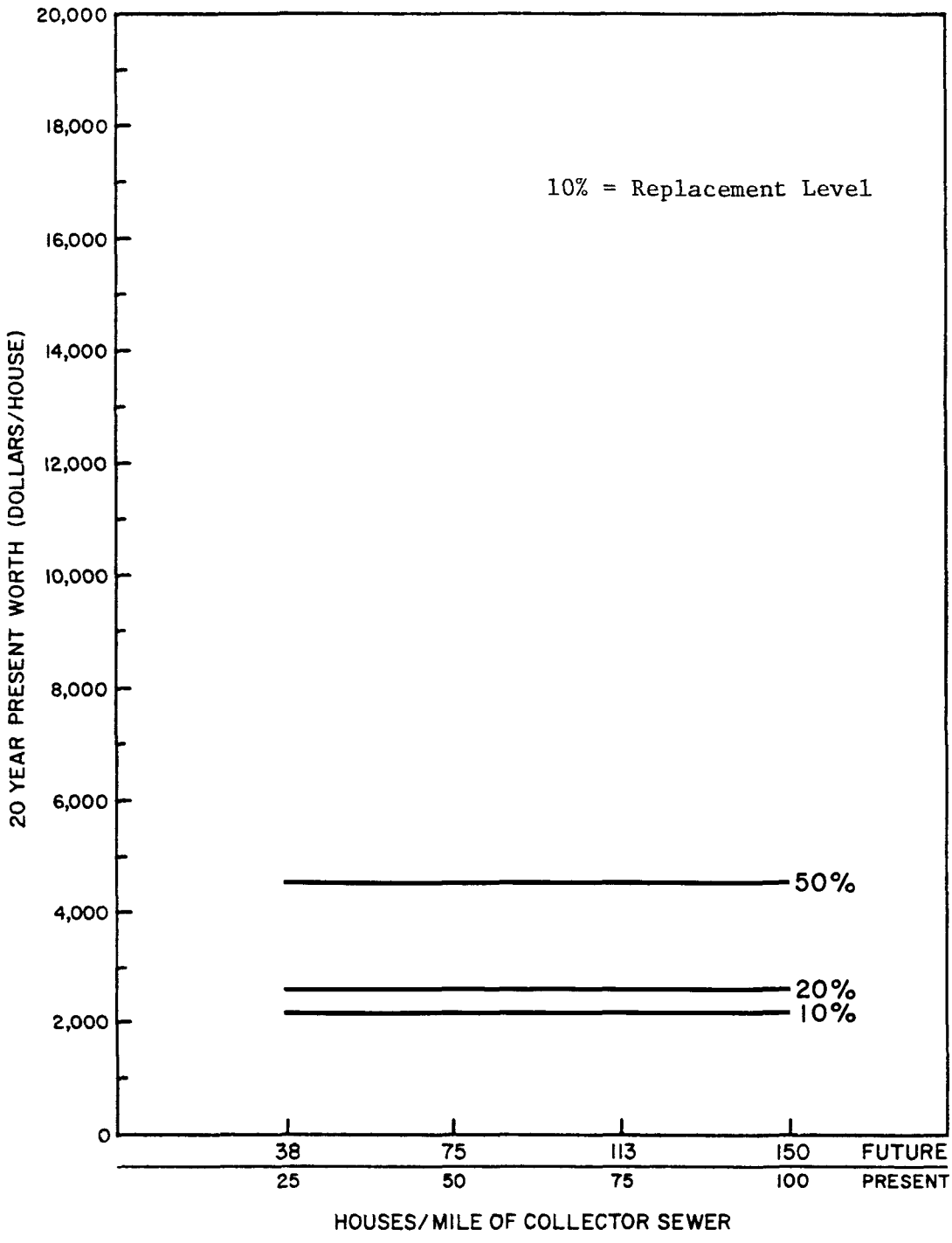

Figure IV-A-4-116. Cost comparison of on-site systems at 3 replacement levels for Scenario 8; 50% growth.

COST-EFFECTIVENESS CURVES
Scenario 1
0% Growth

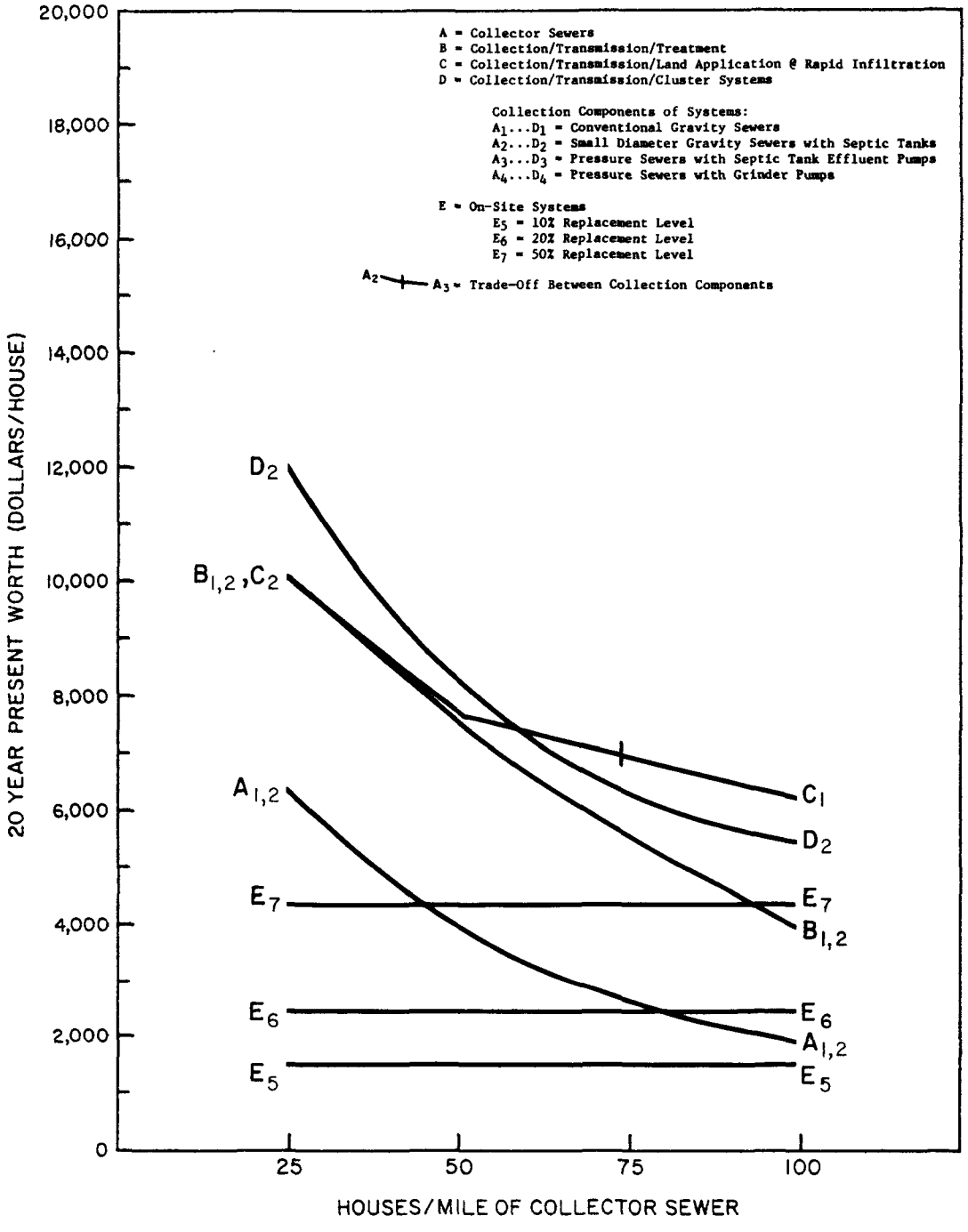

Figure IV-A-4-117. Cost-effectiveness curves for on-site small scale and centralized treatment alternatives for Scenario 1; 0% growth.

COST-EFFECTIVENESS CURVES
Scenario 2
0% Growth

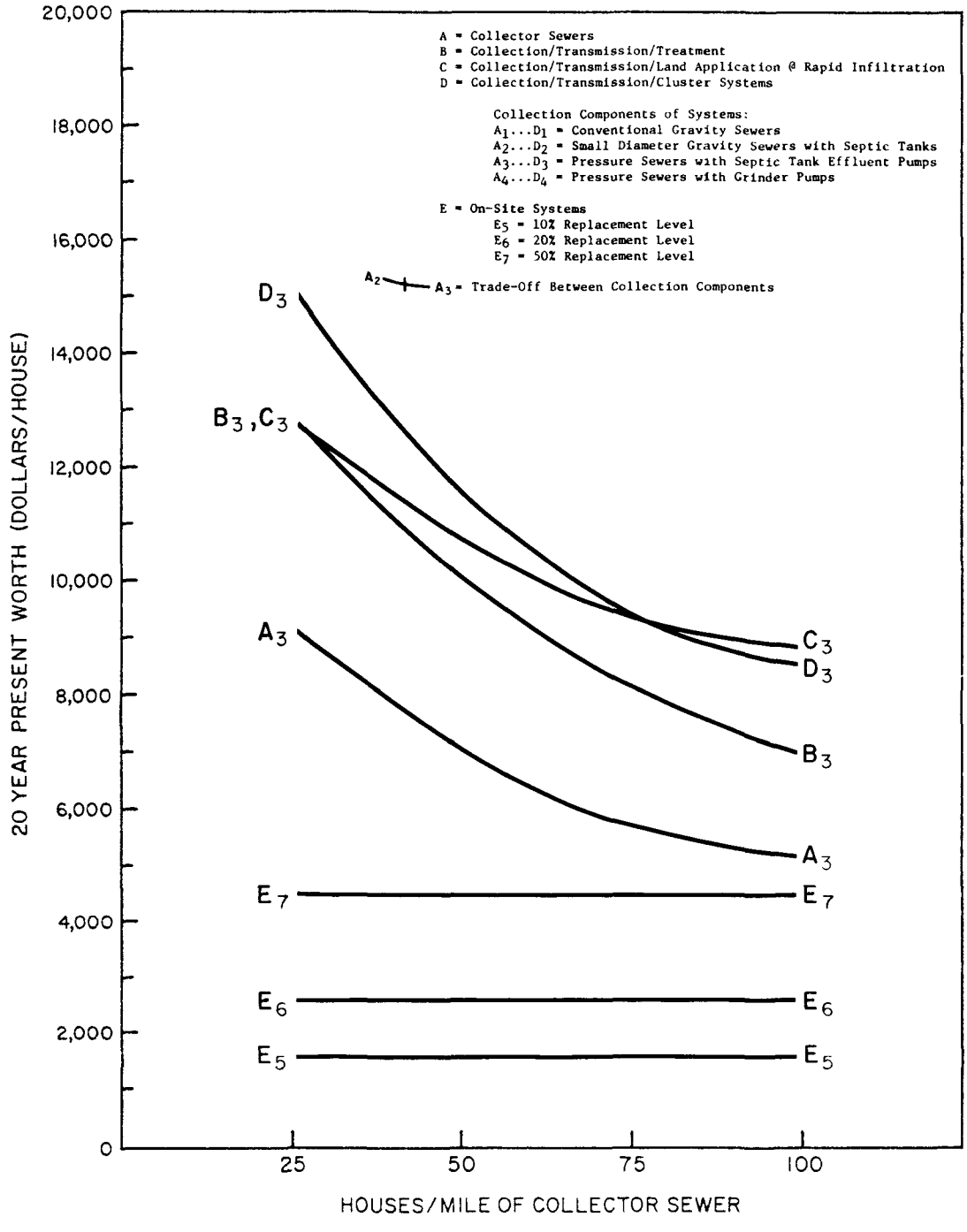

Figure IV-A-4-118. Cost-effectiveness curves for on-site small scale and centralized treatment alternatives for Scenario 2; 0% growth.

COST-EFFECTIVENESS CURVES
Scenario 3
0% Growth

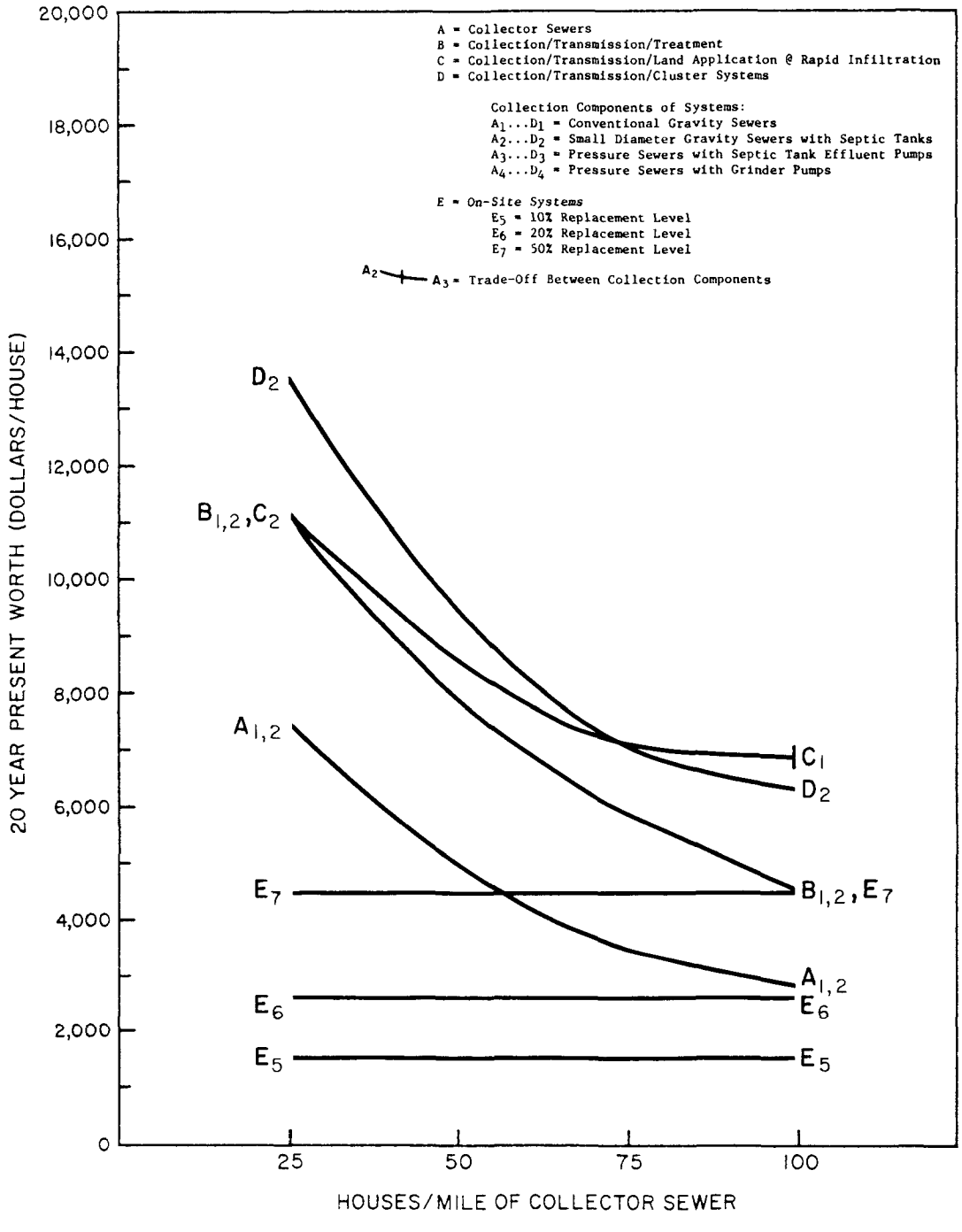

Figure IV-A-4-119. Cost-effectiveness curves for on-site small scale and centralized treatment alternatives for Scenario 3; 0% growth.

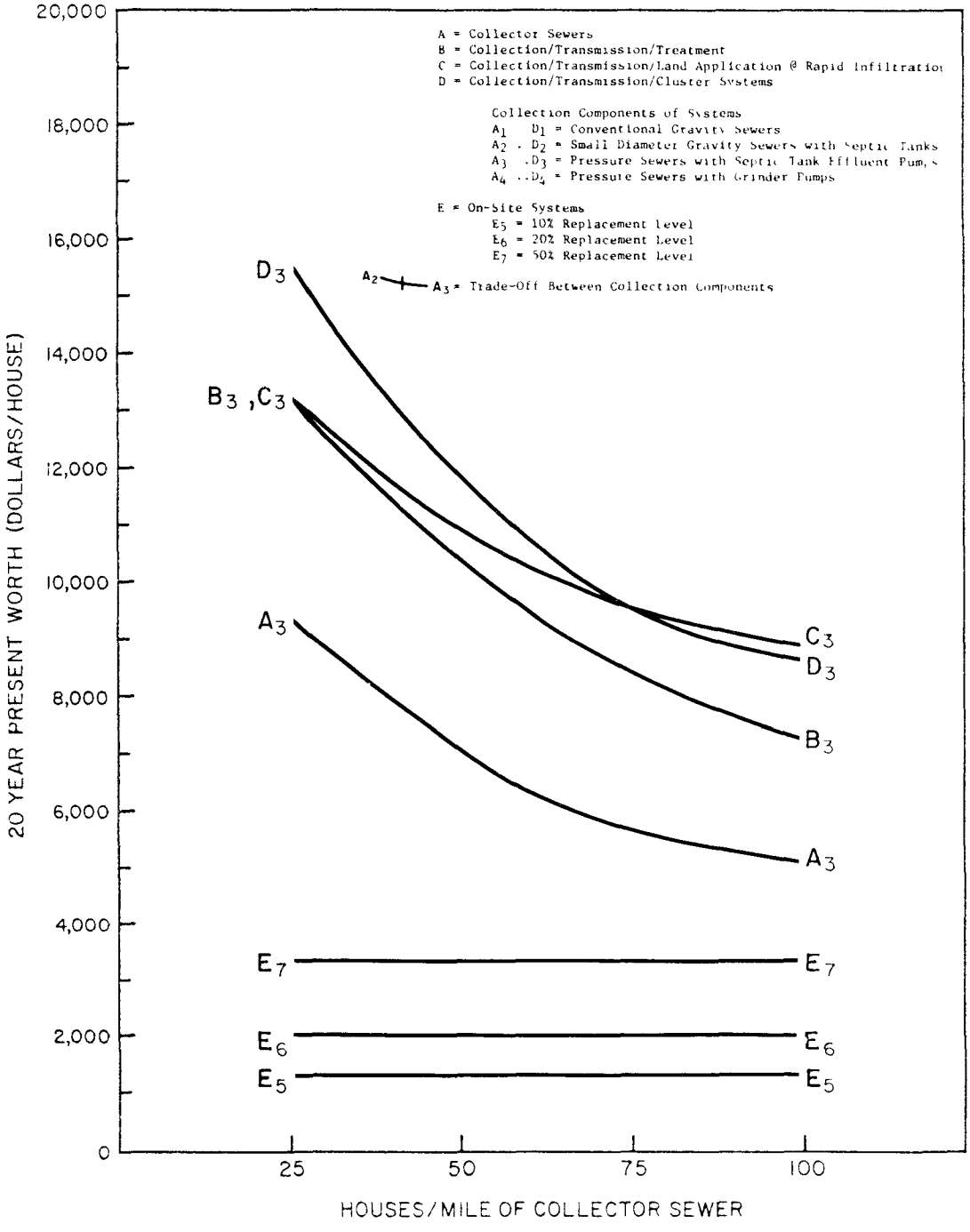

Figure IV-A-4-120. Cost-effectiveness curves for on-site small scale and centralized treatment alternatives for Scenario 4; 0% growth.

COST-EFFECTIVENESS CURVES
Scenario 5
0% Growth

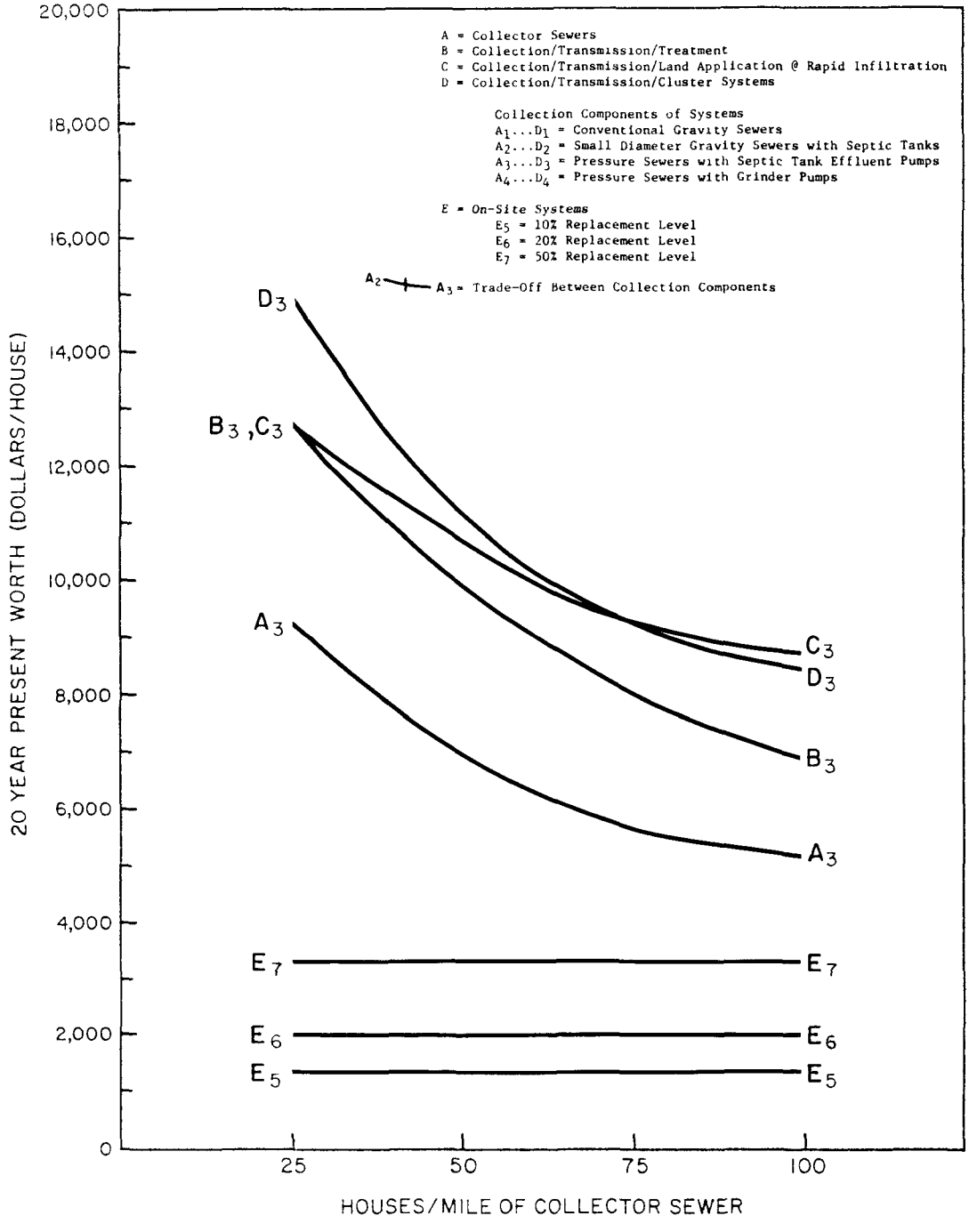

Figure IV-A-4-121. Cost-effectiveness curves for on-site small scale and centralized treatment alternatives for Scenario 5; 0% growth.

COST-EFFECTIVENESS CURVES
Scenario 6
0% Growth

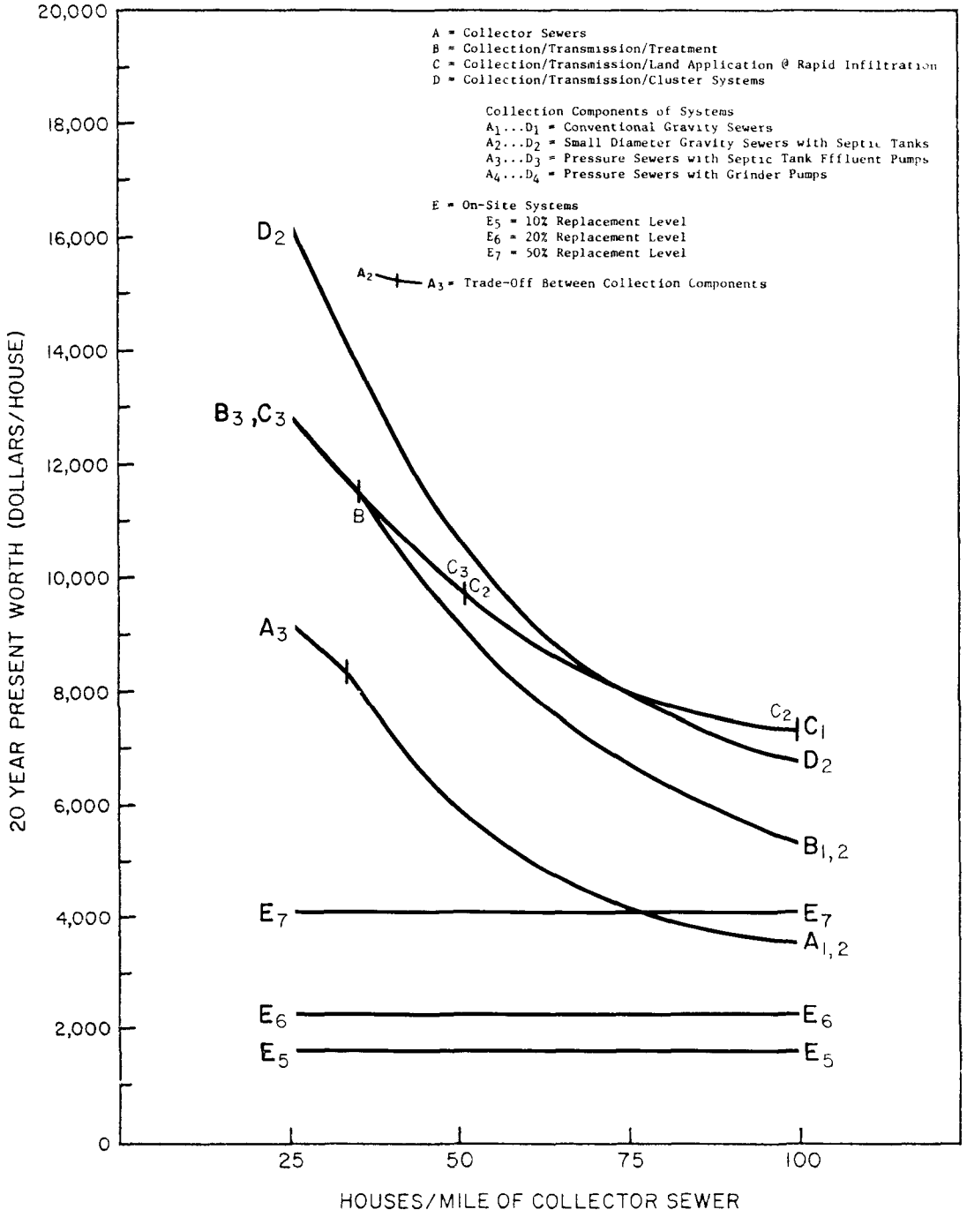

Figure IV-A-4-122. Cost-effectiveness curves for on-site small scale and centralized treatment alternatives for Scenario 6; 0% growth.

COST-EFFECTIVENESS CURVES
Scenario 7
0% Growth

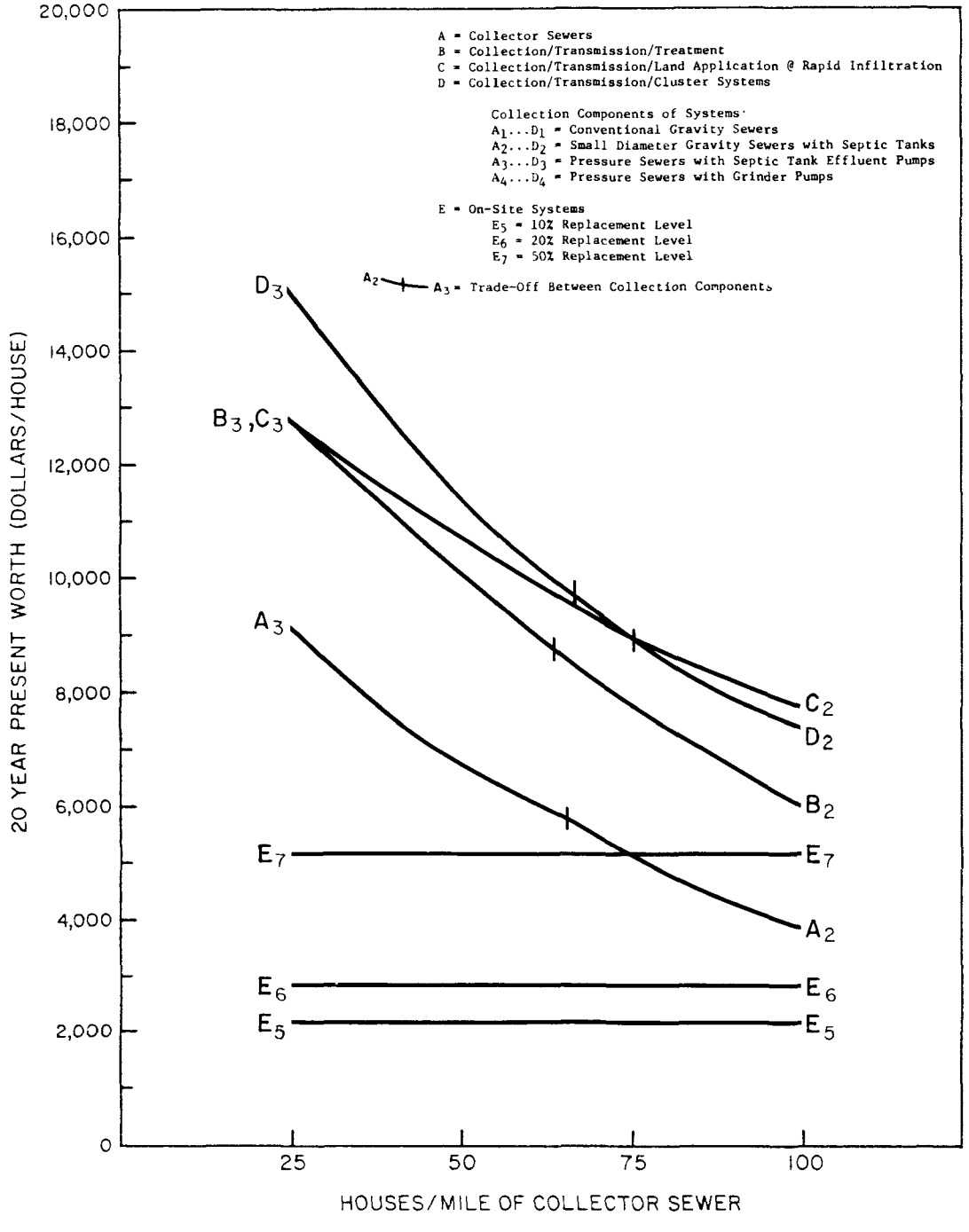

Figure IV-A-4-123. Cost-effectiveness curves for on-site small scale and centralized treatment alternatives for Scenario 7; 0% growth.

COST-EFFECTIVENESS CURVES
Scenario 8
0% Growth

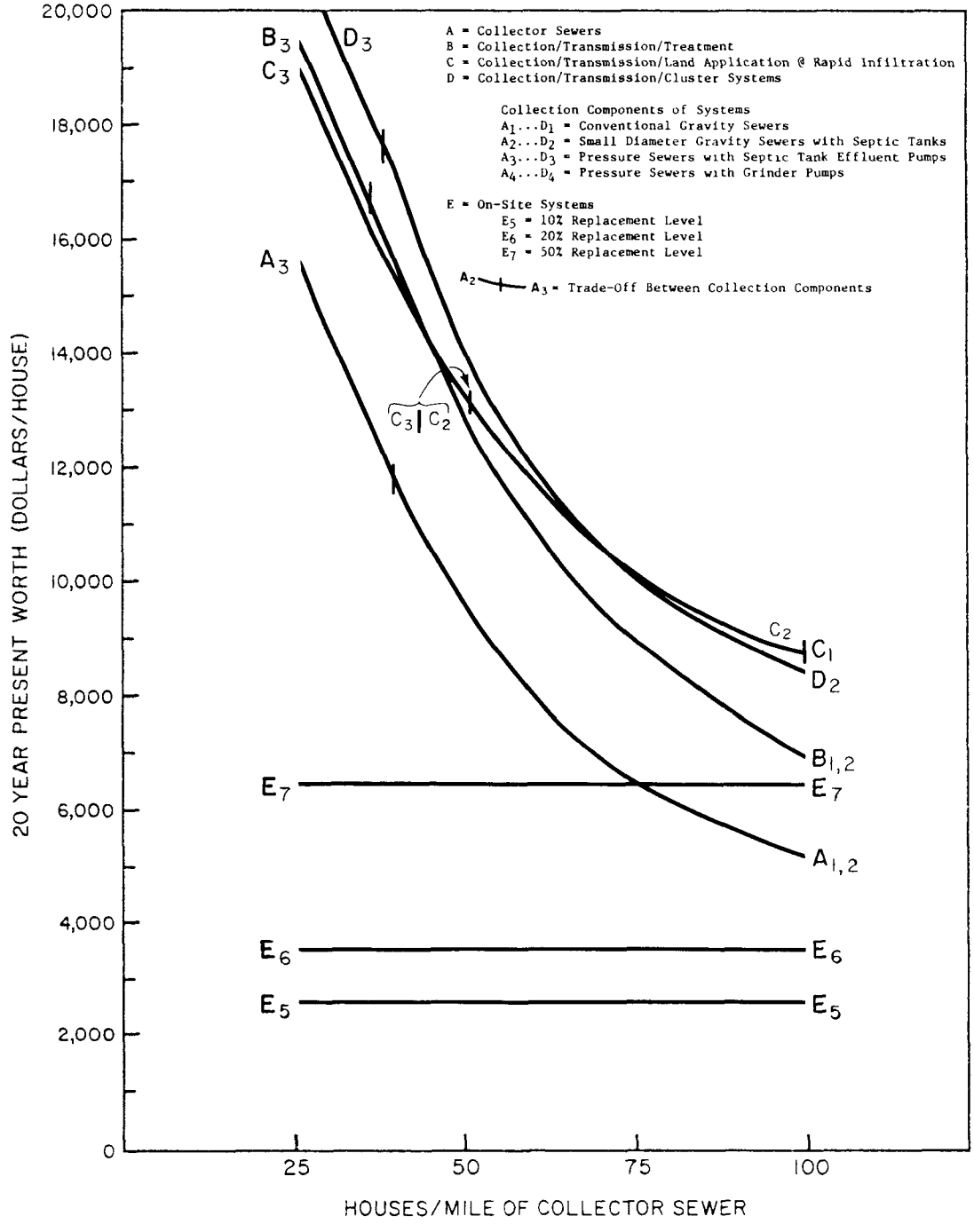

Figure IV-A-4-124. Cost-effectiveness curves for on-site small scale and centralized treatment alternatives for Scenario 8; 0% growth.

COST-EFFECTIVENESS CURVES
Scenario 1
50% Growth

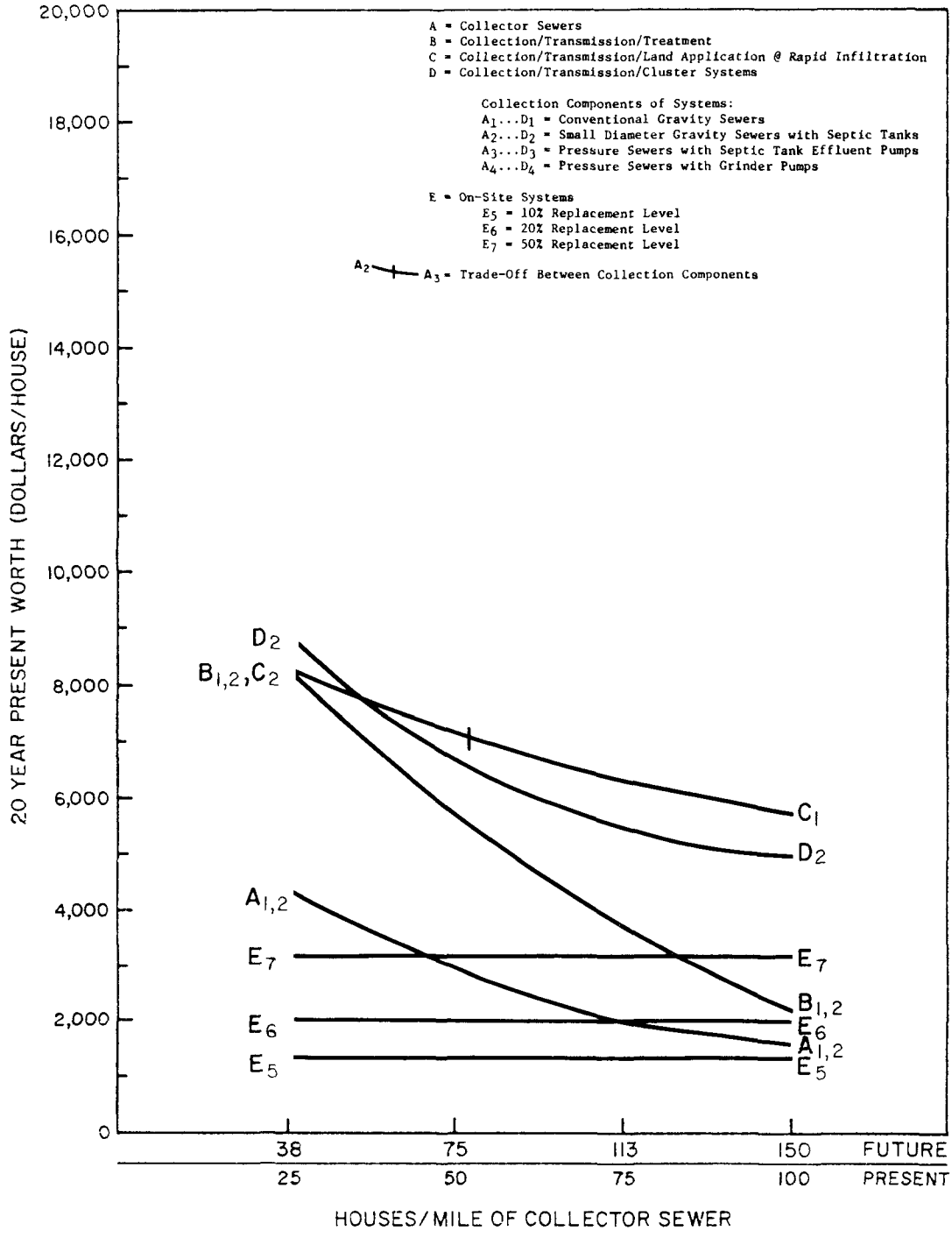

Figure IV-A-4-125. Cost-effectiveness curves for on-site small scale and centralized treatment alternatives for Scenario 1; 50% growth.

COST-EFFECTIVENESS CURVES
Scenario 2
50% Growth

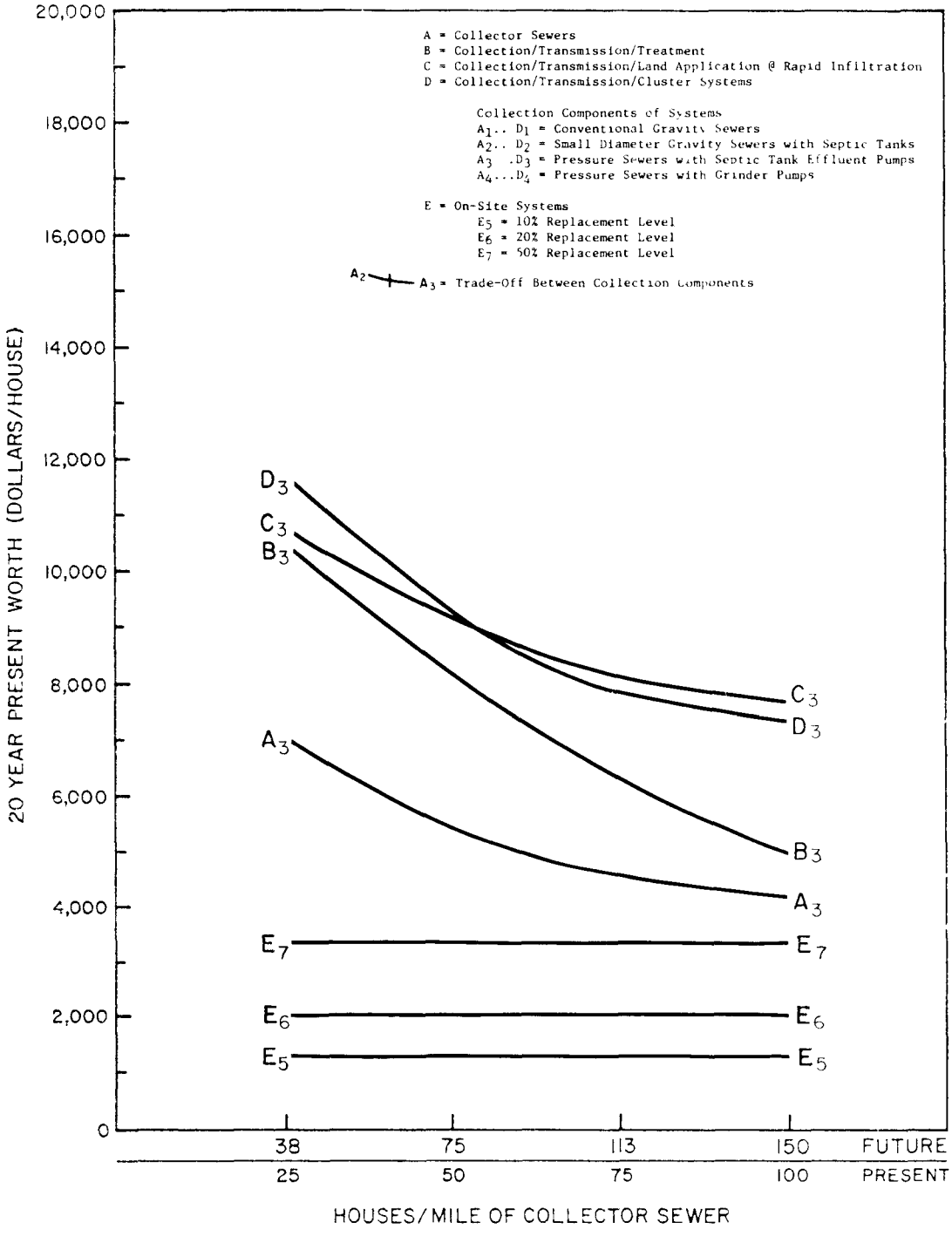

Figure IV-A-4-126. Cost-effectiveness curves for on-site small scale and centralized treatment alternatives for Scenario 2; 50/ growth.

COST-EFFECTIVENESS CURVES
Scenario 3
50% Growth

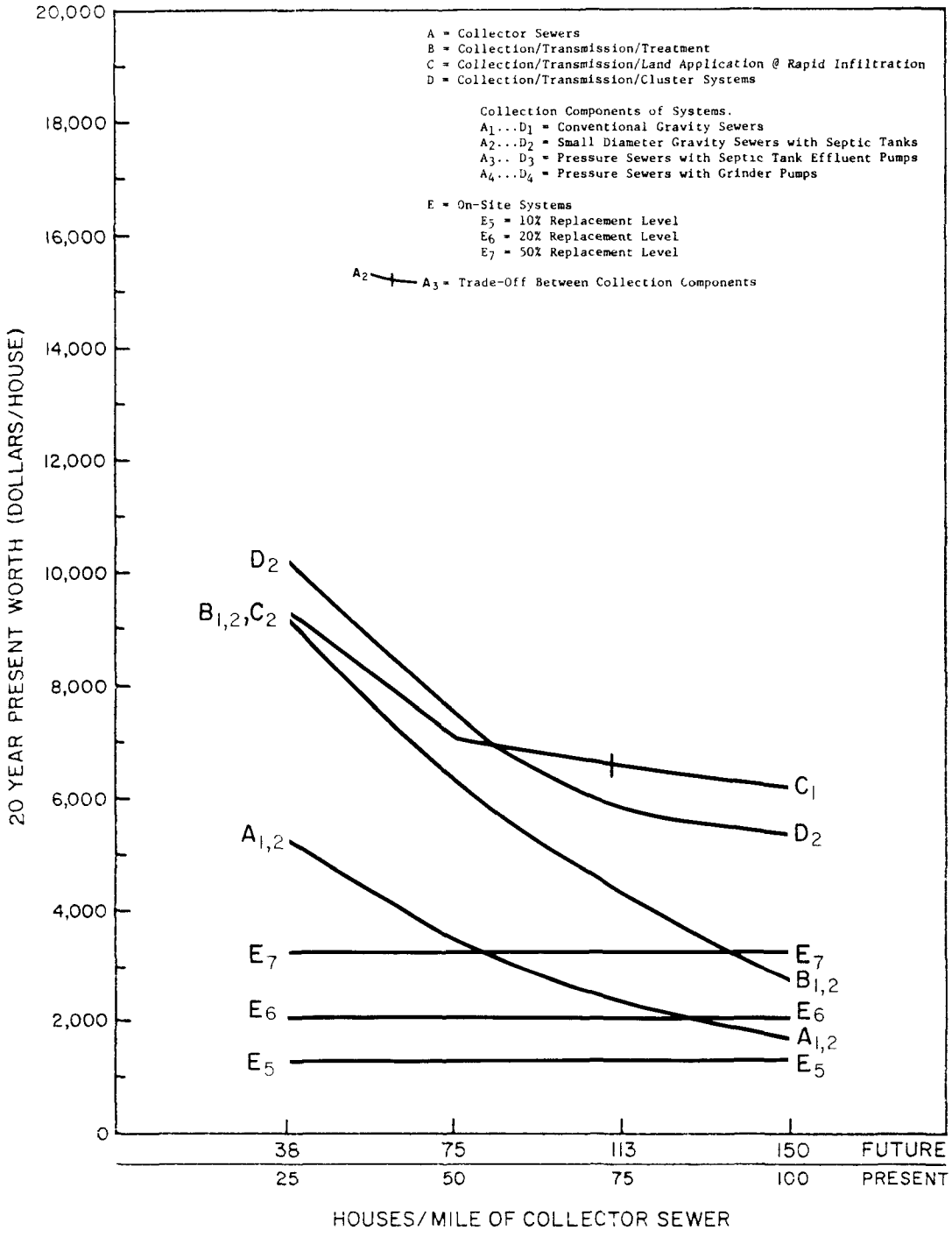

Figure IV-A-4-127. Cost-effectiveness curves for on-site small scale and centralized treatment alternatives for Scenario 3; 50% growth.

COST-EFFECTIVENESS CURVES
Scenario 4
50% Growth

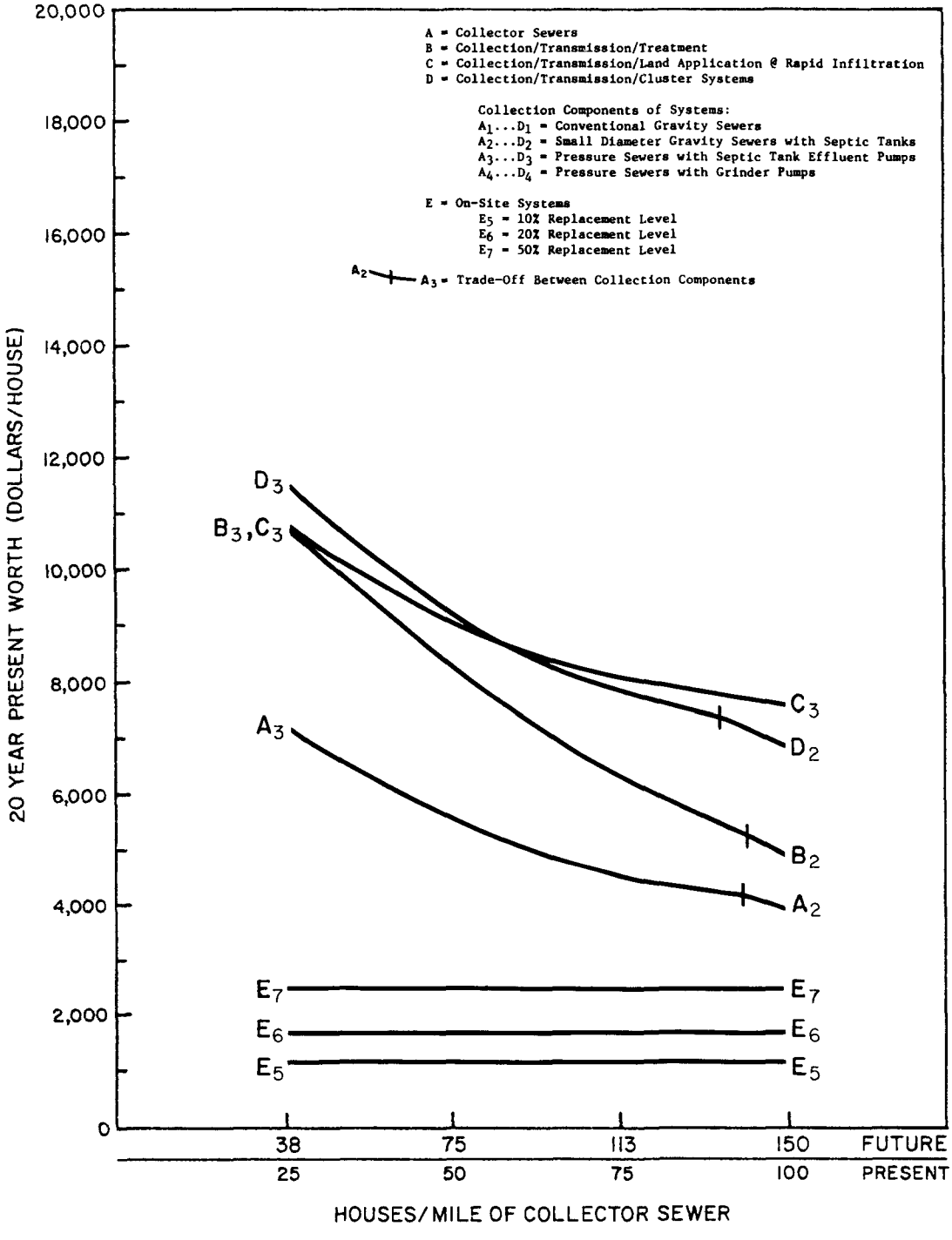

Figure IV-A-4-128. Cost-effectiveness curves for on-site small scale and centralized treatment alternatives for Scenario 4; 50% growth.

COST-EFFECTIVENESS CURVES
Scenario 5
50% Growth

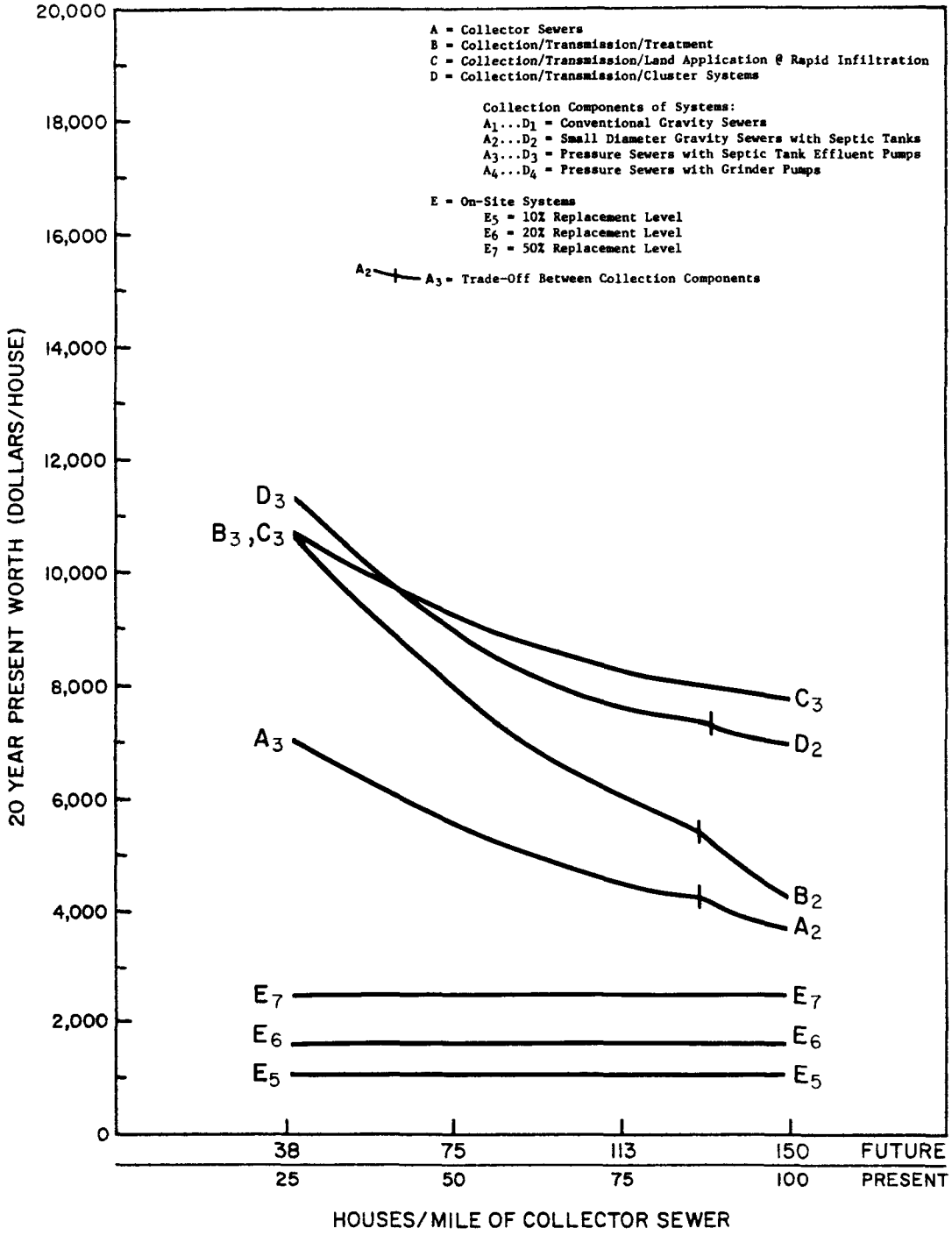

Figure IV-A-4-129. Cost-effectiveness curves for on-site small scale and centralized treatment alternatives for Scenario 5; 50% growth.

COST-EFFECTIVENESS CURVES
Scenario 6
50% Growth

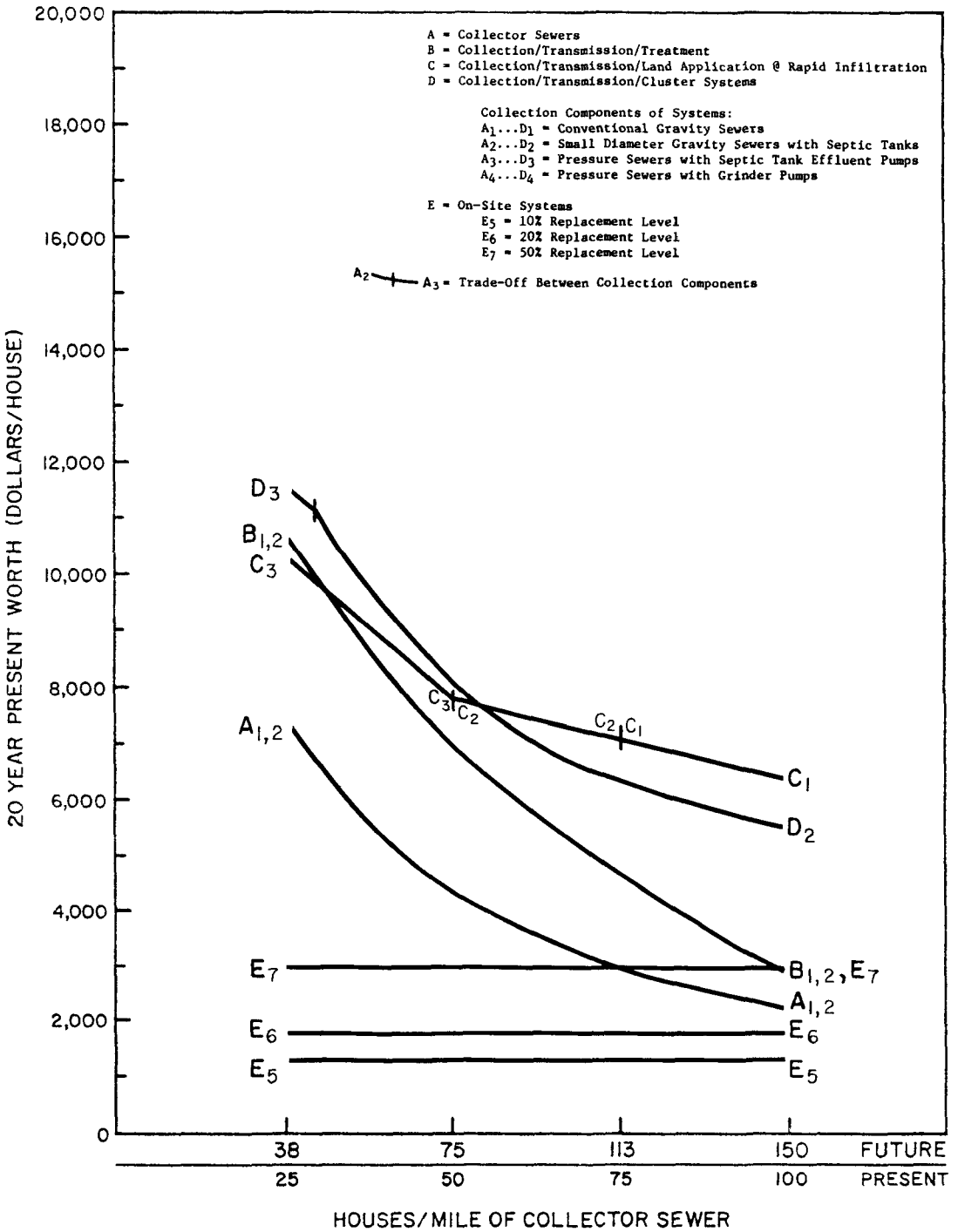

Figure IV-A-4-130. Cost-effectiveness curves for on-site small scale and centralized treatment alternatives for Scenario 6; 50% growth.

COST-EFFECTIVENESS CURVES
Scenario 7
50% Growth

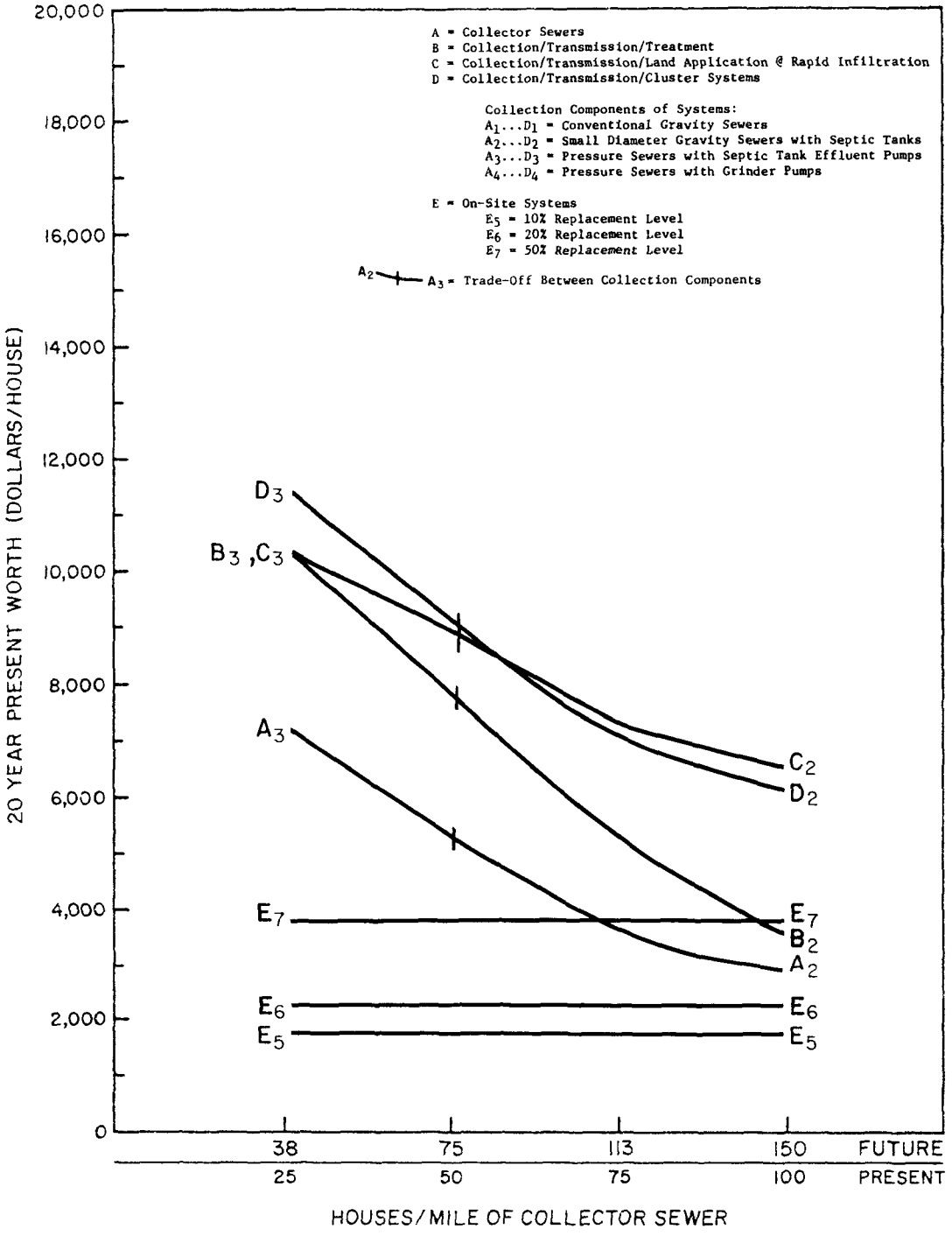

Figure IV-A-4-131. Cost-effectiveness curves for on-site small scale and centralized treatment alternatives for Scenario 7; 50% growth.

COST-EFFECTIVENESS CURVES
Scenario 8
50% Growth

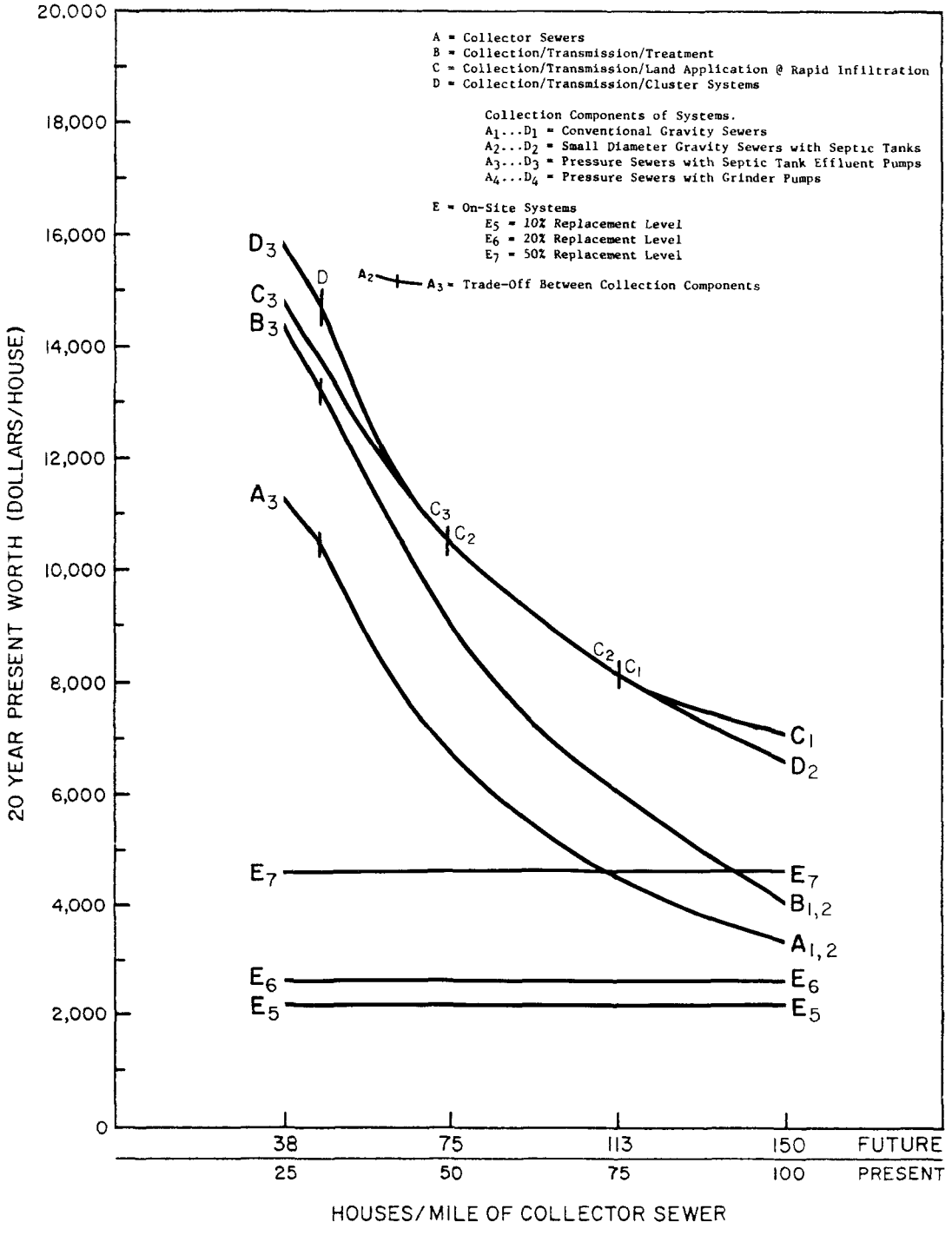

Figure IV-A-4-132. Cost-effectiveness curves for on-site small scale and centralized treatment alternatives for Scenario 8; 50% growth.

APPENDIX B

TABLE IV-A-4-2. COST VARIABILITY STUDY RESULTS

TECHNOLOGY: CONVENTIONAL GRAVITY SEWERS (collection only)

TOTAL PRESENT WORTH DOLLARS PER HOUSE

	DENSITY (houses/mile)	SCENARIO 1	2	3	4	5	6	7	8
0% Growth	25	6,762	26,463	7,855	24,700	24,185	10,360	17,979	17,518
	50	4,011	13,861	4,751	12,980	12,722	6,004	9,619	9,583
	75	2,743	9,309	3,461	8,722	8,550	4,296	6,481	6,682
	100	2,113	7,039	2,652	6,598	6,469	3,278	4,917	5,068
50% Growth	38	4,792	17,753	5,766	16,593	16,254	7,414	12,171	12,123
	75	2,931	9,498	3,649	8,910	8,739	4,484	6,670	6,870
	113	2,003	6,361	2,480	5,971	5,857	3,034	4,484	4,617
	150	1,551	4,834	1,910	4,541	4,455	2,328	3,420	3,520

TABLE IV-A-4-3. COST VARIABILITY STUDY RESULTS

TECHNOLOGY: SMALL DIAMETER GRAVITY SEWERS* with SEPTIC TANKS (collection only)

TOTAL PRESENT WORTH DOLLARS PER HOUSE

	DENSITY (houses/mile)	SCENARIO 1	2	3	4	5	6	7	8
0% Growth	25	6,285	25,580	7,376	19,261	18,550	9,881	13,292	17,039
	50	3,902	13,549	4,641	10,390	10,035	5,894	7,405	9,473
	75	2,786	9,217	3,279	7,111	6,874	4,114	5,121	6,499
	100	2,266	7,090	2,804	5,511	5,333	3,430	4,018	5,220
50% Growth	38	4,582	17,276	5,555	13,119	12,651	7,203	9,192	11,912
	75	2,969	9,401	3,462	7,295	7,058	4,297	5,305	6,683
	113	2,147	6,416	2,623	5,018	4,861	3,177	3,697	4,761
	150	1,777	4,993	2,135	3,939	3,821	2,553	2,944	3,746

TABLE IV-A-4-4. COST VARIABILITY STUDY RESULTS

TECHNOLOGY: PRESSURE SEWERS with SEPTIC TANK EFFLUENT PUMPS (collection only)

TOTAL PRESENT WORTH DOLLARS PER HOUSE

	DENSITY (houses/mile)	SCENARIO 1	2	3	4	5	6	7	8
0% Growth	25	9,086	9,086	9,006	9,326	9,108	9,006	9,006	15,380
	50	6,778	6,778	6,738	6,898	6,788	6,738	6,738	9,925
	75	5,644	5,644	5,617	5,724	5,651	5,617	5,617	7,742
	100	5,114	5,114	5,094	5,174	5,120	5,094	5,094	6,688
50% Growth	38	6,971	6,971	6,918	7,129	6,985	6,918	6,918	11,112
	75	5,376	5,376	5,350	5,456	5,383	5,350	5,350	7,474
	113	4,521	4,521	4,503	4,574	4,526	4,503	4,503	5,914
	150	4,136	4,136	4,123	4,176	4,139	4,123	4,123	5,175

TABLE IV-A-4-5. COST VARIABILITY STUDY RESULTS

TECHNOLOGY: PRESSURE SEWERS with GRINDER PUMPS (collection only)

TOTAL PRESENT WORTH DOLLARS PER HOUSE

	DENSITY (houses/mile)	SCENARIO 1	2	3	4	5	6	7	8
0% Growth	25	9,329	9,329	9,249	9,569	9,351	9,249	9,249	15,704
	50	7,052	7,052	7,012	7,173	7,063	7,012	7,012	10,712
	75	5,953	5,953	5,926	6,033	5,960	5,926	5,926	8,078
	100	5,453	5,453	5,433	5,513	5,458	5,433	5,433	7,047
50% Growth	38	8,039	8,039	7,986	8,197	8,053	7,986	7,986	12,232
	75	6,276	6,276	6,249	6,356	6,283	6,249	6,249	8,716
	113	5,350	5,350	5,332	5,403	5,355	5,332	5,332	6,760
	150	4,935	4,935	4,922	4,976	4,939	4,922	4,922	5,998

TABLE IV-A-4-6. COST VARIABILITY STUDY RESULTS

TECHNOLOGY: GRAVITY SEWERS [collection (CG*), transmission and centralized treatment]

TOTAL PRESENT WORTH DOLLARS PER HOUSE

	DENSITY (houses/mile)	SCENARIO 1	2	3	4	5	6	7	8
0% Growth	25	10,474	30,175	11,567	28,412	27,897	14,072	21,691	21,230
	50	7,108	16,958	7,848	16,077	15,819	9,101	12,716	12,680
	75	5,225	11,791	5,943	11,204	11,032	6,778	8,963	9,164
	100	3,979	8,905	4,518	8,464	8,335	5,144	6,783	6,934
50% Growth	38	8,184	21,145	9,158	19,985	19,646	10,806	15,563	15,515
	75	5,413	11,980	6,131	11,392	11,221	6,966	9,152	9,352
	113	3,549	7,907	4,026	7,517	7,403	4,580	6,030	6,163
	150	2,187	5,470	2,546	5,177	5,091	2,964	4,056	4,156

*CG = conventional gravity sewers

TABLE IV-A-4-7. COST VARIABILITY STUDY RESULTS

TECHNOLOGY: COLLECTION (SD)*, TRANSMISSION and CENTRALIZED TREATMENT

TOTAL PRESENT WORTH DOLLARS PER HOUSE

	DENSITY (houses/mile)	SCENARIO 1	2	3	4	5	6	7	8
0% Growth	25	9,997	29,292	11,088	22,973	22,262	13,593	17,004	20,751
	50	6,999	16,646	7,738	13,487	13,132	8,991	10,502	12,570
	75	5,268	11,699	5,761	9,593	9,356	6,596	7,603	8,981
	100	4,132	8,956	4,670	7,377	7,199	5,296	5,884	7,086
50% Growth	38	7,974	20,668	8,947	16,511	16,043	10,595	12,584	15,304
	75	5,451	11,883	5,944	9,777	9,540	6,779	7,787	9,165
	113	3,693	7,962	4,169	6,564	6,407	4,723	5,243	6,307
	150	2,413	5,629	2,771	4,575	4,457	3,189	3,580	4,382

*SD = small diameter (4 in.) gravity sewers

TABLE IV-A-4-8. COST VARIABILITY STUDY RESULTS

TECHNOLOGY: COLLECTION (PS/STEP)*, TRANSMISSION and CENTRALIZED TREATMENT

TOTAL PRESENT WORTH DOLLARS PER HOUSE

	DENSITY (houses/mile)	SCENARIO 1	2	3	4	5	6	7	8
0% Growth	25	12,798	12,798	12,718	13,038	12,820	12,718	12,718	19,092
	50	9,875	9,875	9,835	9,995	9,885	9,835	9,835	13,022
	75	8,126	8,126	8,099	8,206	8,133	8,099	8,099	10,224
	100	6,980	6,980	6,960	7,040	6,986	6,960	6,960	8,554
50% Growth	38	10,363	10,363	10,310	10,521	10,377	10,310	10,310	14,504
	75	7,858	7,858	7,832	7,938	7,865	7,832	7,832	9,956
	113	6,067	6,067	6,049	6,120	6,072	6,049	6,049	7,460
	150	4,772	4,772	4,759	4,812	4,775	4,759	4,759	5,811

*PS/STEP = pressure sewers with septic tank effluent pumps

TABLE IV-A-4-9. COST VARIABILITY STUDY RESULTS

TECHNOLOGY: COLLECTION (PS/GP)*, TRANSMISSION AND CENTRALIZED TREATMENT

TOTAL PRESENT WORTH DOLLARS PER HOUSE

	DENSITY (houses/mile)	SCENARIO 1	2	3	4	5	6	7	8
0% Growth	25	13,041	13,041	12,961	13,281	13,063	12,961	12,961	19,416
	50	10,149	10,149	10,109	10,270	10,160	10,109	10,109	13,809
	75	8,435	8,435	8,408	8,515	8,442	8,408	8,408	10,560
	100	7,319	7,319	7,299	7,379	7,324	7,299	7,299	8,913
50% Growth	38	11,431	11,431	11,378	11,589	11,445	11,378	11,378	15,624
	75	8,758	8,758	8,731	8,836	8,765	8,731	8,731	11,198
	113	6,896	6,896	6,878	6,949	6,901	6,878	6,878	8,306
	150	5,571	5,571	5,558	5,612	5,575	5,558	5,558	6,634

*PS/GP = pressure sewers with grinder pumps

TABLE IV-A-4-10. COST VARIABILITY STUDY RESULTS

TECHNOLOGY: COLLECTION (MIXED TECHNOLOGIES)*, TRANSMISSION, and LAND APPLICATION @ RAPID INFILTRATION

TOTAL PRESENT WORTH DOLLARS PER HOUSE

	DENSITY (houses/mile)	SCENARIO 1	2	3	4	5	6	7	8
0% Growth	25	(SD) 9798	(PS/ST) 12,599	(SD) 10,889	(PS/ST) 12,839	(PS/ST) 12,621	(PS/ST) 12,514	(PS/ST) 12,519	(PS/ST) 18,893
	50	(SD) 7415	(PS/ST) 10,291	(SD) 8,154	(PS/ST) 10,411	(PS/ST) 10,301	(SD) 9,407	(PS/ST) 10,521	(SD) 12,986
	75	(CG) 6780	(PS/ST) 9,157	(SD) 6,792	(PS/ST) 9,237	(PS/ST) 9,164	(SD) 7,627	(SD) 8,634	(SD) 10,012
	100	(CG) 6156	(PS/ST) 8,627	(CG) 6,695	(PS/ST) 8,687	(PS/ST) 8,633	(CG) 7,321	(SD) 7,531	(CG) 8,581
50% Growth	38	(SD) 8095	(PS/ST) 10,484	(SD) 9,068	(PS/ST) 10,642	(PS/ST) 10,498	(PS/ST) 10,426	(PS/ST) 10,431	(PS/ST) 14,625
	75	(CG) 6974	(PS/ST) 8,889	(SD) 6,975	(PS/ST) 8,969	(PS/ST) 8,896	(SD) 7,810	(SD) 8,818	(SD) 10,196
	113	(CG) 6046	(PS/ST) 8,034	(CG) 6,523	(PS/ST) 8,087	(PS/ST) 8,039	(CG) 7,077	(SD) 7,210	(CG) 8,130
	150	(CG) 5594	(PS/ST) 7,649	(CG) 5,953	(PS/ST) 7,689	(PS/ST) 7,652	(CG) 6,371	(SD) 6,457	(CG) 7,033

*Collection technology is indicated parenthetically in each grid square.

TABLE IV-A-4-11. COST VARIABILITY STUDY RESULTS

TECHNOLOGY: COLLECTION (MIXED TECHNOLOGIES), TRANSMISSION, and LAND APPLICATION @ SLOW RATE

TOTAL PRESENT WORTH DOLLARS PER HOUSE

	DENSITY (houses/mile)	SCENARIO 1	2	3	4	5	6	7	8
0% Growth	25	(SD) 11,528	(PS/ST) 14,329	(SD) 12,619	(PS/ST) 14,569	(PS/ST) 14,351	(PS/ST) 14,249	(PS/ST) 14,249	(PS/ST) 20,623
	50	(SD) 9,145	(PS/ST) 12,021	(SD) 9,884	(PS/ST) 12,141	(PS/ST) 12,031	(SD) 11,137	(PS/ST) 11,981	(SD) 14,716
	75	(CG) 8,516	(PS/ST) 10,887	(SD) 8,522	(PS/ST) 10,967	(PS/ST) 10,894	(SD) 9,357	(SD) 10,364	(SD) 11,742
	100	(CG) 7,886	(PS/ST) 10,357	(CG) 8,425	(PS/ST) 10,417	(PS/ST) 10,363	(CG) 9,051	(SD) 9,261	(CG) 10,841
50% Growth	38	(SD) 9,825	(PS/ST) 12,214	(SD) 10,798	(PS/ST) 12,372	(PS/ST) 12,228	(PS/ST) 12,156	(PS/ST) 12,161	(PS/ST) 16,355
	75	(CG) 8,704	(PS/ST) 10,619	(SD) 8,705	(PS/ST) 10,699	(PS/ST) 10,626	(SD) 9,540	(SD) 10,548	(SD) 11,926
	113	(CG) 7,776	(PS/ST) 9,764	(CG) 8,253	(PS/ST) 9,817	(PS/ST) 9,769	(CG) 8,807	(SD) 8,940	(CG) 10,390
	150	(CG) 7,324	(PS/ST) 9,379	(CG) 7,683	(PS/ST) 9,419	(PS/ST) 9,382	(CG) 8,101	(SD) 8,187	(CG) 9.293

TABLE IV-A-4-12. COST VARIABILITY STUDY RESULTS

TECHNOLOGY: COLLECTION (MIXED TECHNOLOGIES), TRANSMISSION, and LAND APPLICATION @ OVERLAND FLOW

TOTAL PRESENT WORTH DOLLARS PER HOUSE

	DENSITY (houses/mile)	SCENARIO 1	2	3	4	5	6	7	8
0% Growth	25	(SD) 10,258	(PS/ST) 13,059	(SD) 11,349	(PS/ST) 13,299	(PS/ST) 13,081	(PS/ST) 12,974	(PS/ST) 12,979	(PS/ST) 19,353
	50	(SD) 7,875	(PS/ST) 10,751	(SD) 8,614	(PS/ST) 10,871	(PS/ST) 10,761	(SD) 9,867	(PS/ST) 10,711	(SD) 13,446
	75	(CG) 7,246	(PS/ST) 9,617	(SD) 7,252	(PS/ST) 9,697	(PS/ST) 9,624	(SD) 8,087	(SD) 9,094	(SD) 10,473
	100	(CG) 6,616	(PS/ST) 9,087	(CG) 7,155	(PS/ST) 9,147	(PS/ST) 9,093	(CG) 7,781	(SD) 7,991	(CG) 9,471
50% Growth	38	(SD) 8,555	(PS/ST) 10,944	(SD) 9,528	(PS/ST) 11,102	(PS/ST) 10,958	(PS/ST) 10,886	(PS/ST) 10,891	(PS/ST) 15,085
	75	(CG) 7,434	(PS/ST) 9,349	(SD) 7,435	(PS/ST) 9,429	(PS/ST) 9,356	(SD) 8,270	(SD) 7,278	(SD) 10,656
	113	(CG) 6,506	(PS/ST) 8,494	(CG) 6,983	(PS/ST) 8,547	(PS/ST) 8,499	(CG) 7,537	(SD) 7,670	(CG) 9,120
	150	(CG) 6,054	(PS/ST) 8,109	(CG) 6,413	(PS/ST) 8,149	(PS/ST) 8,112	(CG) 6,831	(SD) 6,917	(CG) 8,023

TABLE IV-A-4-13. COST VARIABILITY STUDY RESULTS

TECHNOLOGY: COLLECTION (CG), TRANSMISSION and CLUSTER SYSTEMS

TOTAL PRESENT WORTH DOLLARS PER HOUSE

	DENSITY (houses/mile)	SCENARIO 1	2	3	4	5	6	7	8
0% Growth	25	13,923	33,624	15,016	31,861	31,346	17,521	25,140	24,679
	50	9,589	19,439	10,329	18,558	18,300	11,582	15,197	15,161
	75	7,003	13,569	7,721	12,982	12,810	8,556	10,741	10,942
	100	6,230	11,156	6,769	10,715	10,586	7,395	9,034	9,185
50% Growth	38	10,052	23,013	11,026	21,853	21,514	12,674	17,431	17,383
	75	7,191	13,758	7,909	13,170	12,999	8,744	10,930	11,130
	113	5,842	10,200	6,319	9,810	9,696	6,873	8,323	8,456
	150	5,174	8,457	5,533	8,164	8,078	5,951	7,043	7,143

TABLE IV-A-4-14. COST VARIABILITY STUDY RESULTS

TECHNOLOGY: COLLECTION (SD), TRANSMISSION and CLUSTER SYSTEMS

TOTAL PRESENT WORTH DOLLARS PER HOUSE

	DENSITY (houses/mile)	SCENARIO 1	2	3	4	5	6	7	8
0% Growth	25	12,139	31,434	13,230	25,115	24,404	15,735	19,146	22,893
	50	8,453	18,100	9,192	14,941	14,586	10,445	11,956	14,024
	75	6,362	12,793	6,855	10,687	10,450	7,690	8,697	10,075
	100	5,693	10,517	6,231	8,938	8,760	6,857	7,445	8,647
50% Growth	38	8,982	21,676	9,955	17,519	17,051	11,603	13,592	16,312
	75	6,545	12,977	7,038	10,871	10,634	7,873	8,881	10,259
	113	5,376	9,645	5,852	8,247	8,090	6,406	6,926	7,990
	150	4,870	8,086	5,228	7,032	6,914	5,646	6,037	6,839

TABLE IV-A-4-15. COST VARIABILITY STUDY RESULTS

TECHNOLOGY: COLLECTION (PS/STEP), TRANSMISSION, and CLUSTER SYSTEMS

TOTAL PRESENT WORTH DOLLARS PER HOUSE

	DENSITY (houses/mile)	SCENARIO 1	2	3	4	5	6	7	8
0% Growth	25	14,922	14,940	14,870	15,180	14,962	14,870	14,860	21,244
	50	11,306	11,329	11,289	11,449	11,339	11,289	11,289	14,476
	75	9,204	9,220	9,193	9,300	9,227	9,193	9,193	11,318
	100	8,523	8,541	8,521	8,601	8,547	8,521	8,521	10,115
50% Growth	38	11,360	11,371	11,326	11,529	11,385	11,326	11,318	15,520
	75	8,936	8,952	8,926	9,032	8,959	8,926	8,926	11,050
	113	7,734	7,750	7,732	7,803	7,755	7,732	7,732	9,143
	150	7,210	7,229	7,211	7,269	7,232	7,211	7,216	8,263

TABLE IV-A-4-16. COST VARIABILITY STUDY RESULTS

TECHNOLOGY: COLLECTION (PS/GP), TRANSMISSION, and CLUSTER SYSTEMS

TOTAL PRESENT WORTH DOLLARS PER HOUSE

	DENSITY (houses/mile)	SCENARIO 1	2	3	4	5	6	7	8
0% Growth	25	15,079	15,066	15,005	16,730	16,512	15,005	16,410	21,460
	50	11,571	11,571	11,531	12,751	12,641	11,531	12,590	15,231
	75	9,507	9,507	9,480	9,952	9,879	9,480	9,845	11,632
	100	8,879	8,880	8,840	9,630	9,575	8,840	9,550	10,454
50% Growth	38	12,371	12,363	12,533	12,521	12,377	12,322	12,310	16,568
	75	9,830	9,830	9,803	10,275	10,202	9,803	10,168	12,270
	113	8,577	8,579	8,542	8,632	8,584	8,542	8,561	9,970
	150	7,998	7,998	7,968	8,319	8,282	7,968	8,265	9,044

B. PLANNING AND DESIGN COSTS FOR SMALL WASTE FLOWS AREAS

1. CENTRALIZED VERSUS DECENTRALIZED COSTS

During the planning stage, estimates must be made for the design and construction of the proposed project. Design costs for conventional collection and treatment systems are typically estimated during the planning stage as a percentage of the construction cost. Planning and design costs for decentralized systems, however, cannot usually be estimated as a percentage of construction costs. Decentralized systems require extensive field work to gather data on design and performance of existing systems. In contrast, the field work associated with planning and designing conventional systems includes visiting proposed and existing wastewater treatment plant sites along with reviewing proposed interceptor routes. Soil conditions in each backyard are of no importance to the engineer planning and designing a collection system. Design and planning for decentralized alternatives must include costs associated with data collection such as sanitary surveys, well water sampling, and soil sampling, as well as costs associated with conventional planning and design (i.e., report writing, plans and specifications, etc.). Costs for decentralized planning and design can, therefore, be expected to be higher than those costs for centralized facilities even though construction costs are usually less.

Quantifying the increased costs for planning and design of decentralized systems is difficult because of a lack of data. One approach is to estimate the field work required and its associated costs. A procedure to make such estimates is included in Section B.2 of this chapter. The results of this procedure provide costs for field work and design for on-site systems (site-specific analysis). Combining these costs with other standard costs (contingencies, legal, and administrative) can provide an estimate of costs required in addition to construction costs for a project to proceed. Table IV-B-1 compares the costs for engineering, contingencies, legal, administrative, and site-specific analysis for centralized and decentralized treatment systems. Estimates for these costs will vary from one engineer to another, even for centralized wastewater treatment systems.

TABLE IV-B-1. COSTS FOR ENGINEERING, CONTINGENCIES, LEGAL, ADMINISTRATIVE, AND SITE-SPECIFIC ANALYSIS FOR CENTRALIZED VS. DECENTRALIZED TREATMENT SYSTEMS

Cost component	Centralized system (% of construction cost)	Decentralized system (% of construction cost unless noted)
Engineering	10	per project*
Contingency	10	10
Legal	4	5
Administrative	3	5
Site-Specific Analysis	--	per project*
Total	27%	20% + engineering + site-specific analysis

* See Chapter IV, Section B.2.

As can be seen in Table IV-B-1, the primary difference in the costs estimated for the various cost components results from site-specific analysis and engineering design. For decentralized systems, these costs cannot be estimated as a percentage of construction costs because of the lower construction costs and the variance in these costs from one project to another. The malfunction rate of local on-site systems often dictates the cost of the engineering and site-specific analysis as is reflected in the methodology for estimating these costs in Chapter IV-B-2.

2. ESTIMATING SITE-SPECIFIC ANALYSIS AND FACILITIES DESIGN COSTS

On-site treatment systems cannot be designed without site-specific data. The collection of these data increases the cost of the design phase (typically Step 2 but possible in Step 1) of a project. The cost of collecting site-specific data depends on such factors as size of area, level of development, failure rate of on-site systems, and personnel used for data collection. Keeping in mind the limitations of not knowing these factors along with the lack of experience in such endeavors, the following methodology can be used as an aid in estimating preliminary costs for a site-specific analysis. Two major assumptions are inherent in these estimates: (1) number of dwellings in an area and (2) estimated percent replacement. The estimated percent replacement is used to estimate the work required for various tasks within the site-specific analysis. Replacement levels are cited at 10, 20, and 50 percent throughout the presentation of the methodology. Changes in the percent replacement levels would correspondingly change the work to be performed in each task. Some tasks are estimated on a per-mile basis as noted.* A description for each step is given below and is followed by the cost estimating procedure.

- Work Description for Site-Specific Analysis and Facility Design

Step 1. Sanitary Survey - Survey of each house to include: completion of questionnaire, inspection of site, inspection of well, explanation of access requirements, provision of access papers, education, coordination with survey team on decisions for succeeding steps.

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* Note that this procedure is valid for estimating design and site-specific analysis costs at the alternatives development stage. In estimating these costs for a proposed action or a critical comparison of alternatives, more detailed assumptions incorporating local data should be made. Any costs likely to be incurred that are not mentioned here, should be included and recorded as a means of constantly improving the estimates made through this methodology. Some of the steps included in the site-specific analysis can be performed during Step 1 as well as Step 2. See Chapter XVI-D, Alternative Construction Grant Procedures, for the various points in the Construction Grant Program that certain tasks can be performed.

Step 2. Well Sampling - For homes with private wells less than 50 feet from sewage facilities or otherwise subject to wastewater contamination, collect sample for analysis of fecal coliform bacteria, nitrates, and whiteners. Costs for analysis of well water samples are included under local costs as a service contract.

Step 3. Septic Tank Inspection - For septic tanks: (a) for which no records or memory of size or construction exist, (b) for systems which have reported backups, (c) for systems which have surfaced in the past, and (d) reported to be small. Locate, uncover, pump, and inspect for construction, leaks, deterioration, condition, and type of sanitary tees and baffles. Rod influent line noting roots, other obstructions or collapsed line. Rod effluent line noting same plus distance to headers, distribution box, bends, and obstructions. A three-person crew will be required for the total number of days involved. Costs for crew expenses are included under local costs in the service contracts section along with the cost for treating the septage pumped for the septic tanks.

Step 4. Soil Sampling - For lots with: (a) past or present malfunctions not explained by survey results or septic tank inspection, (b) substandard soil disposal units, or (c) no record for soil disposal unit. Determine soil texture, color, depth to seasonal high groundwater level, water table at time of sampling in suspected area of soil disposal unit and in alternate sites on and near lot. Probe suspected area of soil disposal unit for depth, size, and type of construction.

Step 5. Header and Drainfield Excavation - For systems having recurrent backups or past surface malfunctions not explained in prior steps, hand excavate effluent line from point of obstruction, headers, etc., to uncover distribution system. Hand excavate test pits to verify size, depth, and type of subsurface disposal unit. Evaluate soil crusting, decomposition or silting in of aggregate, poor distribution, and soil hydraulics as reasons for failure.

Step 6. Well Water Meter Installation - For on-site systems with limited hydraulic capacity as determined by sanitary survey, soil sampling or excavation, install water meter (after outside faucet) and read at least monthly during survey. The cost of water meters is included as a local cost under service contracts.

Step 7. Shallow Groundwater Sampling - Shoreline scan with leachate detector to detect emergent plumes. Follow-up at sites with plumes with shoreline transect and 5 samples per plume for bacterial and nutrient analysis. The cost of these analyses is included as a local cost under service contracts and could probably be performed by the same organization that does the well sample analyses. Estimated times are based on surveying one mile of shoreline.

Step 8. Supervision/Documentation/Clerical Support - The total project length for the site-specific analysis is estimated and the sanitarian's supervision hours are derived by subtracting his hours in Tasks 1, 5, and 7 from the total estimated project length. Clerical time is required for the duration of the project. Total project length can be estimated based on the amount of time required for completion of individual steps and rounded up to an even month or half month.

Step 9. Engineering Design for On-Site Alternatives - The total number of systems to be replaced will be designed in some fashion prior to installation. The time required per system is assumed to increase with the overall severity of problems in the area. One reason for this is the reliance on standard designs for areas with only a few problems. Areas with more severe problems require more time per system, especially if many systems have site-specific problems that require detailed individual designs. The design hours per system replaced is only a rough estimate because of lack of actual data in this area.

- Cost Estimate for Site-Specific Analysis and Engineering Design

Assumptions - Number of Dwellings _____
Estimated Percent Replacement _____
Costs are for July 1980 ENR Index = 3260

Step 1. Sanitary Survey (4 dwellings/person/day) - $\frac{\text{No. Dwellings}}{4}$ =

_____ person days

Personnel:	Sanitarian (9%)	_____ person-days
	Sr. Engineer (9%)	_____ person-days
	Soil Scientist (5%)	_____ person-days
	Jr. Engineer (9%)	_____ person-days
	Surveyors (63%)	_____ person-days
	Water Quality Scientist (5%)	_____ person-days

Step 2. Well Sampling - (Assume $\begin{matrix}15\%\\50\%\end{matrix}$ of dwellings for $\begin{matrix}10\%\\50\%\end{matrix}$ replacement)

÷ 10 dwellings/person/day or _____ person days

Personnel:	Surveyors (85%)	_____ person-days
	Water Quality Scientist (15%)	_____ person-days

Step 3. Septic Tank Inspection - ($\begin{matrix}60\%\\70\%\\80\%\end{matrix}$ of dwellings for $\begin{matrix}10\%\\20\%\\50\%\end{matrix}$ replacement)

÷ 6 dwellings/person/day or _____ person-days

Personnel: Jr. Engineer (100%) _____ person-days

Step 4. Soil Sampling - ($\begin{matrix}30\%\\60\%\\80\%\end{matrix}$ of dwellings for $\begin{matrix}10\%\\20\%\\50\%\end{matrix}$ replacement)

÷ 4 dwellings/day/2 persons or _____ person-days

Personnel:	Soil Scientist (52%)	_____ person-days
	Surveyor (48%)	_____ person-days

Step 5. Header and Drainfield Excavation -

$$\left(\begin{matrix}5\%\\10\%\\25\%\end{matrix}\right. \text{ of dwellings for } \left.\begin{matrix}10\%\\20\%\\50\%\end{matrix}\right. \text{ replacement)}$$

÷ (2 dwellings/day/supervisor + 1 dwelling/day/2 persons) = _____ person-days

Personnel: Sanitarian (14%) _____ person-days
Laborer (86%) _____ person-days

Step 6. Well Water Meter Installation -

$$\left(\begin{matrix}10\%\\20\%\\45\%\end{matrix}\right. \text{ of dwellings for } \left.\begin{matrix}10\%\\20\%\\50\%\end{matrix}\right. \text{ replacement) x 6 readings/dwelling}$$

÷ 24 dwellings/person/day = _____ person-days

Personnel: Surveyor (100%) _____ person-days

Step 7. Shoreline Plume Detection and Analysis

Leachate detection survey: $\frac{1 \text{ day}}{\text{persons}}$ x ___ miles of shoreline x 2 = _____ person-days

Transects: $\left(\begin{matrix}3\\5\\10\end{matrix}\right.$ plumes for $\left.\begin{matrix}10\%\\20\%\\50\%\end{matrix}\right.$ replacement x ____ miles of shoreline x $\frac{1 \text{ day}}{2 \text{ plumes}}$ x 2 persons

Personnel: Water Quality Scientist (50%) _____ person-days
Sanitarian (25%) _____ person-days
Surveyor (25%) _____ person-days

Step 8. Supervision/Documentation/Clerical - Assume ____ work days total

Personnel: Sanitarian (100%) including Tasks 1, 5, and 7 _____ person-days
Report-writing time can vary; rough estimate is 5 days per linear mile of dwellings _____ person-days
Secretary (100%) _____ person-days

Step 9. Facility Design for On-Site Alternatives -

% Replacement	No. of dwellings	No. of Systems replaced (% replaced x No. of dwellings	Design Hours per system replaced	Person-days
10			4	
20			6	
50			8	

Personnel: Sr. Engineer (100%) _____ person-days

- Labor Summary

Personnel	Steps 1	2	3	4	5	6	7	8	9	Total
Sanitarian										
Sr. Engineer										
Jr. Engineer										
Soil Scientist										
Water Quality Scientist										
Surveyors										
Laborers										
Secretary										
TOTAL										

- Local Costs

*--Salaries: Sanitarian @ $25,000/year for __ days =
Surveyors @ $11,000/year for __ days =
Laborers @ $12,000/year for __ days =
Secretary @ $12,000/year for __ days = _____

* Assumes 260 working days per year.

20% fringe = _____
Subtotal
--Rent: Office @ $300/month x __ months = _____
--Service Contracts: Well samples (from Step 2 above) = _____
____ samples x $10/sample = _____
Septic tank inspection @ $450/day for a 3-person crew x _____ days (see Step 3 above) = _____
Waste disposal fee @ $20/septic tank x ___ septic tanks pumped (see Step 3 above) = _____
Water meters @ $175/water meter x ___ water meters (see Step 6 above) = _____
Subtotal

--Equipment & Sampling Supplies: Leachate Detector
Groundwater Flowmeter
Field Sampling Equipment
Paper Supplies
Cameras & Film for documentation/records = _____ $13.1 per dwelling x _____ dwellings*

_____ Vans @ $350/month + $120 for gas & oil/month x ____ months = _____
Subtotal

--Summary:

Salaries = _____
Rent = _____
Contracts = _____
Equipment & Supplies = _____
Total Local Costs

_____
* From Seven Rural Lake EIS's

- Consultant Costs

--Direct Labor:
Sr. Engineer @ $35,000/year, ___ days = _____
Jr. Engineer @ $20,000/year, ___ days = _____
Soil Scientist @ $25,000/year, ___ days = _____
Water Quality Scientist @ $25,000/year, ___ days = _____
Total person-days ____ Subtotal

--Other Direct Costs:
Report Reproduction
Communication $3/dwelling x ____dwellings*
Graphics, Report Preparation = _____
Subtotal

--Travel:
House Rental for Office, Sleeping
@ $500/month x ___ months = _____
Other Per Diem @ $20/day x _____
person-days (see "Direct Labor" above) = _____
Transportation _____ R/T @ _____
Miles x $0.20/Mile = _____
Subtotal

--Consultant Costs:
Labor x 3.0 = _____
ODC x 1.2 = _____
Travel x 1.2 = _____
Total Consultant Costs

_____
* From Seven Rural Lake EIS's

- Total Costs

Local = _____
Consultant = _____
Total

Using the preceding methodology to estimate costs of design and the site specific analysis, assuming 100 houses per mile, results in the following per house costs: 10% replacement--$307/house, 20% replacement--$438/house, and 50% replacement--$720/house. These costs are subject to the assumptions made in the estimating procedure including the per house costs from estimates made in the Seven Rural Lake EIS's. This estimating procedure should be adapted to accommodate local costs and assumptions and should be revised when actual field data become available.

C. COST EFFECTIVENESS ANALYSIS IN SMALL WASTE FLOWS AREAS

Results of cost-effectiveness analyses are not altered by omitting costs common to all alternatives. Such is the case when comparing two sewered alternatives each using different treatment processes. For example, if the decision to sewer the service area has been made, the outcome of the cost analysis will not change if collection and transportation costs are excluded, whether or not the sewers are included. Often when two sewered alternatives with varying proposed routes for the transmission lines are compared, common costs such as house sewers are omitted without affecting the results of the analysis. However, when one of the alternatives being compared includes on-site treatment, all costs associated with both alternatives must be included. Collection, transportation and house sewers, for example, cannot be ignored without affecting the results of the analysis because these costs are not common to both alternatives. When comparing centralized and decentralized alternatives, an accurate cost-effectiveness analysis can only be made if all costs for each alternative are considered.

Tables IV-C-1 through IV-C-3 have been prepared to aid in cost-effectiveness analyses when centralized and decentralized alternatives are compared. The tables list factors which should be evaluated and costed out where applicable for inclusion in the cost-effectiveness analysis. Table IV-C-1 lists factors that are common to both centralized and decentralized facilities. Table IV-C-2 is divided into two parts, on-site and cluster system alternatives, each of which lists factors to be considered for decentralized systems. Table IV-C-3 includes items that should be costed for inclusion with centralized systems. Other costs specifically associated with individual alternatives should be included where appropriate. The guidelines for cost-effectiveness analyses in Appendix A of the Federal Register (40 CFR 35) 27 September 1978 should be followed when preparing a cost-effectiveness analysis.

The level of detail that is required for cost analyses during facility planning depends on the stage of development. A facility plan can be divided into three stages of development: 1) preliminary alternatives, 2) final alternatives, and 3) proposed action. The cost analysis requires increasing detail throughout the progression from preliminary alternatives to proposed action.

In the early planning stages (preliminary alternatives), the cost analysis does not need to include detailed costs. Alternatives need only to be compared on a similar basis so that the comparison is valid. During the final alternatives stage the cost analysis must maintain comparability of the alternatives. More attention must be given to significant cost assumptions and items than is required at the preliminary alternatives development stage. For the proposed action of a facility plan the level of detail should be relevant to the detail of technology selection allowed before beginning Step 2 or Step 2 and 3. For example, if a decentralized alternative is proposed and technology selection is estimated on the basis of a limited sanitary survey,

TABLE IV-C-1. COSTS TO BE INCLUDED IN COST ANALYSES FOR CENTRALIZED AND DECENTRALIZED FACILITIES

- Flow reduction devices (installation and operating)
- Implementation of flow reduction program
- Energy savings due to flow conservation
- Abandonment of existing septic tanks and/or wastewater treatment plants
- Equipment replacement
- Administrative costs (billing, clerical, etc.)
- Legal
- Design and construction inspection (engineering)
- Contingencies
- Interest during construction

TABLE IV-C-2. COSTS TO BE INCLUDED IN COST ANALYSES FOR DECENTRALIZED FACILITIES

- On-Site Alternatives
  - -- Implementation of small waste flows district
  - -- Operation of small waste flows district
  - -- Site-specific analysis
  - -- Septage pumping, hauling, treatment, and disposal
  - -- On-site treatment units (capital and O&M)
- Cluster System Alternatives
  - -- House sewers
  - -- Collection system (capital and O&M)
  - -- Land (treatment site and rights of way)
  - -- Monitoring wells (capital and O&M)
  - -- Relocation of displaced people due to project
  - -- Field exploration (hydrogeological studies)
  - -- Income from crop production
  - -- Site preparation: grading, seeding, floodproofing, fencing, etc.
  - -- Site piping
  - -- Electrical (if any)

TABLE IV-C-3. COSTS TO BE INCLUDED IN COST ANALYSIS FOR CENTRALIZED FACILITIES

- House sewers
- Collection system: sewer lines, manholes, pump stations, force mains, rights of way, operation & maintenance, etc.
- Treatment facilities: pumping, unit operations, land, sludge treatment, handling and disposal, capital and O&M costs
- Start-up costs - operator training
- Income from energy recovery or crop production
- Miscellaneous structures: administrative buildings, laboratory, shops, garage facilities, storage buildings, etc.
- Site preparation, grading, seeding, floodproofing, fencing, etc.
- Site piping
- Electrical
- Instrumentation
- Chemicals
- Contractor mobilization
- I/I-SSES work

the costs should reflect the uncertainty present. On the other hand, if the technology selected is not expected to change prior to preparation of bid documents, the costs should reflect this in the detail presented.

Because of the uncertainties involved in costs of alternative on-site treatment systems and other small waste flow technologies, consideration should be given to increasing the contingency or giving centralized technologies a 10-5% preference. Care must be exercised however, so that viable SWF options are not eliminated because of conservative estimates. Cumulative conservative estimates of technologies, failure rates, and unit costs might throw the cost effectiveness analysis out of balance and should therefore be avoided.

D. ECONOMIC ANALYSES OF FLOW REDUCTION DEVICES AND PROGRAMS

1. INTRODUCTION

The economics of flow reduction devices and programs can be analyzed in several ways. Three methodologies are discussed below, including: (1) cost-effectiveness analysis, (2) cost-benefit analysis, and (3) homeowner's analysis. Table IV-D-1 describes these methods and their use. The cost-effectiveness analysis will be described in detail. The other two methods will be described and compared to cost-effectiveness analysis.

TABLE IV-D-1. DESCRIPTION AND USE OF METHODS FOR ECONOMIC COMPARISON

Method of Analysis	Description	Use
Cost-effectiveness analysis	Monetary comparison of all costs (water supply and heating, wastewater disposal, etc.) for a community with and without a flow conservation program	Community-wide evaluation for alternative selection and construction grants application
Cost-benefit analysis	Similar to cost-effectiveness analysis but expanded to include non-monetary costs as well as monetary costs	Community-wide evaluation when non-monetary costs may be determinate
Homeowner's analysis	Comparison of all monetary costs (water supply and heating, wastewater disposal, etc.) that directly affect the individual homeowner	To measure economic incentives for homeowners to install flow-reduction devices

2. COST-EFFECTIVENESS ANALYSIS

a. Costs

Cost imposing elements of a flow reduction program are given in Table IV-D-2. Note that the costs are not affected by the types of water supply or wastewater disposal facilities present in a community.

TABLE IV-D-2 COSTS OF A FLOW CONSERVATION PROGRAM

Capital Costs	Purchase and installation of flow reduction devices Public information (education) materials and distribution Purchase and installation of water meters
Operating and Maintenance Costs	Reading and maintaining meters Operation of flow reduction devices Continuing educational services

b. Benefits of Flow Reduction

Expected benefits of a flow reduction program are given in Table IV-D-3. These benefits vary depending on the presence of central water and wastewater systems. The benefits listed are followed by an abbreviation indicating the circumstances in which the benefit applies according to the following key: PW = private water supply, PS = private sewage disposal, CW = central water supply, and CS = central sewage collection and treatment.

TABLE IV-D-3. BENEFITS OF A FLOW-CONSERVATION PROGRAM

Capital Benefits	Smaller and/or delayed expansion of water treatment and distribution facilities (CW)
	Smaller and/or delayed expansion of wastewater treatment and transmission facilities (CS)
	Smaller and/or delayed replacement of on-site treatment system (PS)
	Smaller and/or delayed replacement of pump for well (PW)
Operating and Maintenance Benefits	Reduced pumping for water distribution (CW)
	Reduced pumping for wastewater transmission (CS)
	Reduced pumping for private wells (PW)
	Reduced hot water heating (PW, CW)
	Reduced water treatment for private supplies (PW)

c. Central Water and Wastewater Systems

Capital Benefits. Calculation of capital benefits involves computing the costs of items listed in Table IV-D-3 expected without flow conservation and subtracting the expected costs computed using reduced flows. In the case of central water and central wastewater systems, the flow projections with and without flow reduction may yield different sizing or staging schemes. Figures IV-D-1 through IV-D-3 (adapted from INTASA, 1980) show the possible changes: change in staging period only (Figure IV-D-1), change in size only (Figure IV-D-2), and change in both staging and size (Figure IV-D-3). If flow reduction estimates result in only minor adjustments to flow projections, the planned staging and sizing may not change significantly.

Operation and Maintenance Benefits. Annual pumping costs for water and wastewater pumping stations can be calculated for conditions with and without flow reduction. The difference in these annual costs can be considered an annual benefit and converted to a present worth for a 20-year period for comparison with other present worth costs and benefits. Additional operating and maintenance benefits that could be considered include reduced chemical costs for both water and wastewater treatment. Other flow-related costs may be applicable in specific situations and may be quantified.

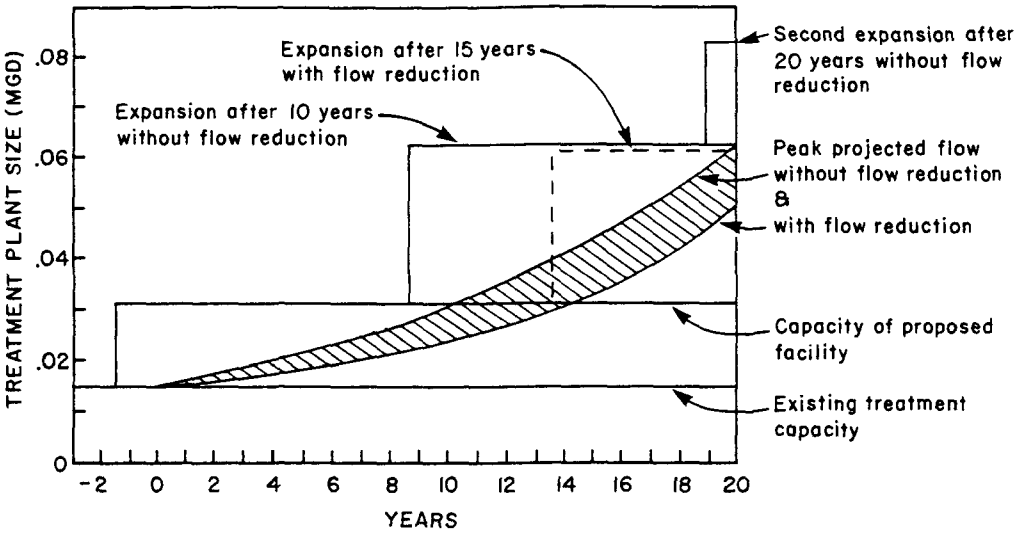

Figure IV-D-1. Change in staging period of treatment facilities as a result of flow conservation.

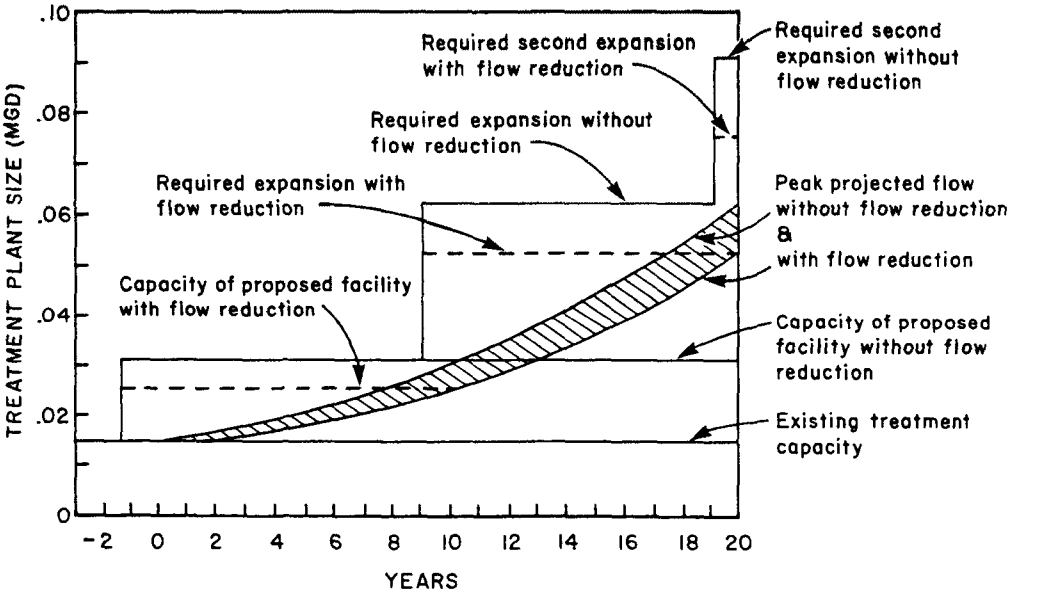

Figure IV-D-2. Change in size of treatment facilities as a result of flow conservation.

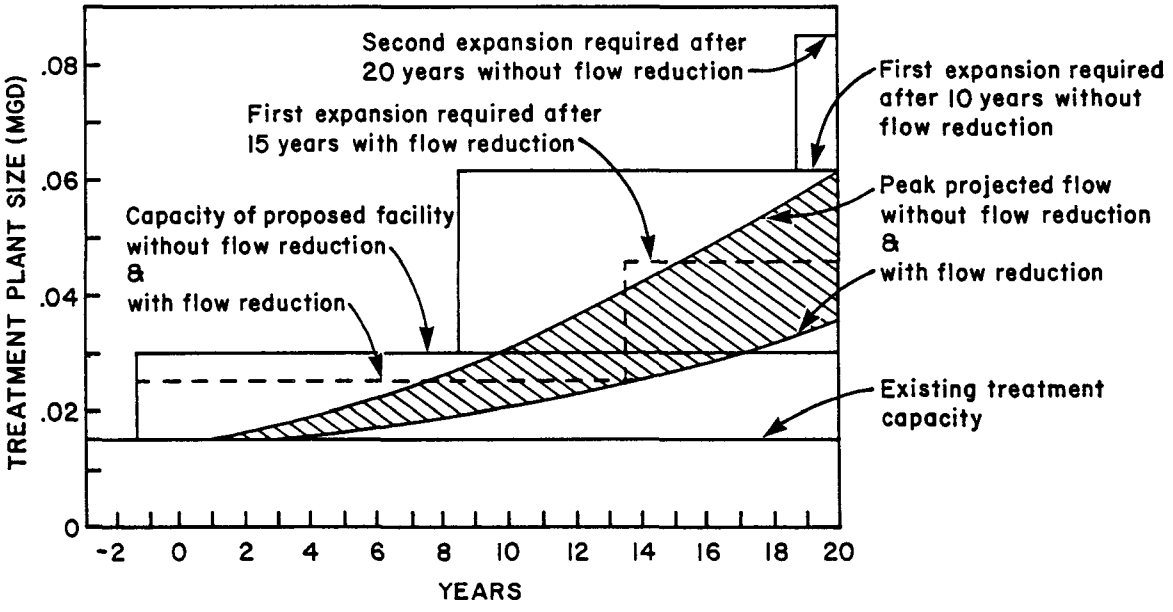

Figure IV-D-3. Change in staging and size of treatment as a result of flow reduction.

Energy Benefits. Flow conservation devices that reduce the amount of hot water required have another monetary benefit to the consumer, that of reduced energy costs. This benefit can be calculated by determining the amount of hot water saved each year and multiplying this by the cost required to heat the water. The following example illustrates this point:

Assumptions:

1. Low-flow shower head saves 3 gpcd x 4 people/house = 12 gpd x 365 days/year = 4,380 gallons/year.

2. Shower water is heated from 55°F to 105°F (ΔT = 50° F).

3. Specific heat for water = $\frac{1 \text{ Btu}}{1\text{b}^{\circ}\text{F}}$

4. 1 kwh = 3,413 Btu.

5. Cost of electricity is $0.04/kwh.

Annual Energy Savings = (4,380 gallons/year) x $\frac{8.34 \text{ lbs}}{\text{gallon}}$ x $\frac{1 \text{ Btu}}{1\text{b}^{\circ}\text{F}}$ (50°F).

Annual Energy Savings = 2.0×10^6 Btu's/year or 589 kwh/year.

Annual Cost Savings = $23.55. (Note that this is a cost savings and not a water heating bill which would include other hot water uses and heater efficiency).

Present Worth at 7-1/8% for 20 Years = $23.55 x 10.4919 = $247.

Depending on the type of energy source, costs can be calculated from the amount of Btu's saved per year (natural gas = 1019 Btu/CF, Heating oil = 5,825,000 Btu/Barrel). The cost-effectiveness guidelines (Appendix A, paragraph 6(d)) allow for the cost of natural gas to be inflated at a rate of 4% per year, which should be accounted for if this type of energy source is used for hot water heating. Inflating the cost of natural gas will increase the energy benefits realized as a result of flow conservation. Energy benefits can be calculated in this manner from the individual homeowner's perspective as well as for the cost-effectiveness analysis or cost-benefit analysis. The community-wide analyses, however, should use the marginal cost of energy for the computation instead of the cost charged to consumers. The marginal cost is the cost required to produce an additional unit of energy. The cost charged to consumers is not the same as the marginal cost because the consumer cost reflects the cost of producing all units of energy and not that of producing only an additional unit. Marginal cost is less than consumer cost partially because no change in amortized capital costs would result from changing production by a single unit. Marginal costs used in calculating community benefits would more accurately represent community savings than consumer costs would. In Section D.4 of this Chapter, the analysis is considered from the homeowner's perspective. In this case, the consumer cost of electricity is used because this is the cost which governs the homeowner's decision.

d. Individual Water and Wastewater Systems

Wastewater Systems. In the case of on-site wastewater treatment, three options exist to provide an estimation of the benefit due to flow conservation. The first two methods can be used for cluster system drainfields as well as for on-site treatment systems. The first method is to calculate the amount of drain field reduction allowed by reducing flows and multiply this number by the unit cost of installing the drainfield. For example, a 40% reduction in flows to the drainfield can reduce the field size by 40% (Machmeier, 1979). The total number of square feet represented by the 40% can then be multiplied by $2.00 per square foot (or other appropriate cost) to obtain estimated benefits. This method assumes that O&M costs and salvage values are not changed by flow reduction, only costs associated with the reduced size of the drainfield.

The second method of estimating benefits to on-site systems from flow reduction involves calculating the life of the treatment system in terms of the total quantity of wastewater treated. If the life of an on-site treatment system can be measured in terms of a fixed number of gallons that it will treat, flow reduction can be assumed to extend the system's life by reducing the annual flow treated. Benefits can be calculated using this method by computing the present worth for the system with and without flow conservation. The present worth of the system with the longer useful life will be lower (all other factors remaining equal) because of the higher salvage value, as shown in the following example:

Assumptions:

1. New conventional ST/SAS (45 MPI/perc rate).

2. Useful life without flow reduction = 50 years.

3. Straight line depreciation.

4. Interest rate - 7-1/8%.

5. Analysis for 20-year period.

6. Maximum flow reduction includes air-assisted toilets and showers.

Present Worth with No Flow Reduction:

Capital cost ST/SAS (construction cost of $2,463 + 20% for contingency, administrative and legal costs)	=	$2,956
PW O&M (@ $20/year)	=	+ 210
PW salvage value [(2,463 x 0.6) x (P/F, 7-1/8%, 20 yrs)]	=	- 373
Total present worth	=	$2,793

Present Worth With Flow Reduction:

First determine life of system in terms of total gallons treated: for a house without flow reduction having a flow rate of 255 gpd (Baker, 1980), the total gallons treated in the 50-year design life would be computed as follows:

255 gpd x 365 days/year x 50 years = 4,653,750 gallons.

With maximum flow reduction, only 95 gpd (Baker, 1980) would be produced yielding a design life as calculated below:

(4,653,750 gallons ÷ 95 gpd) ÷ 365 days/year = 134 years.

After determining the life of the system, the present worth analysis is conducted in the same manner as above for no flow reduction. Note that the system cannot really be expected to have a useful life of 134 years. The calculation is only made to assign a benefit of flow reduction to septic tank/soil absorption systems.

Capital cost ST/SAS (construction cost of 2,463 + 20% for contingency, administrative and legal costs)	=	$2,956
PW O&M @$20/year	=	+ 210
PW salvage value $[2{,}463 \times \frac{(134 - 20)}{134}) \times (P/F, 7\text{-}1/8, 20)]$	=	- 529
Total Present Worth ST/SAS	=	$2,637

The benefit of flow reduction would, therefore, be the difference between the present worth costs of the ST/SAS with and without flow reduction, or,

$2,793 - $2,637 = $156 present worth benefit.

Note that other costs and benefits must be included in order to make the analysis complete as described in the sections on energy benefits, water supply, and the cost of the flow reduction devices.

The last method of estimating benefits of flow reduction for on-site treatment systems applies to upgrading or replacing existing systems. The method is applicable in cases where on-site treatment options would be feasible only with flow reduction. Without flow reduction off-site treatment such as with cluster systems would be required. The difference between the cost of on-site treatment with flow reduction and the cost of collection, transportation, and treatment off-site can be considered the benefit of flow reduction.

To illustrate this method, a simplified example follows. The costs used in this example are total present worth costs taken from the cost variability study (see Section A of this Chapter). For this example, consider a one mile segment with a density of 50 houses per mile. Of the 50 houses, 5 have severe hydraulic limitations. These 5 houses can be served with dosed alternating drainfields if maximum flow reduction is practiced. Without maximum flow reduction, the entire segment of 50 houses will require conventional gravity collection and transmission lines followed by a cluster system drainfield. Total present worth tabulations are as follows:

On-Site option:

For houses with severe hydraulic limitations:

PW maximum flow reduction (air-assisted toilet and shower)	=	$1564/house

PW ST/SAS alternating drainfield gravity dosed (perc rate = 45 MPI) = $5213/house
= $6777/house

Average PW for 50 homes [(6777 x 5) ÷ 50] $ 678/house

Off-Site Option:

Assume 50 houses @ 57 gpcd and 3.5 people/house

Total flow - 10,000 gpd

Cluster system drainfield, dosing siphon and appurtances	$ 85,000
Conventional gravity collection - from Figure IV-A-4-1, average depth of cut = 8', $4,000/house x 50 houses	200,000
Conventional gravity transmission - average depth of cut = 8', 2,700 L.F., $3,800/house x 50 houses	190,000
Total PW	$475,000
Average PW/house	$9,500/house

Therefore, the present worth benefit to the average house due to flow conservation for on-site wastewater treatment is $9,500 - $678 = $8822.

Note that the numbers considered in the above sample were total present worth numbers including capital, O & M and salvage values. A more detailed analysis could compute these parameters individually.

Water Systems. Flow reduction benefits also exist for private water supply systems. The depth and size of the well would probably not change but the size of the pump might. The calculation of such a capital benefit could be made by sizing the well pump for conditions with and without flow reduction. The difference (if any) between the cost of the two pumps would be the monetary benefit due to flow reduction. For existing systems, replacement pumps might be smaller than would be required without flow reduction in which case the benefit would be the present worth of the difference between the two future costs. Even if capital benefits are small due to the expense of replacing an operating pump, calculations can be made for operation and maintenance benefits such as power for well pumping.

Power requirements for well water pumping can be calculated using the following formula (Bacon, 1980):

$$P = \frac{C \times Q \times TDH}{E},$$

Where P = kilowatts input required.

C = constant = 1.88×10^{-4}.

Q = discharge in gpm.

TDH = total dynamic head (elevation head + discharge pressure + friction losses).

E = efficiency of pump (submersible pump efficiency is usually about 30%).

Power requirements can be converted into annual costs by multiplying the kilowatts required by the annual number of hours of operation and then by the cost of electricity ($/kwh). The following example illustrates the method described for calculating annual costs for well water pumping.

Assumptions:

1. Three hours per day of pumping and discharge in system.

2. Average flow rate: without flow reduction 255 gpd - 1.42 gpm.
   with maximum flow reduction 95 gpd - 0.53 gpm.

3. Well depth = 125 feet.

4. Head loss due to friction is negligible.

5. Delivery pressure is 40 PSI - 92 feet of water.

6. Pump efficiency = 30%.

7. Electricity costs $0.04/kwh.

$$P = \frac{1.88 \times 10^{-4} \times Q \times TDH}{E}.$$

Without flow conservation,

$$P = \frac{1.88 \times 10^{-4} \times 1.42 \text{ gpm} \times 217 \text{ feet}}{0.30}.$$

P = 0.193 kw.

Annual energy required = 0.193 kw x 3 hours/day x 365 days/year.

Annual energy = 211 kwh.

Annual cost = $8.46.

With flow conservation,

$$P = \frac{1.88 \times 10^{-4} \times 0.53 \text{ gpm} \times 217}{0.30}.$$

P = 0.072 kw.

Annual energy = 0.72 kw x 3 hours/day x 365 days/year = 79 kwh/year.

Annual cost = $3.16.

The difference between the two costs presented as a present worth value can be considered the monetary benefit of flow reduction for private water supply pumping. Continuing the example, the benefit would be calculated as follows:

Annual benefit - $8.46 - $3.16 = $5.30/year.

Present worth at 7-1/8% for 20 years = $5.30 x 10.4919 = $55.61.

e. Net Monetary Benefits

For a cost-effectiveness analysis, the net monetary benefits for the community with flow reduction must be calculated. This calculation involves comparing the costs and benefits with and without flow reduction for the community. If the community has some areas with central water and wastewater facilities and other areas which rely on individual systems, the various cost components must be calculated individually and then summed. All costs and benefits should be presented on an equivalent basis such as a present worth for 20 years. The total present worth costs and benefits can be summed in a table such as Table IV-D-4. If the net monetary benefits are positive, the flow reduction program is cost effective. Otherwise, the analysis indicates that flow reduction would cost more than the savings realized for the 20-year design period at the specified interest rate.

TABLE IV-D-4. NET MONETARY BENEFITS FOR A COMMUNITY

	Monetary Costs (Present Worth)	Monetary benefits (Present Worth)
Water supply		
Water heating		
Wastewater treatment		

Net Monetary Benefits = Total Monetary Benefits - Total Monetary Costs.

3. COST-BENEFIT ANALYSIS

a. Monetary Costs and Benefits

For a cost-benefit analysis, the monetary costs and benefits should be calculated as presented in the preceding section (IV-D-2). After the net monetary benefits are calculated, non-monetary factors must be considered.

b. Non-Monetary Costs and Benefits

Environmental, social, political, and institutional effects of a flow reduction program must be considered in a cost-benefit analysis. These effects are often not easily quantified but must be considered as part of the overall evaluation of a flow reduction program. Some of the factors which

should be considered are included in Table IV-D-5. The non-monetary costs and benefits should be evaluated in light of the community's goals and individual situation.

TABLE IV-D-5. NON-MONETARY BENEFITS AND COSTS

- Effects on groundwater supplies (cost or benefit)
- Effects on multipurpose surface water reservoirs -- especially for recreational use (cost or benefit)
- Effects on fish and wildlife (cost or benefit)
- Lower pollutant discharges entering streams (benefit)
- Transactions effects (e.g., changing institutional structures, altering public attitudes) (cost or benefit)
- Inconvenience costs (e.g., possible public dissatisfaction with flow-reducing devices, changes in lifestyle, etc.)
- Additional safety value or value of alternative use gained from no longer operating at the margin of the available water supply (benefit)
- Increased public satisfaction from water-using fixtures (e.g., faucet aerators may reduce splashing, etc.) (benefit).

4. HOMEOWNER ANALYSIS

An analysis of the economic feasibility of flow reduction devices from the individual homeowner's perspective differs somewhat from a community-wide analysis. The homeowner is concerned only with the specific purchase and installation costs of the devices and the resultant reduction in his utility bills.

a. Capital Costs and Benefits

The homeowner's capital costs include the cost of the specific devices and their installation (see flow reduction fact sheets, Chapter I). The costs of public education and water meter installation required for community-wide programs will not be evident to the homeowner since they likely will be amortized in his utility bills. Capital benefits to the homeowner might include a smaller pump for well-water supply or a smaller on-site wastewater treatment system. These benefits can be calculated as described previously in Section IV-D-2.d. In the case of central water and wastewater customers, the capital benefits of flow reduction would only be considered from the standpoint of lower amortized costs which would result in reduced monthly utility bills. The effect of one homeowner reducing flows would not be great enough to produce any reduction in capital costs for central facilities. Therefore, capital benefits for individuals on central water and wastewater systems are not considered for the homeowner analysis.

b. Operating and Maintenance Costs and Benefits

Individual homeowners can compute costs of operating any flow-conservation device by multiplying the amount of electricity required per year (kwh/yr) by the cost paid by the homeowner (¢/kwh). Flow reduction devices that require electricity include air-assisted toilets and showers, incinerating toilets and some composting toilets.

Homeowners benefits from flow-conservation devices include reduced bills for water supply, water heating, and wastewater disposal. Those on central water supply and wastewater disposal systems can calculate savings by mutiplying the number of gallons saved per year (gallon/year) through flow reduction by the cost they pay for water supply and waste disposal per gallon (¢/gallon). Reduced costs for on-site water pumping and water heating can be calculated as described previously in Section IV-D-2.d. The summation of all individual costs and benefits will show the cost incentives for individuals to install flow-reduction devices. As decribed before, the capital and O & M costs should be tabulated as present worth values so that all monetary values are equivalent.

5. DOUBLE COUNTING

Double counting of benefits should be avoided. For example, in the case of central water, if the capital and O & M benefits have been calculated for the utility, no allowance should be made for the benefit of reduced user charges to consumers. Double counting may also occur in cases where combinations of flow reduction devices are used. A pressure reducing valve may save 6.5 gpcd individually and a low-flow shower head may save 3 gpcd individually; however, the combination may be somewhat less than 9.5 gpcd because the shower head may not save 3 gpcd at the reduced presssure. Caution should be exercised in case double counting occurs in other instances.

6. SUMMARY

Economic comparisons of flow reduction devices and community-wide programs can be made with or without including non-monetary costs and benefits. The existence of central or individual water and wastewater system does not affect the ability to evaluate flow reduction programs. In a community with both individual and community systems, the percentage of both must be known. The methods presented consider both the capital and operating and maintenance costs and benefits associated with both flow reduction and conventional water use. An economic comparison can also be made from the individual homeowner's perspective. Energy savings of flow reduction (reduced hot water) are substantial and should not be ignored in any of these analyses.

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CHAPTER V

THE ROLE OF ENGINEERS

A. PROBLEMS OF PROFESSIONAL LIABILITY IN RELATION TO INNOVATIVE AND ALTERNATIVE WASTEWATER TREATMENT TECHNOLOGIES FOR SMALL COMMUNITIES IN U.S. EPA REGION V.

1. INTRODUCTION

In 1977, the United States Congress ordered a major change in the nation's sewage treatment construction grants program. A system of incentives and requirements was established by law to encourage greater use of innovative and alternative technologies--defined loosely as techniques for saving energy and water; recycling water and nutrients using soils, plants, and wetlands as treatment media; planning for decentralization, particularly in rural areas; deploying sewerless technology; and furthering integration of community water, waste, energy, and industrial facilities. Now, almost three years after these legislated changes, progress still falls short of what Congress mandated. During the period October 1978 (when regulations were promulgated) through June 1980, 248 innovative and alternative projects were funded by EPA. Of the $84 million set aside for this purpose in fiscal year 1979, only $27,245,633 or 32% was actually spent. More than 35 states may lose a portion of their allotted FY79 funds for failure to use them. These figures come from briefing papers prepared by the national office of EPA in connection with development of a 1990 construction grants strategy.

Numerous groups and individuals have described various institutional obstacles to deployment of the favored technologies. One such obstacle frequently mentioned is the fear of malpractice liability by engineers and design professionals.

2. APPROACH

The principal objectives of this study were to determine: a) whether and to what extent the fear of liability presented an obstacle to innovative or alternative technology in rural communities, and b) if liability anxiety does constitute a serious obstacle, what steps might be taken to circumvent or overcome it.

The analysis focused on the problems of smaller communities in EPA Region V (Indiana, Ohio, Michigan, Illinois, Minnesota, and Wisconsin). It consisted of two parts: library research into the law of malpractice and a review of relevant literature on small community wastewater problems, with special emphasis on Region V, and interviews with people whose careers are shaped by and help shape the development of this law, particularly as it relates to the management of water and waste in small communities. The people interviewed include alternative technologists, contractors, trade and professional association representatives, municipal and state officials, engineers, insurance company representatives, legislators, and others. This paper presents and discusses the findings of the analysis. A list of the people interviewed, with names, addresses, telephone numbers, titles, and short descriptions of their expertise is available from the U.S. EPA Project Officer. Appendix A, immediately following this section, is a detailed, comprehensive memorandum of law on the issue of designer's liability, with special emphasis on problems of innovative and alternative technologies in small communities. This should be of use to government policy-makers, municipal and construction attorneys, engineers, designers, contractors, and insurance companies.

3. FINDINGS

(1) Fear of liability has indeed discouraged engineers with little training or experience in alternative waste technology from entering the field. This is entirely appropriate. Designing and building simple, low-cost solutions to wastewater problems requires a high degree of specialized knowledge and skill. Engineers who undertake such tasks without the requisite background put themselves, as well as their clients, at risk.

The law of professional malpractice does not typically hold engineers strictly liable for poor results; it holds them liable for poor results caused by their own negligent conduct. Design professionals do not usually guarantee their work. They do guarantee their behavior. One of the easiest ways of establishing negligent behavior is to show that a designer undertook work for which he or she was not qualified. Any professional person who misrepresents his qualifications is liable for damages to those who rely on his misrepresentations to their detriment.

A number of the people we interviewed said that it was not so much a fear of liability that kept conventional engineers away from alternative technologies, as ignorance. A basic policy behind professional malpractice law is to discourage untrained people from undertaking work requiring specialized knowledge. To the extent that malpractice law keeps unqualified professionals out of the alternative technology field, it is working as it should.

In real life, wastewater professionals are rarely wholly ignorant or wholly knowledgeable about particular wastewater technologies. This leads to mixed consequences, as illustrated by the following anecdote related by an official of the Minnesota Department of Natural Resources. In Minnesota, cluster systems, in which effluent from a number of home septic tanks is distributed to a common subsurface disposal field, are an effective and increasingly popular technique for managing wastewater in lakeside communities. One engineering firm with little experience in the field specified a large centralized holding tank alongside a new cluster system, just in case the system failed. A designer with more confidence, experience, and knowledge would probably not have taken this extra precaution, which added greatly to the cost of the system. A number of other people expressed the view that the fear of liability often leads to overdesign of innovative and alternative systems.

Other factors frequently cited as obstacles to alternative technologies were the lack of profit from designing low-cost systems, resistance by government and conventional engineering firms to new ideas, difficulties of dealing with state and local codes (a problem investigated and discussed in another portion of this study), and lack of nearby demonstration projects.

(2) Those people with the most knowledge, experience, and skill in the area of alternative technology--while they are concerned about liability--are not deterred by it from practicing their skills. One of the most significant and surprising findings of this study was that those with the greatest expertise in this field generally do business in a way very different from the typical engineering firm. For example, conventional engineering firms restrict themselves to the planning and designing of systems. They generally stay away from actual construction. On the other hand, many successful alternative technology firms are engaged in the business of planning, designing,

constructing, operating, and maintaining wastewater systems. Some also serve as equipment suppliers and distributors. Many have never done EPA-funded work. Most have never participated in negotiating EPA contract terms. These facts suggest that joint ventures between alternative technology companies and firms with experience in dealing with EPA should be a lucrative mode of doing business.

As will be shown later, private insurance programs developed to protect conventional engineering firms are of questionable value to alternative technology companies.

(3) Design professionals who recommend wastewater systems that turn out to be inordinately expensive, or inappropriate in other ways, and who fail to give competent consideration to less expensive alternatives are today especially vulnerable to lawsuits and judgments. In the past, engineers working on EPA-sponsored projects ignored innovative and alternative technology. Standard engineering practice in most places called for conventional sewage collection systems leading to conventional treatment and discharge sewage factories. The Clean Water Act Amendment of 1977 and the regulations that followed one year later have changed the standard of good engineering practice. Engineers must now by law give competent consideration to a number of different approaches, including decentralized and sewerless alternatives, land application systems, energy saving systems, and so forth. Engineers are not liable for making the right choice, but they are liable for negligent failure to investigate the array of choices, for failure to disclose risks and benefits, and for actual or implied misrepresentation as to their ability to analyze alternatives. Thus, an engineer whose expertise is confined to conventional-type systems should review the facts carefully before dismissing or recommending unconventional alternatives.

The Clean Water Act of 1977 creates a liability dilemma for conventional engineers involved in choosing technologies. If they disregard alternatives and recommend conventional systems, they may in effect become guarantors of the systems they advocate. We discovered one small town whose residents pay an average of $700-$800 a year for sewer service. This was the result of an advanced waste treatment system designed by the town's engineer. It now appears that a small-scale alternative system would have resulted in substantially lower costs. The engineer did not and was not qualified to analyze or disclose this alternative. In our judgment, the engineer is liable to the town, not for the poor design of the AWT--he did an adquate job designing the AWT--but for failing to investigate properly the small-scale alternatives. In general, if conventional systems should fail, as many do, and the municipality can show that an alternative system would have been cheaper and more reliable, the municipality need only prove that the engineer had little training or experience in alternative technology and that he failed to reveal this fact with sufficient candor, in order for the municipality to recover damages. An engineer who is unfamiliar with innovative and alternative technologies and who wishes to avoid this additional risk has four courses of action open to him. First, he can avoid work involving technology choice, a process that takes place in Step 1 of the construction grants process. Second, he can educate himself about the new technologies. Third, he can hire someone with experience in this field. Fourth, he can enter into a joint venture with a firm qualified by experience and training to investigate and design innovative and alternative solutions. This last approach would be advantageous to the

alternative technology firms as well, many of whom lack the capital or experience to do large-scale, EPA-funded projects.

(4) To avoid liability, and the circumstances that gave rise to it, successful practitioners of alternative technology carefully evaluate soil and groundwater conditions before deciding on the right approach. Most alternative technologies use soil as a treatment medium. Firms with ability in this field typically have soil scientists and geologists in key positions. An understanding of groundwater conditions and problems is also essential. Some time ago, an engineering group in Georgia tried to convince the state to require that every land application system design be approved by a professional engineer. This recommendation was rejected when the state realized that it would disqualify every Georgia design firm with experience designing land application systems. Designers should also take careful note of the quantity and characteristics of the wastewater that is to enter the system.

Designers of conventional systems should also study soil and groundwater conditions, particularly in connection with new or corrected sewer lines. An example related to us by a construction attorney illustrates why even conventional engineers should study these issues. An engineer was hired by a municipality to do an infiltration/inflow analysis, as required by EPA. The engineer found that a great deal of groundwater was leaking into the sewer system. He designed a program for correcting the situation that worked very well. It worked so well, in fact, that the water table began to rise, causing extensive flooding and property damage to adjoining land owners. More thoughtful consideration of soil and groundwater conditions would have avoided this litigation-breeding situation.

(5) In general, alternative technologists and the municipal officials who hire them should follow good business and contracting procedures--procedures followed by responsible and financially successful construction industry enterprises. Chief among these include reaching clear agreements (that is, delineating who does what, where, by when, for how much, to whom), keeping the other parties informed of developments, handling changes and change orders properly, and, where possible, providing contingency funds or insurance against risk.

Professional societies, defense attorneys, and insurance companies publish books and give periodic courses on liability avoidance. One especially good reference is Guidelines for Improving Practice: Architects and Engineers Professional Liability, published by the Office for Professional Liability Research of Victor O. Schinnerer & Co., Inc., 5028 Wisconsin Ave., N.W., Washington, D.C. 20016. No such guidebook exists for municipal managers or other potential plaintiffs. In general, municipal managers and attorneys, particularly in small communities, need much more training in the field of pursuing liability claims. One common problem that they face is in dealing with change orders. There are a number of contractors who have perfected the skill of bidding low and then increasing the amount paid to them through skillful filing of a multitude of change orders. Construction attorneys and managers follow with keen interest the stream of correspondence from contractors to clients. EPA should initiate training programs along this line for municipal officials.

A recent proposed Program Requirements Memo from EPA purports to settle when EPA will pay for change orders. An informal committee of consulting engineer associations and general contractors is now reviewing the draft at EPA's request.

(6) To avoid liability, professionals should beware of relying too heavily on technical information supplied by clients. Professionals should be particularly wary of survey information. One of the biggest fiascos in the history of the sewage program was caused in part by the engineer's reliance on aerial photographs supplied by the municipality. The engineering firm was able to avoid liability by putting the blame on the client. Nonetheless, the case, which received a great deal of national publicity, was injurious to the engineering firm's reputation. This is not to say that design professionals should "reinvent the wheel" in every instance, but they should use good sense and judgment in regard to information supplied by others. When a client insists that the engineer use particular information of this sort, the prudent engineer will take care to document the situation.

(7) Skilled alternative technologists insist on close supervision and inspection of actual construction, particularly in small communities. Groups representing conventional engineers and their insurance companies have been lobbying for many years to limit the construction supervision and inspection responsibilities of design professionals. The standard contract for architects and engineers developed by the professional associations no longer even uses the word supervision. Engineers no longer supervise construction. They observe it. The profession has also surrendered its right to stop work.

These changes were in response to judicial rulings that the right to supervise and stop work implied a duty to do so for the protection of construction workers injured at the construction site. According to a leading design insurance company, claims by construction workers and their families constitute about 48% of all bodily injury claims against engineers and architects. Bodily injury claims against architects and engineers, as a percentage of all claims against them, rose from 15.1% during the 1960-64 period to 23.6% during the 1970-75 period. During this time, the incurred loss from these claims rose proportionately less, from 11% to 15%.

While an abdication of supervisory responsibilites may benefit conventional engineers seeking to avoid lawsuits from workers, it appears to be inappropriate for alternative technologists. Every person we interviewed with experience in this field insisted on close supervision of construction. They were particularly concerned in the case of on-site waste management systems with the installation of septic tanks and the construction of subsurface disposal fields. Some actually managed the construction themselves, providing services to clients on a turnkey basis.

Even many conventional engineers are eager to supervise construction because supervision is generally a well-paying part of the job, and because pride of workmanship makes engineers want to see their designs executed properly. The efforts of engineering societies and insurance companies are aimed at getting the benefits of supervision without accepting the responsibilities.

EPA has surrendered to the demands of the conventional engineers and eliminated supervision and stop work clauses from its standard contract. The

Farmer's Home Administration has thus resisted this pressure. This investigation raises the following questions:

- To what extent has lack of construction supervision contributed to the high failure rate of conventional systems?

- Are EPA contract provisions with regard to construction supervision and liability, appropriate to small wasteflow projects, given their lower anticipated accident rate?

- Encouragement of construction supervision for small wasteflow systems would prove their reliability. What changes can EPA make to thus maximize reliability?

(8) To avoid liability, designers should carefully examine the representation, warranties, and instructions of equipment suppliers, particularly in connection with innovative equipment. Problems with equipment are a frequent cause of system failure. Three helpful hints to reduce the number of problems are as follows: (A). Reliance on a manufacturer's representations is not a good defense to a malpractice suit. Engineers should check out information themselves. (B). Warranties on equipment frequently lapse before the equipment is put into the ground, particularly with EPA projects where delays are commonplace. (C). Designers should take care that equipment is used for the purpose and in the manner specified by the manufacturer.

(9) An advantage of decentralized wastewater systems is that failures tend to be small, localized, and correctable. Key characteristics of alternative or appropriate technologies are that they bend before they break, and when they break the results are not usually catastrophic. On the other hand, the number of potential lawsuits is increased. If an engineer designs 500 systems and two fail resulting in lawsuits, his profits could be wiped out.

(10) The measure of damages for failed individual wastewater systems is either the cost of repairing the system or the reduction in market value of the affected property, whichever is lesser. Homeowners and businesses are the most likely potential plaintiffs in malpractice actions against on-site waste management designers and the municipalities who employ them.

(11) Where possible, the operation and maintenance plans for small community systems should include a small contingency fund or insurance policy to cover the cost of repairing a certain small number of failed systems, which can often be anticipated but not pinpointed. This is especially useful for more innovative systems or systems with pumps or other moving parts. Development of insurance or bonding programs to cover the small percentage of failures would help eliminate much of the risk of decentralized systems. This subject should be carefully investigated by EPA, municipal governments, and alternative technology companies.

(12) Establishing who is at fault when a wastewater treatment system fails is very difficult. Some of the people we interviewed felt that establishing liability after the fact is virtually impossible. Others disagree. Some people concluded that the difficulty of establishing fault would be adequate protection for designers. Others felt that compromised settlements of problems was the most prudent course of action. The problem of establish-

ing fault is compounded by the fact that state codes and other standards are often unclear, incomplete, or inadequate.

Municipal clients, engineers, and contractors should keep careful records of planning, design, and construction so as to be better able to determine responsibility.

A note of caution: Courts take a very dim view of construction activities on one property that result in flooding of another property. They will often find engineers strictly liable in cases of this sort.

The difficulty of establishing responsibility has a pardoxical effect. It shields professionals from liability and at the same time encourages a multitude of lawsuits against them.

(13) Sewage treatment systems of all sorts generate a disproportionate number of lawsuits and judgments against design professionals. This trend is likely to accelerate as system failures--particularly failures of conventional systems--become more evident. Other litigation-breeding construction activities are churches, schools, and recreational facilities. Reasons for this trend are the client's lack of sophistication, the designer's difficulties, and delays involved in dealing with committees, multiple funding sources, and governmental bodies.

(14) Alternative technologists should investigate novel insurance programs suitable to their needs. Standard professional insurance programs may be inappropriate for this group for a number of reasons. First, standard programs do not cover design-and-build companies. Second, potential exposure for small-scale alternative systems may be significantly less than exposure in other types of construction. Simplicity and reliability are the hallmarks of alternative technologies. Third, most standard policies are of a claims made, rather than occurrence basis. In other words, the policy will protect you for any claims made while the policy is in effect. The premiums paid by new businesses are really paying for the errors and omissions committed in the past by older businesses. An occurrence policy covers claims made for errors and omissions committed while the policy is in effect. The problem of appropriate insurance coverage for the burgeoning innovative and alternative technology industry should be carefully explored by professional associations and governmental bodies.

APPENDIX A
THE LAW OF DESIGNERS' LIABILITY, WITH SPECIAL REFERENCE TO INNOVATIVE AND ALTERNATIVE WASTEWATER TREATMENT

A. AN EVER-INCREASING LIABILITY

Those professionals concerned with the planning, design, and construction of facilities or buildings have been subject to increasing tort liability in the past three decades.[1] Architects, engineers, and other design professionals[2] are not unique in this respect; the trend in America since 1900 has been to extend the limits of liability generally.[3] In fact, the development of design profesionals' liability, as well as the development of general tort law liability, has markedly dragged behind that of the medical and legal professions.[4]

Nevertheless, architects and engineers, the organizations that represent them, and the industry that insures them have regarded the trend that increasingly subjects design professionals to malpractice suits as a real threat to the viability of professional practice. The protests and actions undertaken as a response to this slowly developing, but unrelenting trend in tort law, has been the subject of much legal comment and research.[5] Speculation that design professionals' creativity and willingness to work with unconventional technology, have been adversely affected by these developments and has resulted in the perhaps less predictable concern of the Environmental Protection Agency and conservationists advocating the installation of innovative wastewater treatment systems. Determining the legitimacy of the design professionals' response to this trend and evaluating its impact on the selection of innovative and unconventional technology is best left to the end of this study. This paper will examine 1) the development and extent of design professionals' liability, 2) the activities or omissions for which liability has been imposed[6], and 3) the conflicting interests that continue to resist and compel the ever-increasing liability of design professionals.

1. SHORT SURVEY OF A LONG HISTORY

Claims that arise by operation of law rather than contract form the basis for establishing common law liability. "With few exceptions, the Anglo-American law of tort creats liability only for harmful interference with the interests of others."[7]

Thus, a person is subject to liability in tort only for:

- causing harm to another,
- failing to perform the duty to protect another dependent upon him, or,
- failing to prevent something or someone over which he exercises control from causing harm to another.[8]

Duties are imposed by law to govern the conduct between persons. It is the nature of their relationship that determines the extent of the duties owed by one person to the other. A failure to exercise that degree of skill and care owed another person as a duty results in liability if it results in that person's physical injury or economic harm.

a. Early Exception to the Rule

One of the earliest appearances of what we now know as negligence was in the liability of those who professed to be competent in certain public callings. A carrier, an innkeeper, a blacksmith, or a surgeon, was regarded as holding himself out to the public as one in whom confidence might be reposed, and hence as assuming an obligation to give proper service for the breach of which, by any negligent conduct, he might be liable.[9]

Despite the early recognition of a duty owed by professionals to persons receiving their services, architects and engineers were virtually exempt from liability under English common law. The retribution meted out under ancient codes[10] was not compatible with the advancement of new technologies and construction. The Industrial Revolution was furthered by what has been called "[t]he highly individualistic philosophy of the older common law."[11] It "had no great difficulty in working out restraints upon the commission of affirmative acts of harm, but shrank from converting the courts into an agency for forcing men to help one another."[12] Liability for acts of omission, although slow to be recognized by the courts, also first appeared in cases involving professionals, those engaged in public callings.[13]

Yet, for both active misconduct causing harm and passive inaction or failure to prevent harm, architects and engineers were shielded from liability by legal intepretations and fluctions that worked to advance the needs of infant industry and catered to design professionals as a consequence.

b. Legal Means of Obtaining the Desired End

To make a claim against another for negligence, it is neccessary to establish:

- the existence of a duty or obligation, recognized by the law, which required that person to conform to a certain standard of conduct for the protection of others,
- a failure by him to conform to that standard,
- a sufficient causal connection between his conduct and the injury sustained, and
- that the injury suffered is a type for which damages can be awarded.[14]

In addition, it is necessary to make the claim within the period allocated by law.[15] The burden of proving all the elements of a negligence action is shouldered by the person making the claim.[16] The problematic nature of establishing all the requisite elements is obvious. It was, however, judicial interpretations rather than the nature of the rules that protected architects and engineers from suit for so long.

The English courts often characterized an architect's relation to owners and contractors as one of arbitration. Consequently, decisions made by an architect were considered quasi-judicial in nature and exempted the architect from liability to the owner.[17] More often and for a much longer period of time, however, the scope of an architect or engineer's liability was limited

by the courts' characterization of a design professional's duty as limited to those with whom the architect or engineer had entered a contractual relationship.

Duties or obligations are incurred by a design professional upon entering a contract. For the breach of a contractual duty an architect or engineer was civilly liable only to the party owed that duty, the party in privity of contract. Without privity of contract, one could not sue.[18] In common law England, architects, engineers, and builders were immune from civil liability to third parties, those lacking privity, who were injured as a result of negligence in design or construction.[19] For their negligence, architects, engineers, and builders were liable only to the contracting party; that is, the owner of the structure or facility who employed them.

Expanding the Scope of Liability. The doctrine of privity was developed in the law to restrict what was perceived to be a limitless liability[20] encountered by enterprising individuals, those who took risks calculated to advance their own socially sanctioned interests. The doctrine was applied in America and provided the basis for holding that once a structure was completed by the design professional or builder and accepted by the owner, a negligent architect, engineer, or builder would not be considered responsible for remedying defects.[21] Control shifted to the owner and it was his negligence in maintaining a defective building that was held to be the proximate cause of any injury sustained by third parties.[22] Predictably, the doctrine met with some opposition.

The assault on privity is widely recognized to have been launched in 1916 with the MacPherson v. Buick Motor Co.[23] ruling that rendered manufacturers of inherently dangerous and defectively made products liable for injuries sustained by remote users, parties not in privity of contract with the manufacturer. The arguments for sacking the long-standing doctrine of privity and the number of exceptions to the rule increased in the years following the MacPherson decision.[24] Denial of recovery for economic loss and physical injury caused by a design professional's negligence became difficult for the judicial conscience to justify. It was in cases involving physical injury that the courts first conceded to recognize the claims of third parties against design professionals.

In 1953, immunity from third party suits was implicitly abandoned in a case brought against an architect by a workman injured during construction,[25] but it was not until 1957 that privity was explicitly rejected as a defense to claims by third parties injured as a result of a design professional's negligence.[26] Abandonment of privity in cases involving purely economic loss was not long in being widely accepted[27] and exposed the design professional to suits by contractors,[28] sureties,[29] and tenants forced to relocate because of defects caused by an architect or engineer's negligence.[30]

Extending the Duration of Liability. The disconcerting effect of privity's fall for design professionals is obvious. Developments in the 1950s and early 1960s not only widened the range of their liability to include third party suits, but also extended the duration of their liability as well.[31] Legislative enactments that limit the time in which an action can be brought in a court of law are called statutes of limitation. The time allocated by statute usually begins to run when a cause of action accrues.[32] In the past,

courts imposed the limitations set by statutes stringently, even if the injury hadn't manifested itself until after the designated time had run. Medical malpractice claims were frequently dismissed because of the strict stance taken and it was "[t]he obvious and flagrant injustice of such cases which led to the adoption of a series of transparent devices to get around the rule."[33] Courts were prone to find a continuation of the relationship as perpetuating attendant duties. A failure by the professional to discover and correct the damage was termed continuing negligence and silence with probable knowledge was found to be constructive fraud. Fraudulent concealment was considered to toll running of the statute.[34]

But these exceptions, like the ones developed to foil the doctrine of privity's applicability, were eventually abandoned for a more direct attack. Rejecting the interpretation that statutes of limitation begin to run with the occurrence of a negligent act, courts began to apply the discovery rule that recognizes a cause of action to accrue when the injury sustained by the plaintiff is realized or discovered.[35] The time for which architects and engineers could be held liable for damages caused by their negligence was thus alarmingly increased.[36]

2. GENERAL TRENDS

a. Strict Liability

As already noted and evidenced, developments in tort law have tended to expand the scope of design professionals' liability. Tort law has also developed in directions that permit recovery under certain circumstances without proof that the defendant failed to conform to the required standard of care.[37] Liability imposed without a finding of departure from the required standard has been applied to design professionals,[38] but the vast majority of courts have expressly stated that professional services will not be subject to strict liability in tort.[39] Although the majority has refused to hold an architect or engineer to be a guarantor for his services,[40] some have held that undertaking to furnish plans and specifications warrants their sufficiency for intended purpose.[41]

Thus, even though "no court has expressly or directly imposed liability on a designer based on strict liability in tort,"[42] the exception that holds builders strictly liable may well swallow the rule[43] and subject architects and engineers to strict liability for damages suffered in connection with certain professional services. In the meantime, design professionals can expect courts to occasionally apply strict liability in the language of negligence.[44] The circuitous route is an unfortunate one because when the consequences of imposing strict liability and the arguments against it are not directly addressed by a court, they seem less likely to be adequately assessed.[45]

b. Social Change

Developments in the areas of workers compensation and civil procedure have also influenced the expansion of design professionals' liability. As the doctrine of privity gave way, the other traditional barrier to claims by injured workmen against architects and engineers crumbled. The judicial refusal to interpret a design professional's supervisory function as involving

a responsibility for the methods of construction abated as the burden placed on injured construction workers was increased by the enactment of inadequate compensation programs.

Payments under typical workers' compensation programs are limited to medical expenses incurred and to fixed amounts for disabling injuries that threaten income.[46] Many costs such as lost wages, pain and suffering, and disfigurement, which can be recovered under normal tort doctrine, cannot be recovered under workmen's compensation statutes, and the common law right to sue the employer is lost if payment is provided for by statute.[47] The common law right to sue third parties whose negligence proximately caused the workman's injury is retained, however.[48]

Coupled with the increasing tendency to join all possible defendants when filing suit,[49] and the developments in tort law already discussed, the suits against design professionals by injured workmen can be expected to continue as long as the conditions giving rise to them persist.[50]

B. DETERMINING LIABILITY

1. DUTY

It should be remembered that in order to recover on a claim for negligence it is first necessary to establish the existence of a duty or obligation owed by the defendant.[51] "It is no longer in dispute that one who renders services to another is under a duty to exercise reasonable care in doing so, and that he is liable for any negligence to anyone who may foreseeably be expected to be injured as a result."[52]

The principle of foreseeability provided the basis for Judge Cardozo's conceptualization of duty in the famous case of Palsgraf v. Long Island R. Co.[53] Measuring the scope of duty by the scope of risk that negligent conduct entails, Cardozo stated that negligence, like risk, is a relative term.

> Negligence is not a tort unless it results in the commission of a wrong, and the commission of a wrong imports the violation of a right ... The plaintiff must show that the act as to him had possibilities of danger so many and apparent as to entitle him to be protected against the doing of it though the harm was unintended.[54]

Thus, no matter how novel or extraordinary the consequences to the injured plaintiff might be, the defendant is to be held liable, but only for conduct that was negligent, that is, entailed foreseeable and unreasonable risk, as to the plaintiff.

The concept of duty articulated by Cardozo and adopted as the majority opinion can be examined in terms of what it is not. The court rejected the proposition that a duty to refrain from acts that unreasonably threaten others is owed by everyone to the world at large.[55] Duty, as defined by the court, is a function of the relationship between two particular individuals.

a. To Whom

Design professionals have been held liable for failing to satisfy those duties assumed through contract or imposed by law. An architect or engineer's first duty may be to his client,[56] but design professionals have also been held liable for breach of duty to the owner's successor,[57] patron,[58] neighbor,[59] lender,[60] and tenant.[61] Suits by contractors,[62] subcontractors,[63] workmen,[64] sureties,[65] and other design professionals[66] involved in the construction project have been successful. Even claims by vehicle occupants have been recognized by the courts.[67]

b. For What

The extraordinary variety of claimants is rivaled only by the list of specific functions for which a design professional may be held liable.[68] Liability for negligent conduct in the performance of these many functions, however, cannot properly be assessed without first discussing the standard of conduct or care with which design professionals are expected to comply.

2. STANDARD OF CARE

A design professional must possess that degree of learning, skill, and experience ordinarily possessed by those of his profession and must exercise his best judgment in utilizing his abilities to accomplish the purpose for which he was employed. Ordinary skill and care must be used in the performance of his professional duties, and material deviation from the standard renders an engineer or architect liable for negligence. The standard requiring the exercise of that degree of care, skill, and judgment common to the profession was established in 1898[69] for architects and has been adopted as the applicable standard for engineers.[70]

a. Considerations

Any standard established by authority, custom, or general consent is a means by which dissimilar objects or events can be measured. The standard to which design professionals are held is one of conduct, not consequences.[71] In determining whether an engineer's or architect's conduct complied with the standard, courts weigh the same considerations that were balanced when the standard was set. Like a standard measure set by the king's foot, the standard for professional conduct is subject to change. It is not based upon a single, set value.

Negligence is a relative term.[72] It exists when the probability and severity of the risk incurred by a course of conduct outweigh the burden of taking adequate precautions and the usefulness of such activity to society.[73] The method by which negligence is determined, though expressed as an equation, is obviously value-laden. Social policy weighs heavily in the balance. It establishes the values for each of the factors to be considered in the cost-benefit analysis.

The standard of conduct specified for design professionals is a product of social policy as it is expressed by legislative enactments, administrative regulations, professional codes, and custom.

b. Professional Skill and Knowledge

By undertaking work requiring the services of a professional, an architect or engineer will ordinarily be understood to hold himself out as having a standard minimum of special knowledge and ability. Even if rendered gratuitously, the services of a professional must be performed with the degree of skill and learning commonly possessed by members of the profession in good standing.[74] If an engineer or architect represents himself as a specialist or as having less than ordinary skill and knowledge, and the client accepts his services with that understanding, the standard is modified accordingly.[75]

Many states, however, have statutes that require a design professional to be certified or licensed to practice. Because the licensing and regulation of design professionals have been under state control, there are great variations in the legislated requirements for practice.[76] Generally, such statutes are one of two types. Practice statutes require professionals to obtain a license in order to practice. Holding-out statutes require engineers and architects to be registered or licensed in order to represent themselves as professionals.[77] Both usually require the successful completion of a specified number of years in college, a written and occasionally oral examination, and a period of candidacy or practical experience. Typically, registration statutes exclude certain types of projects from being subject to its requirements.[78]

Despite differences in the form such statutes take, however, the degree of skill and knowledge a design professional may be expected to possess will tend to be standardized by the existence of these regulating statutes. Failure to possess the necessary knowledge and ability renders a design professional liable for any harm that occurs as a result.[79]

c. Exercising Due Care

A design professional is bound "to exercise such care, skill and diligence as men engaged in that profession ordinarily exercise under like circumstances."[80] Failure to do so is taken to be evidence of negligence and subjects the design professional to liability for any damage caused as a result.[81] Where the proper course is open to reasonable doubt, however, an engineer or architect will not be held liable for an honest mistake of professional judgment.[82]

Like circumstances has been translated to mean same locality.[83] Because the degree of care required is that ordinarily used by the profession rather than the common law standard of reasonable care,[84] the testimony of experts familiar with conditions similar to those in the locality must be relied upon in order to assess the conduct of an engineer or architect.[85] However, the problems encountered by those attempting to prove medical malpractice claims with experts unwilling to testify against their colleagues[86] have not existed, to any comparable extent, for those alleging negligence on the part of design professionals. This appears to be due to the fact that a design professional's conduct is related to a greater number of regulated industries and interests. In addition, the nature of an architect's or engineer's practice necessitates the use of contracts that may provide the most stringent and detailed means of assessing a design professional's liability.

3. STANDARD MEASURES

a. Contracts

Although design professionals can guarantee their services, it is not usually done.[87] In the absence of an express agreement, the law does not imply any warranty of perfection for plans[88] and the design professional is not considered to be an insurer that the work of others will be done properly in all respects.[89] Basically, contracts can require that a design satisfy the owner or a more objectively determined performance standard.

As previously noted, the lack of a direct contractual relation between parties to a suit has been held not to be a defense.[90] In Illinois, lack of privity has expressly been rejected as a defense to a tort action and foreseeability has been adopted as the test for determining liability.[91] In some jurisdictions, privity may still provide a design professional with some protection from suits alleging economic rather than personal injury. An Indiana appellate court decision[92] rejected a contractor's claim for damages sustained by his reliance on a soil engineer's erroneous estimate of the amount of dirt to be excavated and the amount of fill required on a construction project because no contractual relationship was established between the contractor and engineer. Although the decision has not been overturned, in the fifteen years since its passage not one case has cited it as precedent.

When not explicitly rejected, other tort doctrines have been adopted to permit recovery in the absence of privity. Misrepresentation of fact[93] subjects a design professional to liability for any harm sustained by those who reasonably relied on the information.[94] Although courts rarely consider a contract between an owner and design professional to be intended for the benefit of a third part,[95] a surety has been allowed recovery for loss caused by an architect's failure to properly supervise construction as required by his contract with the City of St. Louis.[96] The doctrine of third party beneficiary, which allows an action to be maintained on a contract that manifests an intent to benefit another not party to the agreement, has also been used successfully by construction workers.[97]

b. Statute

Compliance with the standard of conduct established by statute is generally considered part of the standard of care common to the profession and is implied if not stated in a contract. Mere compliance with legislated enactments, however, does not determine due care was taken.[98] Statutes merely establish the minimum amount of care to be taken; circumstances establish the amount of care that ought to have been taken.[99]

Violation of a statute can constitute conclusive evidence of negligence, that is, negligence per se,[100] but generally, violation of a statute is merely taken to be prima facie[101] evidence of negligence.[102] The policy adopted by a state is usually provided by legislative enactment,[103] but the violation of certain statutes, such as workmen's safety acts, which set standards to protect a particular class, can give rise to an absolute liability to which

contributory negligence is no defense.[104] Civil, as well as criminal liability, may be expressly provided for violation of a statute, but neither has any effect on the imposition of tort liability unless the statute explicitly denies it.[105]

Violation of applicable municipal ordinances by design professionals frequently serves as the basis for judicial determinations of negligence.[106] If no excuse for noncompliance is recognized, the duty imposed by the ordinance is absolute and, therefore, nondelegable.[107] In St. Joseph Hospital v. Corbetta Constr. Co., Inc.,[108] an architect was held liable for specifying non-fire retardant paneling to be used in the construction of a new hospital contrary to city code requirements. The Illinois Court of Appeals held that the architect was legally required to know city building code requirements and was not excused from liability by 1) the owner's agreement to have the paneling installed, 2) his own refusal to issue final certificate of payment to the contractor who installed it, or 3) the manufacturer's representation that the paneling was not fire rated when, in fact, it had been fire tested at 17 times the maximum permitted by the code.[109] In Fidelity & Casualty Co. of N.Y. v. J.A. Jones Constr. Co., a building ordinance that required every owner of specified construction operations to employ a registered architect or engineer to provide full inspection and supervision of the project with the authority to stop work was cited as the basis for holding a design professional liable for the death of a workman caused by inadequate shoring.[110]

Before any legislative enactment or administrative regulation can be adopted as the controlling standard of care, however, it must be established as applicable by the plaintiff who must demonstrate that the statute or regulation was designed

- to protect a class of persons to which the plaintiff belongs,
- to protect the particular interest that was threatened,
- to protect that interest from the type of harm that occurred, and
- to protect that interest from the particular hazard that caused the harm to occur.

Thus, a statute designed to protect construction workers from physical injury caused by inadequate supervision of construction methods is obviously inapplicable to an action by an owner for economic loss suffered as the result of a design professional's failure to accurately estimate a project's cost. The statute must be applicable to both the parties and conduct involved in a particular case.[112]

c. Regulations, Codes, Custom, and Introduction

Violations of applicable administrative regulations generally do not have the same force and effect as the violation of a substantive legislative enactment. Nevertheless, such violations are generally considered evidence of negligence[113] and can determine negligence as a matter of law.[114]

The standards established in codes or manuals issued by government bodies or private associations are commonly allowed as evidence of negligence but

deviation from such standards will not be considered conclusive evidence of negligence.[115] In Wallner v. Kitchens of Sara Lee, Inc.,[116] an expert witness was permitted to use the American Safety Standards Institute Code to establish the negligence of an industrial engineering firm and the industrial engineering division of the bakery for the installation and use of a conveyor lacking the guards that should have surrounded the machine's moving parts. A worker whose hand had been caught in the machine was granted recovery.

There are, however, several limitations on the use of professional codes. The National Electric Safety Code, issued by the Federal Bureau of Standards, was rejected as inadmissible in Mississippi Power & Light Co. v. Whitescarver,[117] because it had been offered in evidence without proof that the Code's standards were generally accepted by industry experts. And a pamphlet issued by a private organization was not permitted to be introduced as evidence in Hackley v. Waldorf-Hoerner Paper Products,[118] because standards issued as advisory material without the force of law are not admissible as evidence on the issue of negligence.[119] Technical codes and manuals are of limited use in establishing standards of care because they present hearsay problems.[120] This is due to the nature of recommended standards that only represent the preferences of a particular interest's research and evaluation.[121] If used in conjunction with expert testimony, however, they may be considered explanatory evidence serving a function similar to that of custom.[122]

Evidence of custom or usage may be introduced to determine the existing standard of care but is not conclusive.[123] In addition, custom that violates or conflicts with an applicable statute cannot vary the standard established by statute or justify its violation.[124] The same holds for company rules or policy and instructions provided by a manufacturer. Departure from standards set by unofficial sources does not, in and of itself, constitute negligence.[125]

VIOLATION OF CLEAN WATER ACT

C. AREAS OF LIABILITY

A design professional's liability varies with the type of project in which he is involved and the scope of his activities. There are, however, four major areas of claim exposure for architects and engineers. A design professional may be considered liable for negligence concerning any of the four possible areas his work may involve:

1) technology selection,

2) design,

3) construction, or

4) operation and maintenance.

The specific functions performed by a design professional for his client are a matter of contractual agreement. Commentators have differed on which area of professional activity exposes design professionals to the greatest liability,[126] but all acknowledge the greatest number of claims to be lodged by the engineer's or architect's client.[127]

Third party claims are usually asserted against the owner who then cross-claims against the design professional. Most third party suits involve a contractor's claims that the engineer or architect has increased construction costs by 1) making false or inaccurate estimations of the quantities involved in the project, 2) providing defective design plans or specifications, 3) designating construction procedures that cost too much, or 4) refusing to approve the work, materials, or equipment installed by a contractor.[128]

1. SELECTION OF TECHNOLOGY

In the preliminary stages of an engineer's or architect's work there are three recognized areas of potential liability. Design professionals have been found liable for erroneous cost estimates, failure to warn or inform, and delay in the delivery of plans and specifications.

a. Inaccurate Estimates

Where the estimate provided by a design professional is exceeded by the project's actual or probable cost, the design professional's right to compensation depends on:

- whether the cost figure was expressed as an approximation or guarantee,
- whether the client ordered changes that increased the cost,
- whether objections were waived by the client's acceptance of the design professional's services, and
- whether the architect or engineer suggested reasonable revisions to scale down the project's cost.[129]

Because estimating cost is one area in which designers commonly make express representations concerning the results of their work, "courts have been willing to hold that failure to meet agreed-upon costs may constitute a breach of contract rather than negligence."[130]

In Wetzel v. Roberts,[131] an architect was denied recovery of his fee for preparing plans to remodel a bowling alley because the construction bid received "was so much greater than the amount...planned upon, and so in excess of the limitation of expense communicated" to the architect that a contract to build according to the plan was never executed.[132] The standard is one of reasobable nearness to actual cost,[133] and an engineer or architect may be held liable for costs exceeding the estimate in addition to being denied his fee.[134] In Zannoth v. Booth Radio Stations,[135] the Supreme Court of Michigan stated that an architect had an affirmative duty to give his client some idea of the project's final cost regardless of whether his client should have known of its possible cost or not. This duty of disclosure was founded on the relationship between one employed to furnish plans and his employer.[136] Frequently characterized as one of agency,[137] the relationship is considered one of trust and confidence.[138] "Good faith and loyalty to his employer constitute a primary duty of the architect. He is in duty bound to make a full disclosure of all matters, of which he has knowledge, which it is desirable or important that his principle learn."[139]

b. Failure to Inform or Warn

Currently, courts recognize a design professional's duty to inform only in limited contexts. In Seeny v. Dover Country Club Apartments, Inc.,[140] the architect employed by a civil engineering and surveying firm to prepare plans and specifications for construction was held not liable for injuries sustained by a workman when an excavation ditch caved in. Because the architect's supervisory powers were ruled not to have included active control of the manner and method of work performed by the contractor, a duty to warn of soil conditions or potential dangers (considered by the court only for the sake of argument) did not exist.[141] In Loyland v. Stone & Webster Engineering Corp.,[142] however, workmen were allowed to recover from an engineering firm for injuries sustained when a concrete form on a dam construction project broke. The engineering firm that planned and supervised the project was held to owe the employees of the general contractor a duty to exercise ordinary care for their safety and not to expose them to hidden dangers that were known or, through the exercise of reasonable care should have been known to engineers. The plaintiff's claim that failure to warn of any dangers or hazards that might exist constituted negligence[143] was not directly addressed by the court.

When recognized, the duty to warn is considered contractual in nature. Courts focus on contract terms rather than the character of the communication in defining a design professional's duty and determining whether it was breached.[144] Unlike physicians and attorneys, whose duty to inform clients of risks and alternatives has been widely recognized by the courts, architects and engineers have not been held to owe clients a similar duty except in the dicta of two cases.[145] In J. Ray McDermott & Co. v. Vessel Morning Star,[146] a naval architect was held to have a duty to warn the ship's owner of the potentially negative effects of requested changes in design. In Allied Properties v. John A. Blume & Assoc., Engineers,[147] an engineer retained to study the feasibility of a pier for small boats and design it was held not to have warranted the pier's reasonable suitability for use by small craft, but breach of duty to warn was held to be negligence, not a breach of warranty.

An advocate of the legal imposition of a duty to inform notes that "[w]hat professionals actually provide is the ability to define benefits, risks, and alternatives, not any special skills for choosing between them."[148] To be informed is an essential element of the participatory problem-solving process, a process by which clients assume the risks disclosed by professionals in determining which of the available alternatives they choose to take.

Legal recognition of a duty to inform can be expected to result in the profession's attempt to contractually limit their liability by limiting their duty to investigate risks and alternatives. The imbalance of power characteristic of doctor-patient and attorney-client relationships is not typical of designer-client relationships, however, and the bargaining process is likely to reflect this. The 1977 Standard Form of Agreement Between Owner and Architect[149] reflects a greater concern about the information provided clients than the edition issued ten years earlier.[150] It specifies that after reviewing the owner's proposal the architect is bound "to ascertain the requirements of the Project... and to review the understanding of such requirements with the Owner."[151] The architect is also charged to "review

with the Owner alternative approaches to design and construction of the Project."[152]

Actions based on a design professional's breach of a duty to inform or warn are a novel development in tort law by name only. Failure to provide an owner with a project's cost, meet owner's expectations of proper performance, or warn of time required for completion have all subjected design professionals to liability. Increasingly, however, an engineer's or architect's failure to warn or inform is being advanced as breach of a duty owed, not as the result of contractual agreement, but as an obligation imposed by law.[153]

c. Late Delivery of Plans and Specifications

In Edwards v. Hall,[154] an architect suing for his fee recovered it but was held accountable for the damages occasioned by his five-month delay in delivering the plans. Unless expressly specified in the contract, however, courts allow a reasonable time for performance and claimants alleging loss because of a design professional's delay must prove that the delay, even if unreasonable, was attributable solely to the architectural or engineering firm.[155]

2. DESIGN

The work product of a design professional represents the culmination of a creative decision-making process. The performance parameters of a particular design, the basic functional criteria of the structure or facility planned, are determined by the design professional at the commencement of the design process. Each component incorporated in the overall design must then have its performance parameters defined individually and in relation to the whole structure or facility. Each component must be sized, its spatial relationship to other components established, and the material from which it is to be made specified. The engineer or architect communicates this information to the builder in the form of 1) a graphic description in two dimensions such as a drawing, plan, or sketch, 2) a three dimensional representation or model, and/or 3) written specification.[156]

a. Design Defects

Defects in design can be categorized as one of three types. Errors in establishing the sizes or spatial relationships of components primarily structural in fuction, may result in a structure collapsing,[157] settling,[158] or being rendered unfit for intended purpose.[159] Errors in establishing the sizing or spatial relationship of nonstructural components are alleged when the suitability of steps or lack of handrails,[160] the adequacy of ventilation, or the installation of mechanical or electrical equipment are considered defects in design.[161]

b. Failure

Design defects are commonly alleged because the performance parameters sought by an owner have not been met by the completed structure or facility. For legal action to be taken, failure is required. A structure's collapse is a readily acknowledged failure. Malfunctioning facilities and inadequately performing equipment or systems can also be termed failures.

Subsurface waste treatment systems are generally recognized as having failed with the occurrence of any one of the following:

- The system refuses to accept sewage effluent at the rate required by design, resulting in interference with plumbing fixture use.

- Sewage effluent exceeds the infiltrative capacity of the soil resulting in objectionable odors, ponding, seepage, or other discharge of the effluent to the surface of the ground, or surface waters.

- Effluent discharges from the absorption system result in contamination of a potable water supply, groundwater, or surface waters.[162]

The potential health hazard caused by the failure of such systems can subject the owner to legal liability or economic loss[163] and, as a consequence, the design professional responsible for the system's design and/or installation can expect to hear from the owner.

When the design parameters of an innovative technology are in the process of being established as data is collected and the sizing and spacing of design components are still being developed, it is obviously going to be difficult to determine when the performance parameters of a particular system have substantially not been met. When the risks of failure are not yet really calculable and the materials and equipment in the system are relatively novel in design or use, a design professional may well consider the creative approach a short cut through a legal minefield. This need not necessarily be the case,[164] but recent developments concerning the liability of engineers and architects for workers injured in the course of a project's construction have justified the profession's apprehensions of being hit with costs society has not yet seen fit to distribute equitably.

3. CONSTRUCTION

Until recently, the general duty of an engineer or architect to supervise the work merely created a duty to see that the building or facility constructed meet the plans and specifications contracted for. Then, as now, a design professional did not warrant the sufficiency of a contractor's work, but was liable to the owner for failure to exercise that degree of care ordinarily employed to insure a project's completion in accordance with design specifications.[165] In addition, however, the design professional was not considered responsible for the methods of construction used by a contractor and, subsequently, had no duty to make sure the work was done in a manner that did not threaten the lives or safety of the contractor's employees.

The bidding process for construction contracts, workers' compensation provisions, and the widespread use of indemnity clauses made the use of unsafe construction procedures and outdated construction equipment the norm in the construction industry. Design professionals were not the only ones virtually immune from suit. Owners and contractors insured themselves and each other from liability and, not surprisingly, there was little economic incentive for preventing accidents.[166] As a result, thousands of construction workers were killed and hundreds of thousands were injured annually.[167]

a. Supervision

In Illinois, the legislature finally adopted the Structural Work Act[168] that specifically addressed the problem of negligent supervision of construction. It was designed to provide a comprehensive guide for the design, erection, and supervision of structural works.[169] Insuring compliance with statutory provisions was designated to be the responsibility of the person having charge of the work, and willful failure to do so subjects that person to criminal liability.[170] The term having charge of was not defined in the statue, however; that small feat was left to the courts.

An architect's or engineer's right to stop work was first held to make them persons having charge within the meaning of the act in Larson v. Commonwealth Edison Co.,[171] a 1965 decision by the Supreme Court of Illinois.[172] The appellate court opinion in the case of Miller v. DeWitt,[173] and its final resolution by the state Supreme Court in 1967,[174] however, is generally regarded as the landmark decision on the obligations, duties, and liability of design professionals in Illinois. Proceeding on the unstated premise that an architect's power to stop work confers sufficient control to insure more than a contractor's technical compliance with design specifications, the court held that the right to stop work creates a duty to stop work if the supervising architect knew or should have known that the work was unsafe.

Equating a right with a duty is a somewhat unusual practice. It was attacked first by the dissent[175] and later by the Arizona appellate court[176] that rejected the Miller analysis.

The opinion was frequently cited and followed,[177] however. By ruling that design professionals' right to stop work makes them persons having charge under the Structural Work Act, the court simply made an old rule (that one with a statutory duty to provide safeguards or precautions is liable for the failure of an independent contractor to do so)[178] applicable. The Miller decision toppled the last of the long standing obstacles to actions by injured construction workers against design professionals[179] and precipitated revisions in the contracts between design professionals and owners.

b. Inspection

Prior to the Miller ruling, contracts with owners frequently provided for the design professional to make periodic visits to observe the progress of construction and to sample check the quality of a contractor's performance. The right to stop work was considered a mere facet of the architect's or engineer's supervisory powers and was never intended to subject the design professional to liability for failing to exercise it.[180] The standard form contract issued by the American Institute of Architects (A.I.A.)[181] for its members commonly incorporated the power to stop work in an architect's general supervisory powers for leverage in obtaining the contractor's compliance with contract specification.[182]

Following the Illinois Supreme Court decision in Miller, however, the General Conditions of the Standard Form of Agreement Between Owner and Contractor ceased to address the architect's right to stop work. Only the "authority to reject work which does not conform to the Contract Documents"[183]

was granted an architect and all subsequent revisions since 1970 have remained silent regarding an architect's right to stop work.[184]

Omission of the right to stop work language may be all that is required to render an architect a mere inspector of construction methods instead of a supervisor responsible for insuring that only safe methods be used,[185] since the architect's right to reject defective materials and workmanship and require its correction has been held not to be the equivalent of a right to stop work;[186] and similarly, the owner's right to stop work upon an architect's refusal to grant certification is not enough, in itself, to impose liability on the architect.[187] Unlike the right to stop work, the right to inspect work is considered to be merely ancillary to the design professional's supervisory powers.[188] The McGovern v. Standish[189] ruling, in which these distinctions were made, further elaborated that to establish a person as one in charge of construction operations, he "must have been in charge of the particular operations which involved the violation from which the alleged injury arose."[190]

Obviously, requiring a direct connection with the particular aspect of the construction that violated the statute and resulted in a worker's injury would severely limit the use of the Miller ruling by construction workers suing the few design professionals who may have retained the right to stop work in their contracts. The inequities of holding design professionals liable for the use of dangerous construction methods have not been resolved by efforts that leave injured workers uncompensated. Soon after the McGovern decision, the imposition of liability under the Structural Work Act was held not to require that a person be in direct charge of the particular operation in which the injury arose if the person is in charge of overall work for the project under construction.[191] So the burden to insure only safe construction methods are used, continues to rest, provisionally, with the design professional.

With few exceptions, owners continue to enjoy the advantage of the low bid and indemnification by the contractor whose workers' compensation payments alleviate the need to reduce profits in order to use safer (that is, more costly) construction methods.[192] The owner, with economic control of the project's construction, and the contractor, with physical control of the construction process, are given little incentive to prevent injury to workers under the present system of allocating liability. The design professional, arguably the least capable of insuring workers' safety,[193] continues to be held liable for injuries sustained by workers during a project's construction.

c. Refused or Improper Certification

Design professionals are subject to suit for specifying construction methods considered excessive by a contractor's trying to operate within the limits set by their low bid for the construction project. Altering the original design specifications, refusing to approve completed work, recommending a contractor's replacement, or failing to insure one contractor's work does not damage another's have all been alleged to have caused a contractor economic loss.[194]

More commonly, however, design professionals are sued for certifying work that allegedly fails to meet the performance parameters of the design contracted for by the owner.[195] Negligent recommendation or selection of an

incompetent contractor can subject a design professional to suit by the owner,[196] and improper certification of payments to a contractor can render an architect or engineer liable to the owner's surety as well as the owner.[197]

D. IMPOSING LIABILITY

1. BURDEN OF PROOF

It should be remembered that in order to recover from an architect or engineer the plaintiff must not only have established that a legal duty or obligation required the design professional to conform to a certain standard of conduct and that he failed to conform to that standard.[198] A causal connection between the design professional's negligent conduct and the injury sustained must also be established by the plaintiff. The injury must be a type for which damages can be awarded and all four elements must be proved more likely than not;[199] that is, the burden of persuading the trier of fact is upon the plaintiff. If there is insufficient evidence to prove the existence of a fact essential to establishing negligence, it is a court's duty "to remove the issue from the jury,... direct a verdict for the defendant, or even to set aside a verdict once rendered."[200] If the evidence is sufficient to establish negligence, but the jury remains unconvinced (does not consider it more likely than not), the plaintiff cannot recover from an architect or engineer.

2. CAUSATION

Proximate or legal cause[201] exists if the defendant's negligent conduct is a substantial factor in bringing about the plaintiff's harm and is not excused by law.[202] In determining whether a design professional's negligent conduct was a substantial factor in causing the plaintiff's injury, courts consider:

- the number of other contributing factors and the extent of their effect in producing the injury,
- whether the design professional's conduct created a continuous and active series of events up to the time of the harm, and
- the lapse in time between the design professional's negligent acts or omissions and the plaintiff's injury.[203]

a. Intervening and Supervening Cause

"If the defendant's conduct was a substantial factor in causing the plaintiff's injury, it follows that he will not be absolved from liability merely because other causes have contributed to the result."[204] Where simultaneous or concurring acts or omissions occur, the defendant's conduct is considered to be a cause of the plaintiff's injury if it was "a material element and substantial factor" in bringing it about.[205]

Only when other, intervening causes that follow or concur with the defendant's conduct actively and independently produce the plaintiff's harm is the defendant relieved of liability.[206] Such causes are considered supervening or superseding causes.

3. HARM

Recovery for both physical and economic harm may be awarded plaintiffs suing design professionals. It has been suggested[207] that generally, courts consider cases involving physical injury using the principle of products liability[208] and that the principle of foreseeability is relied on in cases involving economic loss. Generally, mental anguish unaccompanied by physical injury is not compensated by law unless the defendant is guilty of outrageous conduct, but the trend in tort law increasingly recognizes emotional harm as compensable.[209] The mere threat of harm, however, is not enough to bring suit.[210] The harm must be realized to be recognized by the courts.[211]

4. STATUTE OF LIMITATIONS

Most harm caused by another's conduct is concurrent or closely related in time with the other's negligent act or omission. When defective products or services are involved, however, a substantial period of time may pass between the negligent conduct and resultant injury. Generally, the period of time in which a negligence action must be brought to be allowed recovery commences when the injury is sustained.[212] The intervening period of time is considered only in determining proximate cause.[213] This policy and the nature of design professionals' work created a problem for victims of a building's collapse, owners of a wastewater treatment system that failed, and others sustaining harm due to a structure or facility's design defects. Did the harm occur at the time the faulty plans and specifications were completed or at the time the design's defects were discovered?

a. Discovery v. Occurrence

Statutes of limitation are generally two-three years for tort actions and five-six years for actions based on breach of contract.[214] The most basic theory or principle explaining their existence is that

> statutes of limitation are statutes of repose, the object of which is to suppress fraudulent and stale claims from springing up at great distances of time and surprising the parties or their representatives when all the proper vouchers and evidences are lost or the facts have become obscure from the laspse of time or the defective memory or death or removal of witnesses.[215]

As noted earlier,[216] courts have tended to strictly apply the limitations set by statute.[217] In Board of Education v. Joseph J. Duffy, an Illinois court determined that a cause of action for malpractice against a design professional accrues at the date of occurrence rather than the date of discovery.[218] The security provided design professionals by fixing their liability within set parameters was short lived, however.

In 1969, the discovery rule[219] was first applied by the Illinois Supreme Court in Rozny v. Marnal.[220] The decision in Auster v. Keck[221] reversed the Duffy ruling in effect and with the ruling in Society of Mt. Carmel v. Fox[222] issued the following month, established Illinois as a date of discovery state. Florida, California, Iowa, and Pennsylvania also applied the discovery rule against design professionals[223] who confronted, as a result, the possibility

of indefinite liability; not only for the duration of the owner's life or even their own, architects and engineers were liable for the life of the structure of facility constructed according to their design.

b. Limiting Duration

Anticipating these developments, the American Institute of Architects (A.I.A.), the National Society of Professional Engineers (N.S.P.E.), and the Associated General Contractors (A.G.C.) began to lobby for limiting the duration of liability imposed on their members. In the span of two years (1965-67) thirty jurisdictions had enacted or amended statutes of limitation specifically for architects, engineers, and builders.[224] Forty-six jurisdictions eventually adopted such statutes,[225] and many were patterned after the Model Act endorsed by the A.I.A., N.S.P.E., and A.G.C.[226] Each sets a definitive period of years (ranging from 4 -20 years)[227] that begins at the time of completion or acceptance of the work, after which no civil actions against an architect, engineer, or builder may be brought. The limitation can serve as an offensive as well as defensive legal tool. One insurance authority pointed out that a design professional suing for his fee would do well to wait until the statute of limitations barred any possible cross claims the owner might have.[228]

Even this powerful effort to secure limitations for the liability of design professionals, however, was not completely successful. As one commentator has observed,

> [a] jurisdiction's legislative enactments alone fail to indicate fairly the duration of liability. In assessing the effect of the statue of limitations in a given jurisdiction, due regard must be accorded plaintiffs to proceed on a given theory or different theories, and statutory or judicial rules concerning the effect of the continuing nature of a tort and of plaintiff's knowledge or lack thereof.[229]

The enacted statutes are not only affected by a plaintiff's ability to bring an action under contract theory or delay the running of a specified period of time until the harm is discovered; challenges that the statutes are unconstitutional have caused five to be repealed.[230]

Most, like the Illinois statute,[231] limit actions against persons designing, planning, supervising, or building a structure. Some, like those of Michigan[232] and Ohio,[233] restrict their application to architects and engineers licensed by the state. Others[234] extend their coverage to include surveyors. Most expressly exclude persons in actual possession or control, that is, owners. The distinctions gave rise to challenges that equal protection provisions were violated by the statutes. In Skinner v. Anderson,[235] the Illinois Supreme Court held that granting the architect, engineer, and contractor immunity while denying similar protection to the owner and supplier of defective products violated a section of the state constitution that provides: "The General Assembly shall not pass local or special laws...granting to any corporation, association or individual any special or exclusive privilege, immunity or franchise whatever."[236] A ruling based on similar considerations was made in Kallas Millwork Corp. v. Square D. Co.[237] in which the Wisconsin statute of limitations was struck down as unconstitutional.[238]

The Wisconsin Supreme Court also noted that the statute appeared to be unconstitutional because it deprived potential plaintiffs of remedy for a wrong recognized by the law of the state.[239] Because the statutes enacted or amended under the A.I.A., N.S.P.E., and A.G.C. lobbying effort speak only in terms of substantial completion of the construction, they conflict with the basic premise underlying statutes of limitation that requires that the plaintiff suffer harm and thereby acquire a right of action.

> In barring actions that have yet to accrue, these statutes are unique, since a statute of limitations proceeds on the theory that a right of action exists, with the limitation defining the period for pursuit of judicial redress. For a statute to bar an action which has not yet accrued is anomalous; such a statute does not merely limit the remedy, but bars the right of action from ever coming into existence.[240]

Thus, the statutes enacted as a response to professional prodding, operate as statutes of limitation only for those claims against design professionals that accrue before the statutory period has run. Claims that accrue after the statutory period has run are, in effect, barred completely. The plaintiff's right of action is, in fact, never even allowed to come into existence.

Nevertheless, the effectiveness of the statutes has withstood the majority of constitutional attacks,[241] and the repeal of a particular statute does not necessarily leave a design professional unprotected. Two years after the Skinner ruling, the Illinois General Assembly enacted a statute[242] that declares that their absence of harm in the six years following the structure or design's complexion is taken to be presumptive proof of reasonable care in design and/or construction. There has been no litigation concerning the statute, so the nature of the presumption is unclear. It has been suggested, however, that unless the presumption is irrebuttable, the Act is void of any real meaning since it would merely serve to codify already accepted principle.[243]

What prompts design professionals to push such obviously unbalanced enactments limiting the time in which actions can be brought against them? The lengthy passage of time hinders the plaintiff as much as the design professional in producing evidence, and it is the plaintiff who bears the burden of proof. It is up to the plaintiff to establish that a defect in design or construction, rather than improper maintenance, was the cause of injury. And, as one critic notes,[244] professional liability insurance exists and is readily available. It is not, however, the nature of design professionals so much as the nature of their liability insurance[245] that appears to be responsible for the self-serving efforts to unfairly limit the duration of their liability.

E. EFFECTS OF LIABILITY

1. COSTS

Because the design and construction of buildings and facilities involves such a complex and varied number of contracts (between the owners, architects, consulting engineers, general contractors, materialmen, and subcontractors),

"the normal lawsuit will involve myriad co-defendants, third party defendants, fourth party defendants, counterclaims, gross claims and third party claims."[246] The testimony of expert witnesses and the often technical nature of the subject matter add to the complexity of a case. "As a consequence, litigation is bulky, complex, time consuming, and expensive."[247]

If held liable, in addition to the cost of defending a negligence action, an engineer or architect can lose his professional license temporarily, or for good.[248] Many state licensing agencies have the power to revoke, suspend, or refuse to renew a design professional's license to practice for serious misconduct.[249] An architect or engineer suing for his fee may not recover it. Adverse publicity, possible criminal prosecution, and substantial damage awards also accompany the imposition of liability.

2. INSURANCE

The statutory and common law reforms to expand tort law remedies were not the source of the dramatic increase in malpractice actions against design professionals. Tort law develops as a response to the socio-economic forces evolving and in conflict. The population explosion in the 1950's resulted in a concomitant acceleration in the construction industry. An increase in the demand for architectural and engineering services corresponded with the industry's growth. The increased use of design professionals was matched by an increase in claims against them and when the demand for insurance to cover professional practice constituted a lucrative market, the industry cashed in.[250]

Prior to the mid-fifties, Lloyd's of London was the only company to issue errors and omissions policies for design professionals.[251] Negotiations begun in 1955 with the A.I.A. and N.S.P.E. by Vincent McKerrow, Executive Vice-President of the Continental Casualty Co. (formerly a Lloyds underwriter) and Victor O. Schinnerer, head of a Washington, D.C. based insurance agency bearing his name, produced a marketable policy in 1957 that provided a comprehensive insurance program for the design professional.[252]

Opposition from state regulatory agencies made it difficult for the insurance industry to obtain policy rate hikes as desired. The agencies were challenged with limited success by the industry and the Model Act limiting the duration of engineers and architects liability was subject to constitutional challenge, so the public-law arena was abandoned for that of private-law "where freedom of contract and negotiation of terms offered a greater degree of flexibility"[253] for profit margins. The development of policy exclusions[254] and restrictive provisions such as the claims made clause are the result of the insurance industry's effort to "eliminate some of the cost increases as well as to impose self-limitations on its area of liability exposure."[255]

a. "Claims Made" and "Occurrence Policies"

There are two types of policies available for insuring professional services. The claims made policy covers those errors and omissions reported to the insurance company during the policy period no matter when they were committed. The occurrence policy covers those negligent acts or omissions that occurred during the policy period no matter when they are discovered.

The claims made policy is preferred by the insurance industry because there is a greater degree of certainty in forecasting losses and computing premiums. This is due to the limits placed on the duration of the industry's liability and the lessening of inflation's importance. In addition, increases in jury awards, statutes of limitation, and rates of inflation can all be passed on to policy holders with a corresponding increase in premiums. Because there is less risk, however, premiums for claims made policies are lower than those for occurrence policies.

Nevertheless, the industry's ability to estimate those errors and omissions that occurred prior to existing policies presents a problem for young professionals who pay rates computed to cover the cost of long-practicing professional's policies. And because only those claims made during the policy period are covered, retiring professionals are forced to pay until they drop. "Thus the purchase of a claims made policy is in a commercial sense an addictive device. The design professional is forced to continue to repurchase the claims made policy on a yearly basis in order to cover for potential past errors not currently reported and for present errors which may constitute claims in the future."[256]

The claims made clause in professional malpractice policies has been subject to increasingly intense legal dispute[257] and criticism. The industry acknowledges the claims made policy to be less attractive to consumers,[258] but considers the device a last resort to deal with the problems created by time-delayed actions against design professionals for services rendered. One commentator predicts that legal invalidation of claims made clauses will almost certainly cause all or most insurers to stop underwriting professional services,[259] because while "competition among insurers may in theory sustain the life of the occurrence policy in professional malpractice insurance, in practice the underwriting of professional malpractice occurrence policies has been virtually eliminated."[260] The cost incurred by society's technological advance will not be borne by the insurance industry. That much is certain.

3. INDEMNITY

A design professional's malpractice liability can be eliminated by requiring indemnification from the party employing him. Chances are better than average, however, that the design professional himself will be asked to indemnify one or more parties to the construction contract.[261] To indemnify or hold harmless is to assume an obligation to "make good any loss or damage another has incurred or may incur by acting at his request or for his benefit."[262] An indemnity clause does not preclude a victim's recovery for injuries sustained, it merely reallocates the cost of the victim's claim from the tortfeasor to the indemnitor.[263]

Subcontractors are commonly required to indemnify general contractors who frequently agree to hold the owner harmless in order to acquire a construction contract. The highly competitive nature of the construction industry and inadequate workers' compensation awards are the primary sources for the widespread use of indemnity clauses. In the interest of free enterprise and contract, the courts have declined to curtail the use of such clauses in construction contracts. Most jurisdictions, like Wisconsin, uphold the clauses as valid and not against public policy,[264] although they are strictly construed.[265] Other jurisdictions, like Minnesota, apply the more lenient test of party intent.[266]

"[T]he courts have refused to uphold such agreements, however, where one party is at such obvious disadvantage in bargaining power that the effect of the contract is to put him at the mercy of the other's negligence."[267] In Illinois, the legislature enacted a statute[268] that declares that contracts or agreements (whether public or private) that indemnify or hold harmless another person from that person's own negligence void as against public policy. Although the act limits its application to contracts for the construction, alteration, repair, or maintenance of structures, agreements to render design services for a structure seem certain to be subject to the restriction.[269] The Illinois statute is a unique exception, but a more pervasive limitation on the use of hold harmless clauses is provided by the insurance industry. Insurance carriers generally exclude coverage for liability assumed under indemnity clauses.[270]

F. ALTERNATIVES

Traditional defenses for malpractice or breach of contract actions are available to design professionals and provide some protection against a claimant's recovery,[271] but the effects of being subject to such widespread and ever increasing liability are not countered by existing legal doctrines. It has been observed that

> [t]he most important effect of design professionals' liability, and yet the hardest to document, is the chilling effect on creativity. The ...engineer who pioneers a new technology is particularly vulnerable... This makes him a prime candidate for a malpractice action when his creation is a financial disaster because his work is not that of the more traditional practitioner. It is here that the law may have its most subtle yet most egregious effect.[272]

Law, however, does not operate independent of the economic and political aspects of a society. The obstacles to implementing innovative technology, it is submitted, are not products of legal doctrine. The law is only a component in a complex institutional framework that has not been structured to accomodate innovations that conserve rather than produce value.

1. DISINCENTIVES

The implementation of innovative technologies is a process that involves behavioral as well as technical factors. Commonly either neglected entirely or viewed as obvious but unmanageable, the behavioral components of

- uncertainty,
- habit,
- political alignments, and
- official response

are suggested to be the greatest obstacles to innovative change.[273]

Uncertainty in the area of innovative wastewater treatment is due largely to purely technical considerations. Incomplete knowledge of the extent to which new technologies will result in improved water quality and the difficulty of estimating a particular design's performance in varying on-site

conditions can be resolved only by time and testing. Uncertainty has also been needlessly created, however, by the failure of available institutional mechanisms to encourage public acceptance, adopt progressive standards,[274] and reward innovative efforts.[275]

Habit reduces the burden of making decisions. It is an efficient means of reducing risk and, in the short run, costs. Spending time and resources to investigate, test, get official approval, and then sell a new technology is expensive. Consequently, there is a tendency among engineers, state regulatory agencies, and others to do things a certain way because it has always been done that way. One commentator has noted that

> [c]onsulting engineers are content to use outmoded wastewater design concepts because their engineering fees are based on a percentage of construction costs,[276] and because federal and state regulatory agencies virtually refuse to allow new ideas to be developed because it upsets their status quo, and forces them to revise their arbitrary engineering manuals.[277]

The political alignments that result in the ultra-conservatism[278] that has dominated local and regional agencies have not been provided economic incentive to change until recently. The Clean Water Act's allocation of funds for research, construction, and back-up systems for innovative and alternative wastewater treatment systems[279] may well result in the reallocation of liability for professional services involving on-site systems.

2. MUNICIPAL LIABILITY

Like private owners, municipalities are typically exempt from bearing the full cost incurred by their undertakings. Municipal immunity from suit has a long history. In England, sovereign immunity was first extended to a municipality because in addition to the fear of an infinity of such actions and the lack of any precedent to grant recovery, there existed no corporate funds out of which a settlement could have been made in 1798.[280] The policy was adopted in America on the basis of several arguments, none of which were sound.[281]

Municipal corporations act as both subdivisions of the state endowed with the power and responsibility of government, and corporate bodies with special and local interests not shared by the state. This dual nature has prompted the courts to distinguish municipal functions as either governmental or proprietary in nature. Although the function of laying out sewers and administering sanitation laws is regarded as governmental and does not render a municipality liable to suit, the construction of public improvements is regarded by most courts as proprietary or ministerial in nature and subjects the city to tort liability.[282] The maintenance of sewer systems is treated by the majority of courts as a ministerial function as well.[283]

The obviously highly artificial distinctions began to give way as the criticism lodged for over a century began to erode municipal immunity.[284] The Wisconsin Safe Place Statute[285] was the basis for a holding that rendered Milwaukee liable for injuries sustained on public property.[286] More direct repudiations of municipal immunity were made in Illinois,[287] California, and Michigan.[288] Indiana, Minnesota, and Wisconsin[289] soon followed suit.

Risk is a relative term. Like liability it can be assumed. If informed of the risks and alternatives, a city may assume the liability for potential injury posed by a hazardous undertaking. In Pittman Construction Co. v. New Orleans[290] a city's decision to accept an engineer's plans to use floating rather than supported slabs in order to save money was held to be an assumption of the risk involved. Neither the engineer nor contractor was liable for damages that resulted.

Municipal powers can be interpreted to create corresponding duties. Issuance of building permits has been argued to have amended the relevant code or waived the necessity of a design professional's compliance,[291] but the issue of whether local authority can be held liable if permits are issued for the construction of a building or facility that does not comply with statutory requirements and later causes injury has not been resolved. A municipality's decision to construct or issue a permit for an innovative wastewater treatment system after weighing the costs and benefits should not subject the design professional to any greater liability than is encountered in more traditional practice. Indeed, with its capacities to self insure and the assurance of federal funds for back-up systems, the design professional's scope of liability should be greatly diminished.

FOOTNOTES

1. See Goldberger, Architectural Malpractice Suits Reported Increasing 20% a Year, N.Y. Times, Feb 12, 1978, at 1, col. 1.

2. Surveyors are also described as design professionals, but the term is used here primarily in reference to architects and engineers whose similarities justify the application of court rulings and legal commentary on one to the other profession. See, Liability of Design Professionals--The Necessity of Fault, 58 Iowa L.R. 1221 note, (1973).

3. See 2, Harper and James, The Law of Torts §12.3, at 751-52 (1956); W. Prosser, The Law of Torts §4, at 19-21 (4th ed. 1971) (hereinafter cited as Prosser).

4. See, Liability of Design Professionals - The Necessity of Fault, 58 Iowa L.R., at 1221 note, (1973).

5. See, Selected Materials on Architects, Engineers and Contractors, 34 the Record 247 (1979).

6. Most of the cases and statutes are those of Illinois, Indiana, Michigan, Minnesota, Ohio, and Wisconsin. The six states comprise the Environmental Protection Agency's Region V, which is the site of much precedent-setting litigation and trend-setting legislation in the area of design professionals' liability. Major developments in the law that have occurred elsewhere are cited, but an attempt has been made to limit such references. As a result, some of the cases chosen are not the most frequently cited for precedent. Certainly, case references should not be taken to be exhaustive of all those on point.

7. Seavy, Principles of Torts, in Essays on the Law of Torts 75 (1964).

8. Id.

9. Prosser §28, at 139.

10. The Code of Hammurabi provided that: If a builder builds a house for a man and does not make its construction firm, and the house which he has built [collapses] and causes the death of the owner of the house, that builder shall be put to death. (§229). If it [causes] the death of a son of the owner of the house, they shall put to death a son of that builder. (§ 230). If it [the collapsing of the house] [destroys] property, he shall restore whatever it destroyed, and ... he shall rebuild the house which collapsed from his own property [at his own expense]. (§ 232). If a builder [builds] a house for a man and does not make its construction meet the requirements and the wall [falls] in, that builder shall strengthen that wall at his own expense. (§ 233). R. Harper, The Code of Hammurabi (1904) cited in Liability of Design Professionals --The Necessity of Fault, 58 Iowa L. R. at 1221 note, (1973).

11. Prosser §56, at 339.

12. Id.

13. Id.

14. Prosser § 30, at 143, 144.

15. Id.

16. Id., § 28, at 142.

17. See Guilian, Liability of Architects and Engineers, 35 Tenn. L. Rev. 9 (1967).

18. Winterbottom v. Wright, 10 M. & W. 109, 113, 152 Eng. Rep. 402, 404 (Exch. Ch. 1842).

19. Stevenson v. Watson, 4 C.P.D. 148 (1879).

20. See, Liability of Architects and Engineers to Third Parties, 53 Notre Dame L. 306 note (1977).

21. Ford v. Sturgis, 14 F. 2d 253, 254 (D.C. Cir. 1926).

22. Id. See also Daugherty v. Herzog, 145 Ind. 255, 44 N.E. 457 (1896); and Sherman v. Miller Construction Co., 90 Ind. App. 462, 158 N.E. 255 (1927).

23. 217 N.Y. 382, 111 N.E. 1050 (1916).

24. See Prosser § 85 (2d ed. 1955) and § 93 (3d ed. 1964).

25. Paxton v. Alameda County, 119 Cal. App. 2d 393, 259 P. 2d 934 (1953).

26. Inman v. Binghamton Housing Authority, 3 N.Y. 2d 137, 164 N.Y.S. 2d 699, 143 N.E. 2d 895 (1957).

27. See, Liability of Architects and Engineers to Third Parties, 53 Notre Dame L. 309 note (1977).

28. M. Miller Co. v. Central Contra Costa Sanitary District, 198 Cal. App. 2d 305, 18 Cal. Rptr. 13 (1961).

29. Aetna Ins. Co. v. Hellmuth, Obata, & Kassabaum, 392 F. 2d 472 (8th Cir., 1968).

30. A.E. Investment Corp. v. Link Builders, Inc., 62 Wis. 2d 479, 214 N.W. 2d 764 (1974).

31. See Comment, Recent Statutory Developments Concerning the Limitations of Actions Against Architects, Engineers, and Builders, 60 Ky. L.J. 462 (1972).

32. Prosser § 30, at 144.

33. Id.

34. Prosser § 30, at 144.

35. See Ronzy v. Marnal, 43 Ill. 2d 54, 250 N.E. 2d 656 (1969) and Auster v. Keck, 31 Ill. App. 61, 333 N.E. 2d 65 (1975), rev'd on other grounds 63 Ill. 2d 485, 349 N.E. 2d 20 (1975).

36. See Comment, Limitation of Action Statutes for Architects and Builders -- Blueprints for Non-Action, 18 Catholic L.R. 361 (1969).

37. See, Liability of Design Professionals -- The Necessity of Fault, 58 Ia. at 1225 note (1973): Prosser §§ 78, 98, 99; 2 Restatement (2d) of Torts note §§ 402A, 351-353 (1965).

38. Schipper v. Levitt & Sons, Inc., 44 N.J. 70, 207 A 2d 314 (1965) held that one who designed, built, and sold mass-produced homes could be held strictly liable for defect in water distribution system which caused excessively hot water to issue from bathroom taps and resulted in injury to child of purchaser's lessee.

39. See Taff, A Defense Catalogue for the Design Professional, 45 UMKC L.R. 75 (1976); Laukkanen v. Jewel Tea Co., 78 Ill. App. 2d 153, 222 N.E. 2d 584 (1966); Prosser § 104.

40. See Aetna Ins. Co. v. Hellmuth, Obata & Kassabaum, 392 F. 2d 472 (8th Cir., 1968); Allied Properties v. John A. Blume & Assoc., Eng'rs., 25 Cal. App. 3d 848, 102 Cal. Rptr. 259 (1972); Bayne v. Everham, 197 Mich. 181, 163 N.W. 1002 (1917); and Prosser § 97.

41. Broyles v. Brown Eng'r Co., 275 Ala. 35, 151 So. 2d 767 (1963); Niver v. Nash, 7 Wash. 558, 35 P. 380 (1893); Hill v. Polar Pantries, 219 S.C. 263, 64 S.E. 2d 885 (1951).

42. Liability of Design Professionals -- The Necessity of Fault, 58 Ia. L.R. at 1228 note (1973).

43. Prosser, § 104, at 682.

44. See Jones v. Boeing Co., 153 N.W. 2d 897 (N.D. 1967) in which the court held an architect-engineer could not avoid liability for water damage to nearby property even though there was no competent proof that the design could or should have been different or that a reasonably prudent architect or engineer would have designed it differently.

45. The policy reasons articulated for applying strict liability to manufacturers provide some support for its application to design professionals, but most commentators consider the arguments supporting its desirability to be outweighed by those against imposing strict liability for design professionals' services. One such commentator concluded a thorough examination of the issue with the submission "that such expansion of liability is unwarranted under any circumstances, and should certainly not be imposed as a result of conclusory judicial pronouncements in place of rigorous examination of the policy considerations involved in such an extension." Liability of Design Professionals -- The Necessity of Fault, 58 Ia L.R. at 1249 note (1973).

46. 1 A. Larson, the Law of Workmen's Compensation § 1.10 (1972).

47. Id.

48. Id.

49. See 59 Am Jur. 2d Parties § 112 which observes that to avoid multiplicity of suits and to permit, so far as possible, the adjudication of all issues in one action, most, if not all, jurisdictions, have enacted statutes or rules of practice declaring that any person who 1) has or claims an interest in the controversy adverse to the plaintiff, or 2) is a necessary party to the complete resolution of a question raised by the controversy may be made a defendant. See J. Sweet, Legal Aspects of Architecture, Engineering, and the Construction Process § 34.03 (1970).

50. See H. Philo, Revoke the Legal License to Kill Construction Workers, 19 DePaul L.R. 1 (1969).

51. Prosser § 30, at 143.

52. Prosser § 140, at 679 (emphasis added).

53. 248 N.Y. 339, 162 N.E. 99 (1928). Mrs. Palsgraf, a passenger standing on a station platform, was injured by the fall of scales that were upset by an explosion some distance away. The explosion resulted when a package containing fireworks was dropped by a passenger boarding a train. A railroad employee had pushed the passenger in an effort to help him board the crowded train. Mrs. Palsgraf alleged that the employee's action was negligent.

54. Id. at 347, 162 N.E. at 102 (emphasis added).

55. Id. at [ ], 162 N.E. at 103 (This view was expressed in Judge Andrew's dissenting opinion).

56. See J. Acret, Architects and Engineers: Their Professional Responsibilities, Owners § 9.2 (1977).

57. Id, Remote Owners § 9.3.

58. Id. Patron § 9.6.

59. Id. Neighbor § 9.5.

60. Id. Lender § 9.7.

61. Id. Tenant § 9.4; see also Schipper v. Levitt & Sons, Inc., 207 A. 2d 314 (1965) in which designer-builder was held liable to child of purchaser's lessee.

62. J. Acret, Architects and Engineers: Their Professional Responsibilities, Contractor § 9.12.

63. Id. Subcontractor § 9.13.

64. Id. Workman § 9.11.

65. Id. Surety § 9.14 (Even the surety's indemnitor has successfully filed suit against the design professional).

66. Id. Architect § 9.15.

67. Id. Vehicle Occupant § 9.9.

68. Twenty-four functions which may be performed for a client as a matter of contractual agreement are listed in Liability of Design Professionals -- the Necessity of Fault 58 Iowa L.R. at 1229-30 note.

69. Chapel v. Clark, 117 Mich. 638, 76 N.W. 62 (1898).

70. Cowles v. Minneapolis, 128 Min. 452, 151 N.W. 184 (1915).

71. Prosser § 31, at 146.

72. See nn. 54 and Cardozo's discussion to which it refers.

73. The Hand Formula, as this calculation for determining the existence of negligence is commonly called, was articulated by Judge Learned Hand as a quantitative test by which the determinative factors in any given situation could be considered in deciding whether certain conduct had been negligent. See United States v. Carroll Towing Co., 159 F. 2d 169 (2d Cir. 1947); Moisan v. Loftus, 178 F. 2d 143 (2d Cir. 1949); and Cases and Materials on Torts, ch. 4, pp. 102, 110 (C.O. Gregory, et. al., ed. s 1977).

74. Prosser § 32, at 161; Restatement (2d) of Torts § 299A, at 73.

75. Prosser § 32, at 163.

76. See J. Sweet, Legal Aspects of Architecture, Engineering and the Construction Process § 31.01 (1970).

77. Id. §31.02.

78. Id. §31.03. In Illinois, architects are required to obtain a certificate of registration which is issued only upon satisfaction of the educational requirement. See Ill. Rev. Stat. ch. 10½ (1975), commonly known as the Illinois Architectural Act.

79. Prosser § 32, at 163.

80. Cowles v. Minneapolis, 128 Min. 452, 151 N.W. at 185 (1915).

81. See T. Herbst, Malpractice: The Design Professional's Dilemma, 10 John Marshall J. 287, 288 (1977) and Restatement (2d) of Torts § 299A.

82. Prosser § 32, at 162.

83. Paxton v. Alameda County, 259 P. 2d at 938. But see Restatement (2d) of Torts § 299 A(g) at 75, 76, (1965).

84. But see Miller v. DeWitt, 37 Ill. 2d 273, 226 N.E. 2d 630 (1967) which, according to the court's interpretation of his contract, held an architect was bound to exercise reasonable care; and Aetna Ins. Co. v. Hellmuth, Obata, & Kassabaum, 392 F 2d at 476 (8th Cir. 1968) which established that an architect whose contractual duties include the supervision of a construction project must supervise the project with reasonable diligence and care.

85. See City of Eveleth v. Ruble, 302 Min. 249, 225 N.W. 2d 521 (1979) in which expert opinion was held to be needed to determine whether a design engineer had been under a duty to ascertain the pressure-bearing capacity of water distribution lines and whether failure to do so made design of safety valves defective.

86. Prosser § 32, at 164.

87. See Standard Form of Agreement Between Owner and Architect on a Basis of a Percentage of Construction Cost (A.I.A. Document B131, Sept. 1963 ed.); But see Rozny v. Marnal, 43 Ill. 2d 54, 250 N.E. 2d 656 (1969) in which a surveyor made an "absolute guarantee for accuracy" for what turned out to be an inaccurate plat.

88. Chapel v. Clark, 76 N.W. at 62; City of Moundsview v. Walijarvi, 263 N.W. 2d 420 (Min. 1978); and Prosser § 32, at 164. But see Broyles v. Brown Eng'r. Co., 275 Ala. 35, 38, 151 So. 2d 767, 770-71, in which the court implied a warranty of fitness for intended purpose in civil engineer's design for housing subdivision drainage plan.

89. Cowles v. Minneapolis, 128 Min. 452, 453, 151 N.W. 184, 185 (1915).

90. See nn. 26 and the discussion of privity to which it refers.

91. See Rozny v. Marnal, 250 N.E. 2d 656, in which a surveyor was held liable for the pecuniary loss caused the purchasers of a home who had relied on his inaccurate plat. Even though the plat was prepared for a real estate developer, as home purchasers the plaintiffs were held to be among a foreseeable and limited class who would be injured by negligence in surveying the property for which the plat was drawn.

92. Peyronnin Construction Co., Inc. v. Weiss, 137 Ind. App. 417, 208 N.E. 2d 489 (1965).

93. See generally Prosser § 105.

94. See note 91; Prosser § 108, at 715; and Restatement (2d) of Torts § 552.

95. See J. Acret, Architects and Engineers: Their Professional Responsibilities, who states such claims are usually dismissed as inapplicable; and C.W. Regan, Inc. v. Parsons, Brinckerhoff, Quade and

Douglas, 411 F. 2d 1379 (1969), which held that under applicable law, a person not party to an agreement could not recover for negligent breach of warranty and therefore denied a contractor's claim against the engineer whose approval of another contractor's faulty work resulted in flooding of the plaintiff's work on a tunnel.

96. Aetna Ins. Co. v. Hellmuth, Obata & Kassabaum, Inc., 392 F. 2d 473 (8th Cir. 1968).
97. See Miller v. DeWitt, 37 Ill. 2d 273, 226 N.E. 2d 630 (1967) that considered the safety provisions of the architect's contract to render the design professional liable for any injury occasioned by his negligent supervision of construction.

98. 1 J.A. Dooley, Modern Tort Law: Liability and Litigation § 3.32, at 64 (1977) [hereinafter cited as Dooley], and Prosser § 36, at 191.

99. Dooley § 3.52, at 64; and Restatement (2d) of Torts § 288A, at 33.

100. See Hardaway v. Consolidated Paper Co., 366 Mich. 190, 114 N.W. 2d 236 (1962); and Prosser § 36, at 191.

101. Black's Law Dictionary (5th ed.) at 1071 defines prima facie evidence as "evidence which, if unexplained or uncontradicted, is sufficient to sustain a judgment in favor of the issue which it supports."

102. See Dini v. Naiditch, 20 Ill. 2d 406, 170 N.E. 2d 881 (1960); Larkins v. Kohlmeyer, 229 Ind. 391, 98 N.E. 2d 896 (1951); and Dooley § 3.32, at 64.

103. Minnesota provides that in all civil actions, violation of any statute is to be considered prima facie evidence of negligence only.

104. See Bruemmer v. Clark Equipment Co., 341 F. 2d 23 (7th Cir. 1965); White v. Morris Handler Co., Inc., 7 Ill. App. 3d 199, 287 N.E. 2d 203 (1972); Swarthout v. Beard, 388 Mich. 637, 202 N.W. 2d 300 (1972); Mississippi Meadows, Inc. v. Hodson, 13 Ill. App. 3d 24, 299 N.E. 2d 359 (1973); Vannoy v. City of Warren, 15 Mich. App. 158, 166 N.W. 2d 486 (1968); Kawneer v. Kalter, 187 Ind. 99, 118 N.E. 561 (1918) in which a safe scaffolding statute to protect workmen was held to establish a strict duty of compliance with the standards set; and Larson v. Commonwealth Edison Co., 33 Ill. 2d 316, 211 N.E. 2d 247 (1965) in which the Supreme Court of Illinois held a consulting engineering firm liable under Ill. Rev. Stat. ch. 48, §§ 60-69 (more commonly known as the Scaffold Act) for the injuries sustained by a workman when a scaffold on which he was working collapsed. Although the faulty scaffold was erected by the employer-contractor and no control was exercised over the work by the engineering firm, a contract's specification that the firm had the right to control work was ruled sufficient to establish its liability under the statute; and Dooley § 3.32, at 62.

105. Restatement (2d) of Torts § 285, at 21 and § 286, at 26. Note, however, that legislative enactments and regulations are not adopted as the standard of care to determine negligence in actions brought by individuals if the obligation created is due only to the state.

106. See Holt v. A.L. Salzman & Sons, 88 Ill. Ap. 2d 306, 232 N.E. 2d 537 (1967) in which an architectural firm was held to be negligent for preparing or approving plans that did not meet the standards set by municipal code and failed to provide for adequate bridging and proper joist spacing. Sheet metal workers injured when the joists collapsed were allowed recovery under the Structural Work Act; Miller v. DeWitt, 37 Ill. 2d 273, 226 N.E. 2d 630 (1967) where architects were found liable under the Structural Work Act for injuries received by construction workers when school gymnasium roof collapsed as a result of the contractor's faulty shoring operation. The work had been negligently inspected by the architects who were held to be persons "having charge" under the Act; and Voss v. Kingdon and Naven, Inc., 60 Ill. 2d 520 (1975) in which the issue of whether a construction and engineering firm that contracted to provide inspection of work to assure compliance with plans and specifications for a sewage disposal plant was to be liable, under the Structural Work Act, to employees injured when scaffolding collapsed as a result of the removal of a vertical support, was remanded for a jury's determination. The court noted, however, that the presence of an engineer at the job site with the power to stop work gave the firm "broad and sweeping" authority such as that contemplated under the Act; But see Vonasek v. Hirsch & Stevens, Inc., 6 Wis. 2d 1, 221 N.W. 2d 815 (1974).

107. Prosser § 36, at 191.

108. 21 Ill. App. 3d 925, 316 N.E. 2d 51 (1974).

109. Id. The manufacturer was declared liable, however, for loss resulting from use of the paneling based on fraud and deceit.

110. 325 F. 2d 605 (8th Cir. 1963).

111. Restatement (2d) of Torts § 286, at 25.

112. Restatement (2d) of Torts § 286, at 25.

113. Dooley § 3.35, at 66; and see Douglas v. Edgewater Park Co., 369 Mich. 320, 119 N.W. 2d 567 (1963) that specified that although violation of the Fire Prevention Act is negligence per se, violation of rules and regulations thereunder is only evidence of negligence.

114. See Presser v. Siesel Constr. Co., 19 Wis. 2d 54, 119 N.W. 2d 405 (1963) that held violation of safety order requiring all pits, openings in floors, etc. to be guarded by defendant contractor to be negligence per se.

115. Dooley § 3.28, at 57.

116. 419 F. 2d 1028 (C.A. 7th 1969).

117. 68 F. 2d 928 (1934).

118. 149 Mont. 286, 425 P. 2d 712 (1967). Use of the pamphlet, issued by the American Standards Association, was intended to establish the negligence of a mill owner and an architect for providing the ladder from which the plaintiff, a steamfitter, had fallen.

119. See 75 A.L.R. 2d §§ 778, 780 that states this to be the majority view.

120. Dooley § 3.28 at 56, 57.

121. See Ten State Standards: Recommended Standards for Individual Sewage Systems (1980), a report by the Committee of the Great Lakes Upper Mississippi River Board of State Sanitary Engineers that characterizes itself as a medium for the exchange of policies, procedures, and standards. Its publications are claimed to provide "what are considered to be the best engineered design standards for private waste disposal systems, as determined by research and practical experience." (at V.) They are worthless, however, without assurances that the "best" design standards comply with state or local ordinances and codes. The extent to which modification of or deviation from design specifications is permitted without falling below the standard is not stated, and the duration of such design standard's viability is not indicated. Finally, those standards considered to be the best" may not be generally accepted as such and, consequently, may not be admitted as evidence to either defend or defeat an action against an engineer for negligence. See note 119.

122. Dooley § 3.28, at 56, 57.

123. See Darling v. Charleston Community Memorial Hospital, 33 Ill. 2d 326, 211 N.E. 2d 253 (1965); 2 Harper and James, The Law of Torts, § 17.3, at 977, 978; and Prosser § 33, at 166.

124. Dooley § 3.27, at 51.

125. Id. §§ 3.29, 3.30, at 58, 59.

126. In 1964, the assistant secretary of a major professional liability insurer, Continental Casualty Co., cited the negligent preparation of plans and specifications as the most important area of a design professional's claim exposure. This area was calculated to be the basis for 20-25% of all malpractice claims against design professionals. Claims made in connection with site surveys, profiles, soil conditions, grades, and elevations account for another 20-25% of all claims asserted against architects and engineers according to the insurance company's figures. An estimated 15-20% involve the negligent selection of materials or equipment, 10% are claims by the contractors, and 10% involve claims for bodily injury sustained by the public after the project's completion or by workers during its construction. See Drake, Adjusting and Defending Claims Under Architects' and Engineers' Professional Liability Policies, 499 Ins. L.J. 466 (1964); and Triunfol, Professional Liability of Architects and Engineers, 499 Ins. L.J. 461 (1964). In 1973, however, a commentator claimed that design and supervision of construction "have given rise to the bulk of legal problems." Liability of Design Professionals --The Necessity of Fault,

58 Ia. L.R. at 1230 note. And in 1980, representatives of Schinnerer & Co., Inc., another major professional liability insurer, estimated that 22.5% of the 35,000 cases they had encountered involved claims of bodily injury. The dollar costs of these claims were noted not to have escalated anywhere near as much as the number of claims, however, and claims by construction workers only accounted for 48% of those cases involving bodily injury. Nevertheless, a substantial increase in the frequency of bodily injury cases was documented (from 15.1% of claims made in 1960-64, to 22.3% in 1965-69, and 23.6% in 1970-75) by the company's representatives at a meeting on August 7, 1980.

127. See Drake, Adjusting and Defending Claims Under Architects' and Engineers' Professional Liability Policies, 499 Ins. L.J. at 468 (1964). The author notes that a design professional's exposure to liability is a function of the type of work being undertaken and the geographical features of the area. (The litigious nature of the local population is also cited as a factor influencing the amount of claims made in an area by other commentators.)

128. Id.

129. Griswold and Rauma, Arch., Inc., v. Aesculapius Corp., 301 Min. 121, 221 N.W. 2d 556 (1974).

130. Shapiro, Design Professionals: Recognizing a Duty to Inform, 30 Hastings L.J. 729, 736 (1979).

131. 296 Mich. 114, 295 N.W. 580 (1941).

132. Id., at 120, 295 N.W. at 583. The architect had been advised before beginning to prepare the plans and specifications that the cost of improvements was not to exceed $15,000. The bid on the plans provided by the architect amounted to $28,000.

133. Capitol Hotel Co., Inc. v. Rittenberry, 41 S.W. 2d 697 (Tx. 1931).

134. Id.

135. 333 Mich. 233, 52 N.W. 2d 678 (1952).

136. Zannoth v. Booth Radio Stations, 333 Mich. at 244, 52 N.W. 2d at 684 (1952).

137. 5 Am. Jur. 2d Architects § 6, at 668 (1962).

138. Id. § 7, at 669.

139. Id.

140. 318 A. 2d 619 (Del. Super. 1974).

141. Id., at 624. See also Vonasek v. Hirsch & Stevens, Inc., 65 Wis. 2d 1, 221 N.W. 2d 815 (1974) in which an architect was held not bound to warn a contractor of the dangerous nature of ironwork that was or ought to have been known to the contractor.

142. 9 Wash. App. 682, 514 P. 2d 184 (1973).

143. Id., at _____, 514 P. 2d at 186.

144. See Associated Engineers, Inc. v. Job, 370 F. 2d 633 (1966) in which the duty to warn contractor's employees of latent dangers is said to be satisfied by notifying their superiors.

145. See Shapiro, Design Professionals: Recognizing a Duty to Inform, 30 Hastings L.J. 729 (1979) for a full discussion of the issue.

146. 431 F. 2d 714 (5th Cir. 1970).

147. 25 Cal. App. 3d 848, 102 Cal. Rptr. 259 (1972).

148. Shapiro, Design Professionals: Recognizing a Duty to Inform, at 742.

149. A.I.A. Document B141 (1977). See note 190 for information concerning the American Institute of Architects and standard form contracts.

150. Standard Form of Agreement Between Owner and Architect, A.I.A. Document B131 (1963).

151. Standard Form of Agreement Between Owner and Architect, A.I.A. Document B141, 1.1.1 (1977).

152. Id., 1.1.3.

153. Miller v. DeWitt, 37 Ill. 2d 273, 226 N.E. 2d 630 (1967) held that an architect with the power to stop work is duty-bound to do so if construction methods threaten the safety of a contractor's workers. Many of the actions by third parties that claim design professionals have a duty to warn rely on the Miller court's interpretation of a design professional's responsibility. Also, because engineers are licensed to protect the public health and safety, failure to warn of risks to public health or welfare inherent in a particular design (e.g., a rise in water levels caused by certain methods of wastewater treatment) may subject the engineer to liability.

154. 293 Pa. 97, 141 A. 638 (1928).

155. Giffels & Vallet, Inc. v. Edward C. Levy Co., 337 Mich. 177, 58 N.W. 2d 899 (1953).

156. See Liability of Design Professionals -- The Necessity of Fault, 58 Iowa L.R. at 1230 note (1973).

157. See Holt v. A.L. Salzman & Sons, 88 Ill. App. 2d 306, 232 N.E. 2d 537 (1967); and Simoniz v. J. Emil Anderson & Sons, Inc., 81 Ill. App. 2d 428, 225 N.E. 2d 161 (1967).

158. See A.E. Investment Corp. v. Link Builders, Inc., 62 Wis. 2d 479, 214 N.W. 2d 764 (1974).

159. See J. Ray McDermott & Co. U. Vessel Morning Star, 431 F. 2d 714 (5th Cir. 1970).

160. See Inman v. Binghamton Housing Authority, 3 N.Y. 2d 137, 164 N.Y.S. 2d 699, 143 N.E. 2d 895 (1957).

161. See Liability of Design Professionals -- The Necessity of Fault, 58 Iowa L.R. 1221-1223 note (1973); and Hill v. Poplar Pantries, 219 S.C. 263, 267-69, 64 S.E. 2d 885, 886 (1951).

162. Policy Statement on System Failures by the Committee of the Great Lakes Upper Mississippi River Board of State Sanitary Engineers in Ten State Standards: Recommended Standards for Individual Sewage Systems (1980). Malfunctions caused by a rarely occurring, non-seasonal phenomenon that are self-correcting without physical alteration of the absorption system are not considered to be failures by the Board. But see note 122.

163. See Jones v. Boeing Co., 153 N.W. 2d 897 (N.D. 1967) in which an architect-engineer was held liable to the owner for the amount secured by the owner's neighbor for damages caused by the flooding of his property.

164. See discussion of design professionals' liability for innovative technologies at conclusion of this paper.

165. See Chapel v. Clark, 117 Mich. 638, 76 N.W. 62 (1898); Ambassador Baptist Church v. Seabreeze Heating and Cooling Co., 28 Mich. App. 424, 184 N.W. 2d 568 (1970); and Cowles v. Minneapolis, 128 Min. 452, 151 N.W. 184 (1915).

166. Lower workers' compensation payments for contractors with good safety records provided some financial incentive.

167. See Philo, Revoke the Legal License to Kill Construction Workers, 19 De Paul L.R. 1 (1969); The Statistical Abstract of the United States issued by the U.S. Department of Commerce; and 342 U.S. Bureau of Labor Statistics Report, Injury Rates: by Industry, at 18 (1964-65) that indicates that the construction industry had frequency rates twice as high, and severity rates three times as high as those of the manufacturing industry.

168. Ill. Rev. Stat. ch. 48, § § 60-69 (1975); See also Burns Ann. Stats. Ind., 20-301 to 307 (Indiana Dangerous Occupations Act of 1911).

169. Id. § 60.

170. Ill. Rev. Stat. ch. 48, § 69.

171. 33 Ill. 2d 316, 211 N.E. 2d 247 (1965); and see Fidelity & Casualty Co. of N.Y. v. J.A. Jones Construction Co., 325 F. 2d 605 (8th Cir. 1963) for a slightly earlier ruling of similar effect.

172. See note 105 for details.

173. 59 Ill. App. 2d 38, 208 N.E. 2d 249 (1965).

174. 37 Ill. 2d 273, 226 N.E. 2d 630 (1967).

175. 37 Ill. 2d 273, J. House at 293-94, 226 N.E. 2d at 643.

176. Reber v. Chandler High School Dist. #202, 13 Az. App. 133, 474 P. 2d 852 (1970).

177. See, for example, Associated Engineers, Inc. v. Job, 370 F. 2d 633 (1966) in which the obligation to supervise construction was considered to entail more than assuring the end product's conformance with specifications, and the engineer was held liable for a worker's injuries sustained by electric shock; Swarthout v. Beard 388 Mich. 637, 202 N.W. 2d 300 (1972); Voss v. Kingdon and Naven, Inc., 60 Ill. 2d 520, 328 N.E. 2d 297 (1975); and Geer v. Bennett, 237 So. 2d 311 (Fla. App. 1970).

178. Restatement (2d) of Torts § 414, at 387; Seeney v. Dover Country Club Apartments, 318 A. 2d 619, 623 (De. 1974); and Weber v. Northern Illinois Gas Co., 10 Ill. App. 3d 625, 295 N.E. 2d 41, 49-50 (1973 that states, "This rule is applicable to anyone with authority who entrusts work to an independent contractor, e.g., an owner, general contractor or an architect."

179. See discussion on the doctrine of privity's fall.

180. See Herbst, Malpractice: The Design Professional's Dilemma, 10 John Marshall J. 287 (1977).

181. The A.I.A. is a national society whose membership is primarily made up of registered architects. The preparation and periodic updating of its standard form contracts for the planning, construction, or alteration of structures is only one of the services provided by the A.I.A. Intensive Lobbying on behalf of its members is another. See note 230.

182. A.I.A., Standard Form of Agreement Between Owner and Architect on a Basis of a Percentage of Construction Cost (A.I.A. Document B131, September, 1963 ed.).

183. A.I.A., Standard Form of Agreement Between Owner and Contractor, 9.6 (A.I.A. Document A107, September, 1970 ed.).

184. See The General Conditions of the Contract for Construction, 2.2.13 (A.I.A. Document A201, 1976 ed.) that specifies that "[t]he Architect will have authority to reject work which does not conform to Contract Documents...However, neither Architect's authority to act under this Subparagraph 2.2.13, nor any decision made by him in good faith either to exercise or not to exercise such authority, shall give rise to any duty or responsibility of the Architect to the Contractor, any subcontractor, any of their agents or employees, or any other person performing any of the Work." See also the similarly exorcised standard form documents of the National Society of Professional Engineers (N.S.P.E.).

185. See McGovern v. Standish, 65 Ill. 2d 54, 357 N.E. 2d 1134 (1976) in which the Illinois Supreme Court considered whether an architect possessing only supervisory powers was liable for a worker's injuries sustained in a fall from a scaffold that failed to comply with provisions of the Structural Work Act. A jury originally found for the plaintiff, the appellate court reversed, and the Illinois Supreme Court affirmed the appellate court's reversal. In its decision, the Court set specific interpretive guidelines and limitations for future applications of the Miller ruling by determining that because the architect's contract had conferred no right to stop work, and because the architect neither gave orders relating to construction procedures nor sought to exercise control over them, the architect was not a person "in charge of" the work and was therefore not liable for the use of construction methods in violation of the Act that resulted in injury to workers.

186. McGovern v. Standish, 65 Ill. 2d 54 at 68, 357 N.E. 2d at 1141.

187. Id. at 69, 357 N.E. 2d at 1142.

188. Id.

189. 65 Ill. 2d 54, 357 N.E. 2d 1134 (1976).

190. Id. at 67, 357 N.E. 2d at 1141.

191. See Emberton v. State Farm Mut. Auto. Ins. Co., 71 Ill. 2d 111, 119, 373 N.E. 2d 1348, 1351 (1978); and Norton v. Wilbur Waggoner, 76 Ill. 2d 481, 490, 394 N.E. 2d 403, 407 (1979).

192. See note 47.

193. For excellent analysis of the problem and proposals for its resolution see Liability of Design Professionals--The Necessity of Fault, 58 Iowa L.R. note 1221 (1973) and Philo, Revoke the Legal License to Kill Construction Workers, 19 De Paul L.R. 1 (1969). Both commentators point out the inefficiency of imposing liability on the design professional for injuries occasioned by the method of construction used.

194. See Acret, Architects and Engineers: Their Professional Responsibilities § 9.12 (1977).

195. See discussion on "Failure" above.

196. See Morse v. Michaelson, Rabig & Ramp, 10 Ill. App. 2d 366, 243 N.E. 2d 271 (1968) in which an architect was held liable for selecting an incompetent and impoverished heating contractor.

197. See Peerless Ins. Co. v. Cerney & Assoc., Inc., 199 F. Supp. 951 (1961) in which the 5% error accepted by the profession on certification of payments was exceeded by the architect's approval of estimates totaling $314,000 when only $202,000 had been earned by the contractor. The surety was allowed recovery; Hall v. Union Indem. Co.,

61 F. 2d 85, (8th Cir. 1932) in which a contract calling for the architect to certify monthly payments upon the contractor's presentation of invoices was considered breached by the architect's acceptance of monthly "estimates". The architect was held liable for the damages incurred by the surety; Bump v. McGrannaham, 61 Ind. App. 136, 111 N.E. 640 (1916) in which a surety recovered for a contractor's default occasioned by the architect's premature certification of payment, i.e., before the proportion of work paid for was actually completed; and Palmer v. Brown, 127 Cal. App. 44, 273 P. 2d 306 (1954) in which an owner recovered for an architectural firm's negligence in issuing certificates to contractors for completion of work without ascertaining whether the premises were free from liens chargeable to the contractors or examining the work to see if it had been done properly.

198. See note 14.

199. Restatement (2d) of Torts § 328A, at 149.

200. Prosser § 37, at 205.

201. Legal or proximate cause is that which satisfies the law's requirement of a causal connection sufficient to impose liability. It is a limitation of responsibility for consequences developed by the courts as a policy matter. See notes 53, 54 and the discussion to which they refer.

202. Restatement (2d) of Torts § 431, at 428.

203. Id. § 433, at 432.

204. Prosser § 41, at 240.

205. Id.; and see Anderson v. Minneapolis, St. P. & S.S.M.R. Co., 146 Min. 430, 179 N.W. 45 (1920).

206. Restatement (2d) of Torts § 440, at 465.

207. See Liability of Design Professionals--The Necessity of Fault, 58 Iowa L.R. 1221 note (1973).

208. See Inman v. Binghamton Housing Authority, 3 N.Y. 2d 137, 164 N.Y.S. 2d 699, 143 N.E. 2d 895 (1957).

209. See McDonough v. Whalen, 364 Mass. 837, 313 N.E. 2d 435 (1974) in which purchasers of a home were allowed to recover for property damage caused by the negligent installation of a septic system recommended by the same engineer who later inspected and approved the system acting in his capacity as agent for the town board of health. The claim that "great mental anguish and embarassment" had also been suffered by the flooding and odor of the plaintiff's yard was dismissed by the court as lacking sufficient evidence. It was not, however, dismissed on the basis that the alleged mental anguish was unaccompanied by any physical injury.

210. §30, at 143.

211. See Johnson v. Rouchleau-Ray Iron Land Co., 140 Minn. 289, 168, 1 (1918).

212. "The Statute of Limitations does not begin to run against a negligence action until some damage has occurred." Prosser § 30, at 144.

213. See note 211.

214. This difference has resulted in many claims of economic loss rather than personal injury or porperty damage in order to bring an action under contract rather than tort theory. See Schenburn v. Lehner Associates, Inc., 22 Mich. App. 524, 177 N.W. 2d 699 (1970); and City of Milwaukee v. Schmidt, Garden, & Erikson, 43 Wis. 2d 445, 168 N.W. 2d 559 (1969).

215. 53 C.J.S. Limitations of Actions § 1 (1948).

216. See note 33 and the discussion to which it refers.

217. See Simoniz v. J. Emil Anderson & Sons, Inc., 81 Ill. App. 2d 428, 225 N.E. 2d 161 (1967).

218. Bd. of Education v. Joseph J. Duffy Co., 97 Ill. App. 158, 240 N.E. 2d 5 (1968).

219. See note 55 and the discussion to which it refers.

220. 43 Ill. 2d 54, 250 N.E. 2d 656 (1969).

221. 31 Ill. App. 3d 61, 333 N.E. 2d 65 (1975).

222. 31 Ill. App. 3d 1060, 335 N.E. 2d 588 (1975). Note, however, that the discovery rule's application was used to determine that a cause of action accrues "when a plaintiff knew or should have known of the error," not necessarily at the time the defect was actually discovered. Id. at 1061, 335 N.E. 2d at 589.

223. Neeson, Current Status of Professional Malpractice Liability Insurance, 45 Ins. Counsel J. 39, note 45, at 44 (1978).

224. Comment, Recent Statutory Developments Concerning the Limitations of Actions Against Architects, Engineers, and Builders, 60 Ky. L.J. 462 (1972).

225. Taff, A Defense Catalogue for the Design Professional, 45 U.M.K.C. L.R. 75, note 130 at 92 (1976). All of the Region V states enacted statutes in response to the lobbying efforts. See Ill. Rev. Stat. ch. 83, § 24f (Smith-Hurd 1966), Ind. Code §§ 34.4.20.1 to 34.4.20.2 (Burns 1971), Mich. Comp. Laws § 600.5839 (1968), Minn. Stat. § 541.051 (supp. 1971), Ohio Rev. Code Ann. § 2305.131 (supp. 1970), and Wis. Stat. § 893.155 (1966).

226. Neeson, Current Status of Professional Malpractice Liability Insurance, 45 Ins. Counsel T. at 44.

227. The model statute endorsed by the professional associations suggested four years as a reasonable time limitation. Hearings on H.R. 6527, H. R. 6678 and H. R. 1154 Before Subcommittee No. 1 of the House Committee on the District of Columbia, 90th Cong., 1st Sess. 34 (1967). The Illinois statute fixed the recommended four year limitation. The Indiana, Minnesota, and Ohio statutes are among the substantial number that set a ten year limitation period.

228. Drake, Adjusting and Defending Claims Under Architects' and Engineers Professional Liability Policies, 499 Ins. L.J. 466, 467 (1964).

229. Comment, Limitation of Action Statutes for Architects and Builders--Blueprints for Non-Action, 18 Catholic L.R. 361, note 17 at 363 (1969).

230. Nine statutes attacked as unconstitutional have been upheld by the courts. See Taff, A Defense Catalogue for the Design Professional, 45 U.M.K.C. L.R., note 132 at 92.

231. Ill. Rev. Stat. ch. 83, § 24 f (Smith-Hurd 1966).

232. Mich. Comp. Laws § 600.5839 (1968).

233. Ohio Rev. Code Ann. § 2305.131 (supp.1970).

234. See for example Ark. Stat. Ann. § 37-241 (supp. 1967), and Cal. Civ. Pro. Code § 337.1 (a)(1) West Supp. 1968).

235. 38 Ill. 2d 455, 231 N.E. 2d 588 (1967).

236. Ill. Const., art. IV § 22.

237. 66 Wis. 2d 382, 225 N.W. 2d 454 (1975).

238. Material men, owners, and occupants were cited by the court as classes denied equal protection without reason.

239. 66 Wis. 2d 382, 393 225 N.W. 2d 454, 460 (1975).

240. Comment, Limitation of Action Statutes for Architects and Builders--Blueprints for non-Action, 18 Catholic L.R. at 372 (1969).

241. Neeson, Current Status of Malpractice Liability Insurance, 45 Ins. Counsel J. at 45. This conclusion was offered with the observation that

> [t]he Model Act itself, through either inadvertance or poor draftsmanship, arbitrarily discriminates in the length of time allowable for a claimant to file suit when the discovery of the cause of action occurs in the fourth year after completion of a project as compared to a claimant who had discovered his cause

of action in either the third or fifth year after completion. Thus the Model Act by itself presents a clean violation of equal protection.

242. Ill. Rev. Stat. ch. 51, § 58 (1975).

243. See Herbst, Malpractice: The Design Professional's Dilemma, 10 John Marshall J. 287 (1977).

244. Comment, Recent Statutory Developments Concerning the Limitations of Actions Against Architects, Engineers, and Builders, 60 Ky. L.J. 462 (1972).

245. See discussion of "claims made" policies below.

246. Taff, A Defense Catalogue for the Design Professional, 45 U.M.K.C. L.R. note 1, at 75 (1976).

247. Id.

248. See Shapiro v. Bd. of Regents of Univ. of State of N.Y. 29 A.D. 2d 801, 286 N.Y.S. 2d 1001 (1968) in which an engineer lost his license for six months; and Vivian v. Examining Board of Architects, 61 Wis. 2d 627, 213 N.W. 2d 359 (1974) in which an engineer's license was threatened with revocation of the Examining Board of Architects, Professional engineers, Designers, and Land Surveyors found that the engineer's failure to properly design garage addition that collapsed constituted incompetency.

249. The Ill. Rev. Stat. ch. 10½, § 13 (1975) provides that the Department of Registration and Education may refuse to renew, may suspend or may revoke any certificate of registration for a) gross incompetency or b) recklessness in the design, planning or supervision of construction of buildings.

250. The widespread use of liability insurance also increases the number of claims made against design professionals, for once insured the engineer and architect provide a "deep pocket" the injured worker can look to for help.

251. Neeson, Current Status of Professional Malpractice Liability Insurance, 45 Ins. Counsel J. 39, 45 (1978).

252. Triunfol, Professional Liability of Architects and Engineers, 499 Ins. L.J. 461 (1964). The history of the formation and continuing association among these groups has been verified by affidavits signed by Milton Lunch, General Counsel for N.S.P.E. and J. Winfield Rankin, Administrator for the A.I.A.

253. Neeson, Current Status of Professional Malpractice Liability Insurance, 45 Ins. Counsel J. at 43 (1978).

254. A "comprehensive" policy may exclude professional services (malpractice) liability. See Wheeler v. Aetna Casualty and Surety Co.,

11 Ill. App. 3d 841, 298 N.E. 2d 329 (1973) that declared a carrier can safely refuse to defend if the injury complaint shows on its face that liability arose out of activities specifically excluded (e.g., prepartion of plans, making cost estimates, etc.). And a professional liability policy may exclude actions for breach of contract, indemnity agreements, or injury to worker or third party not caused by error in preparation of plans or specifications. Supervision is a general liability according to insurance policies. See Triunfol, Professional Liability of Architects and Engineers, 499 Ins. L.J. 461 (1964).

255. Neeson, Current Status of Professional Malpractice liability Insurance, at 43.

256. Neeson, Current Status of Professional Malpractice Liability Insurance, 45 Ins. Counsel J. at 48.

257. This is so despite the development of "post-practice protection policies" for retiring design professionals that lower rates in accordance with the decreasing risk they represent by ceasing practice.

258. See Continental Casualty Co. v. Enco Associates, Inc., 66 Mich. App. 46, 238 N.W. 2d 198 (1975).

259. Neeson, Current Status of Professional Malpractice Liability Insurance, 45 Ins. Counsel J. at 43 (1978).

260. Id. at 47. In addition, the number of professional malpractice insurers has been decreasing in recent years.

261. Maurer, Architects, Engineers and Hold Harmless Clauses, 1976 Ins. L.J. 725 (1976).

262. 42 C.J.S. Indemnity, § 1 (1944).

263. There are five types of hold harmless clauses:

1) A indemnifies B for liability arising out of A's own negligence,

2) A indemnifies B for liability arising from negligence of A and B, but not that resulting from B's negligence alone,

3) A indemnifies B for all liability arising from either's negligence without regard to fault.

4) A indemnifies B for all liability arising from negligence of anyone performing project work, and

5) A indemnifies B for all loss, injury, or damage regardless of fault or cause.

264. See Becker v. Black & Veatch Consulting Engineers, 509 F 2d 42 (S.D. 1974).

265. See Mustas v. Inland Construction Co., 19 Wis. 2d 194, 120 N.W. 2d 95 (1963) and Brown v. Wis. Nat. Gas Co., 59 Wis. 2d 334, 208 N.W. 2d 769 (1973).

266. See Jacobsen v. Ravenhorst Corp., 301 Min. 202, 221 N.W. 2d 703 (1974).

267. Prosser § 68, at 442.

268. Ill. Rev. Stat. ch. 29, § 61 (1975).

269. See White v. Morris Handler Co., Inc. 7 Ill. App. 3d 199, 287 N.E. 2d 203 (1972).

270. Maurer, Architects, Engineers, and Hold Harmless Clauses, 1976 Ins. L.J. 725 (1976).

271. An architect or engineer can defeat a plaintiff's action if one of the following is established:

    1. No duty existed according to

        a) contract (Terms of the contract or lack of privity may be sufficient.),

        b) case law (The standard of conduct required of design professionals may be determinative.), and

        c) statutory requirements (The statute may be inapplicable or irrelevant to the issue of negligence. If controlling, the statute may have been complied with or waived by the issuance of a permit.).

    2. The design professional was not negligent. (The standard of care required by contract, case law, and statute was exercised.)

    3. Even if negligent liability does not attach because

        a) the design professional's negligence was not established as the proximate cause of plaintiff's harm, or

        b) another's negligence was a supervening cause.

    4. The plaintiff's harm is not recognized as compensable.

    5. The plaintiff's right to recovery is barred by

        a) completion and acceptance of the structure or facility [But see Elliot v. Nordlof, 83 Ill. App. 279, 227 N.E. 2d 547 (1967)],

b) the plaintiff's assumption of the risk or contributory negligence, or

c) the statute of limitations.

A plaintiff's recovery from a design professional may also be limited if:

1. another's negligence constituted intervening cause,

2. the plaintiff's contributory negligence is considered in accordance with comparative negligence doctrine, or

3. the design professional was indemnified by another party.

272. Liability of Architects and Engineers to Third Parties, 53 Notre Dame L. at 317 note (1977).

273. Dines Implementation of Onsite Wastewater Management Arrangements -- Overcoming Incentives for Inaction, in Individual Onsite Wastewater Systems, at 70 (N. McClelland, ed. 1978).

274. "Standards in the field of onsite waste disposal tend to be characterized by inflexibility and antiquity." Reed, State Programs and Perspectives, in Individual Onsite Wastewater Systems, at 29 (N. McClelland, ed. 1978).

275. See Williams, You Broke My Cookies, in Individual Onsite Wastewater Systems, at 230 (N. McClelland, ed. 1978) that states that difficulty in obtaining regulatory agency approval to use innovative developments is characteristic of such efforts and that typically, the "hundreds of man-hours, stacks of correspondence, hours of meetings and phone calls" involved in developing and testing new methods of treatment are not financially compensated.

276. Construction contracts generally include a schedule of fees according to which the design professional is paid. Consulting engineers find it is hardly worth the cost and expense of opposing a staunchly entrenched bureaucracy to explore new technologies. Engineers cannot afford to concern themselves with the cost-effectiveness of a conventional design.

277. Address by Cecil W. Rose, Fourth National Conference on Individual Onsite Wastewater Systems (1977) (published in Individual Onsite Wastewater Systems, at 26 (N. McClelland, ed. 1978).

278. Ind.

279. See 33 U.S.C. 1254 (v), 1282 (a) (2), 1285 (h).

280. Russell v. Men of Devon, 2 Term Rep. 667, 100 Eng. Rep. 359 (1798).

281. Prosser § 131, at 978. And see Monroe v. Pape, 365 U.S. 167, 81 S. Ct. 473 (1961) in which a municipality was held not to be a person for the

purposes of seeking money damages; Moor v. County of Alameda, 411 U.S. 693, 93 S. Ct. 1785 (1973) which held a county is not a person for purposes of economic liability; and Monell v. Dept. of Social Services of City of N.Y., 532 F. 2d 259 (1976).

282. See Ostrander v. City of Lansing, 111 Mich. 693, 70 N.W. 332 (1897).

283. See Svendsen v. Village of Alden, 101 Min. 158, 112 N.W. 10 (1907); but see Erickson v. City of West Salem, 205 Wis. 107, 236 N.W. 579 (1931).

284. Prosser § 131, at 984.

285. Wis. Stats. ch. 101 (1933).

286. Heiden v. City of Milwaukee, 222 Wis. 92, 275 N.W. 922 (1940).

287. Molitor v. Kaneland Community Unit dist. No. 302, 18 Ill. 2d 11, 163 N.E. 2d 89 (1959) held a school district liable when a child was injured by the negligent operation of a school bus.

288. Williams v. City of Detroit, 364 Mich. 231, 111 N.W.2d 1, (1961).

289. Holytz v. City of Milwaukee, 17 Wis. 2d 26, 115 N.W. 2d 618 (1962).

290. 178 So. 2d 312 (La. App. 1965), cert. denied, 248 La. 434, 179 So. 2d 274 (1965).

291. See Taff, A Defense Catalogue for the Design Professional, 10 John Marshall J. 287 (1977).

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