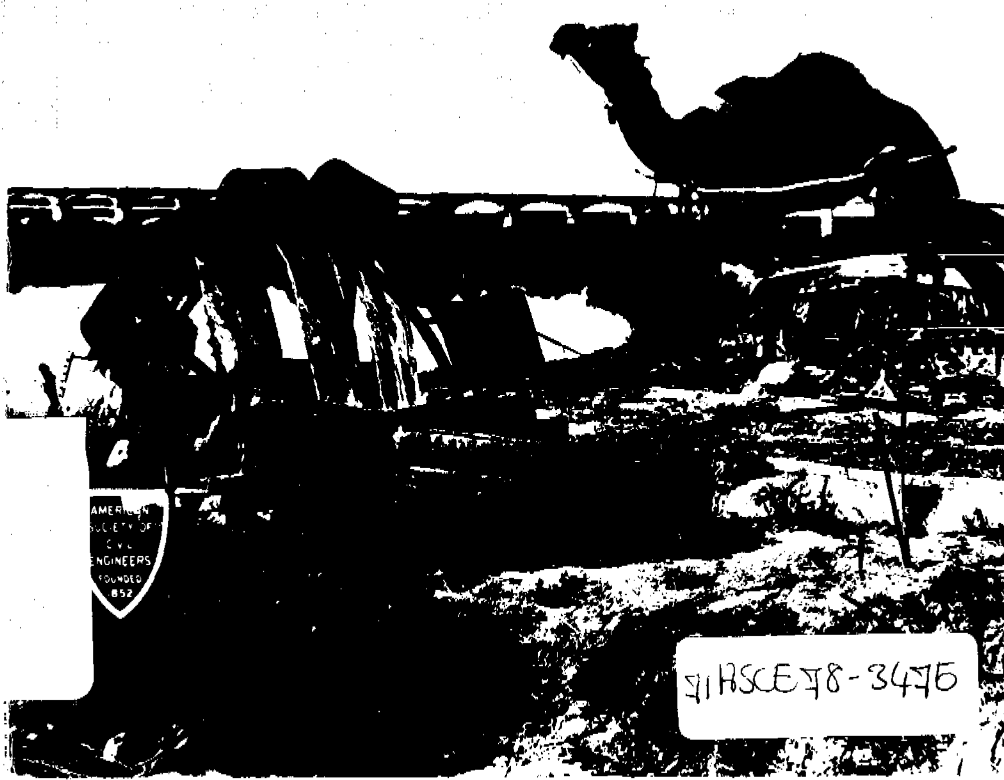
Appropriate Technology in WATER SUPPLY AND WASTE DISPOSAL



dec. 1983
ASCE
$ 20.–
(thv PSWS bibliogr.)

Appropriate Technology in WATER SUPPLY AND WASTE DISPOSAL

Proceedings of a Session sponsored by the Research Council on Environmental Impact Analysis of the ASCE Technical Council on Research at the ASCE National Convention in Chicago, Illinois, October 16-20, 1978.
(Originally published as ASCE Preprint 3453)

Charles G. Gunnerson and
John M. Kalbermatten, Editors

Published by
American Society of Civil Engineers
345 East 47th Street
New York, New York 10017

Price: $16.00

AMERICAN SOCIETY OF CIVIL ENGINEERS

ENVIRONMENTAL IMPACT ANALYSIS

Research Council

APPROPRIATE TECHNOLOGY IN WATER SUPPLY AND WASTE DISPOSAL

FOREWORD

This workshop is a follow-up to one conducted by the Environmental Impact Analysis Research Council at San Francisco, October 1977 on Environmental Impacts of International Civil Engineering Projects and Practices.

The objective of this workshop is to review current information on policy and technological choice in sanitary and environmental engineering. Included in the session are papers dealing with historical, behavioral, economic, public health, engineering and environmental determinants. Information from case studies in both developing and industrial countries is presented. Emphasis is given to the fact that different technologies are appropriate for different environmental conditions, different stages of urbanization and industrial development, and different institutional constraints.

The long-term objective of EIARC is to identify and develop guidelines which will lead to greater acceptance, utilization, environmental strength, and economic viability of the projects to which civil engineers contribute.

Charles G. Gunnerson
Chairman, EIARC

John M. Kalbermatten
Member, EIARC

ABSTRACT

KEY WORDS: Appropriate technology, Behavioral sciences, Developing countries, Disease classification, Economic analysis, Environment, Epidemiology, Financial analysis, Irrigation, Land treatment, Marine disposal, Oceanography, Ponds, Sanitation, Service levels, Social sciences, Public participation, Water distribution, Water supply, Water treatment, Waste disposal, Waste reclamation, Waste treatment.

ABSTRACT: Twelve contributions to proceedings of an April 1978 ASCE workshop deal with both developing and industrial countries with the (1) historical development of water and sanitation systems and related environmental and institutional factors, including effluent regulation and zero discharge, (2) behavioral factors in technology selection and the characteristics of pre- and post-project public participation activities, (3) economic incentives, (4) ancient and modern irrigation practices and current Israeli research, (5) water treatment plant designs in which pipe galleries are eliminated and operations are simplified, (6) factors in selection of water service levels (standpipe, yard connection, or full plumbing) and distribution networks, (7) sanitation selection factors, economic costs, and upgrading schedules for systems ranging from pit latrines to conventional sewerage, (8) environmental classification of excreta-related disease and the epidemiological effectiveness of sanitation systems, (9) cost-effectiveness of waste treatment ponds, (10) land treatment systems for municipal wastes, (11) European waste treatment practices, legislation, and institutions, and (12) measuring and predicting the effects of marine waste disposal.

REFERENCE: Gunnerson, Charles G. and Kalbermatten, John M., Editors, Appropriate Technology for Water Supply and Waste Disposal, American Society of Civil Engineers, New York, 1979.

Cover photo: Persian water wheel with Sukkar Barrage in the background in Pakistan (Pakistan Embassy photo).

CONTENTS

List of Tables

List of Figures

Plate I. Pumping water in village within Lelongwe Development scheme, Malawi (World Bank photo by James Pickerell)

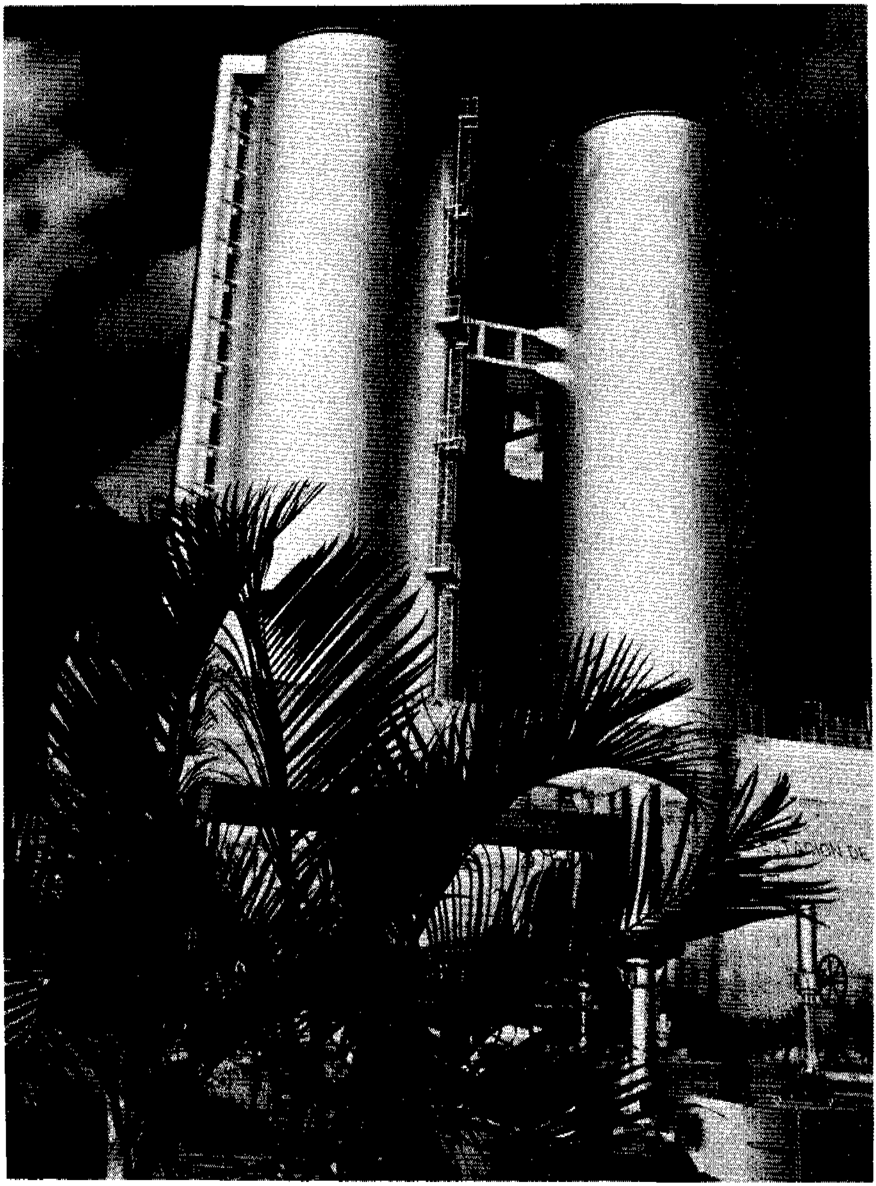


Plate II. Pumping water in urban Caracas, Venezuela. The tanks are part of the No 4 Pumping Station, Caracas Water treatment plant (UN photo by Ray Witlin).

THRESHOLDS IN APPROPRIATE TECHNOLOGIES FOR WATER SUPPLY AND WASTE DISPOSAL

by

Charles G. Gunnerson, F. ASCE*

Abstract

Appropriate technologies come from a variety of sources. Most, though not all, technologies are appropriate to the specific time and place in which they are developed. Some can be transferred to other times and places and many can be improved in stages as additional resources become available. A review of historical development in water supply and waste disposal practices contributes to a general model of the thresholds (or constraints) which affect technological choice. This study of historical and recent environmental, technological, and socio-economic determinants in water supply and waste disposal reveals that the most important thresholds in technological choice are those of willingness to pay, the flow of energy and material resources in environmental systems, and the development of appropriate institutions which can respond to the dynamics of economic development and technological changes.

Background

The history of water supply and waste disposal reveals a large variety of environmental, technological, and socio-economic thresholds. These thresholds are reflected in service levels, costs, benefits, and institutional response. Ancient technologies, some of which may be appropriate today, were developed by peasant artisans whose ancestors were the progenitors of the Industrial Revolution (1). A straight-forward example which includes several thresholds is that of communities which developed along streams which provided water, transportation, and possibly food. Periodic flooding was always a problem so that

* Environmental Engineering Advisor, National Oceanic and Atmospheric Administration, Environmental Research Laboratories, Boulder, CO 80302

the smaller tributraries were gradually improved to better handle the runoff. Sometimes major drains were built from mosquito-laden marshes to the main stream(2).

Meanwhile domestic water was brought in from local lakes, streams or wells. When water demand exceeded the reliable local supply, community systems tapped more distant sources. Sullage from kitchens, baths, and laundries was used for garden watering in arid zones or tossed onto the street or yard in more rainy ones. Sanitation was often primitive and limited to open areas, small or large pits, (occasionally with admonitions such as those in Deuteronomy(3) to cover it up), or streams. In urban areas the problems of disposal of nightsoil (human excreta) increased along with the numbers and density of people. As at present, these problems were generally solved by transportation or on an individual basis by tossing it out the window as in Rome(4) or more recently in Edinburgh ("Gardy-loo!"). When the police and courts could no longer keep up with the offenders, community transportation schemes for removal of nightsoil buckets were established. Regulation was important; Socrates' Athenian Constitution(5) provides for ten City Commissioners whose responsibilities included seeing that "no collector of sewage shall shoot any of his sewage within ten stadia (about 2 km) of the city walls." There were exceptions. The Chinese have for at least 4000 years fertilized their fields with nightsoil(6) and there are records of sales during the 19th and 20th century of nightsoil in England(7) and today in Taiwan(8).

Flushing of wastes seems always to have been a luxury of the wealthy beginning with drains in Egyptian temples of c.2500 B.C. and in 3rd millenium Indus Valley and Mesopotamian cities(9). When community water systems became available, flushing of excreta to streams or to smaller improved channels or drains became possible. When the stink from these drains which by this time were combined sewers became intolerable, the drains were covered over and the problem moved downstream.

Flushing of wastes is still a luxury today. It requires large immediate investments in community water supply, individual or community sewerage systems, and household plumbing. In fact, it costs three to six times as much to get rid of wastewater as it does to supply the water in the first place. Some of these costs are the environmental ones of pollution impacts which result from relying upon transportation to solve waste disposal problems.

Another element of luxury is privacy in bathing and defecation. This does not seem to have concerned the ancient Greeks who could appreciate Aristophanes' scatological humor and wonder with Xenophon at the Persians' modesty. Most people, then and now, whether from rural areas of developing countries or from industralized cities want privacy and are willing to pay for it.

Environmental costs of water pollution have caused a variety of responses for generations, particularly in the industralized countries. Examples of these responses range from the Urban Sanitary Act of 1388, drawn up as a result of the Black Death in which putting wastes into surface waters was forbidden(10) to the Federal Water Pollution Control Act Amendments of 1977. Thus the political response is that of regulation and regulation is inherently inflationary(11).

Clearly, a number of thresholds in water supply and waste disposal can be identified in both the historical record and in their modern counterparts in developing countries. These are discussed according to the classification with which this paper began.

Environmental Thresholds

Regional environmental thresholds derive from climatic and geologic factors which include the amount of water available, soil fertility, temperatures and length of growing season. These combine to determine the potential productivity shown in Figure 1 in grams carbon fixed by photosynthesis per square meter per year. While the highest values are found in the tropics, high commercial values of crops produced are also found in temperate zones. Water is clearly abundant in areas with more than 400 g $C/m^2/yr$ and deficient in desert areas with less than 100 g $C/m^2/yr$. Waste disposal in the tropics reflects the rapid cycling of organic material through the system where oxidation ponds are particularly effective in wastewater treatment(13).

Areas in the intermediate zones with 100 to 400 g $C/m^2/yr$ include deserts and areas subject to desertification. The aridity index is quantified by computing the ratio of potential evaporation to precipitation.

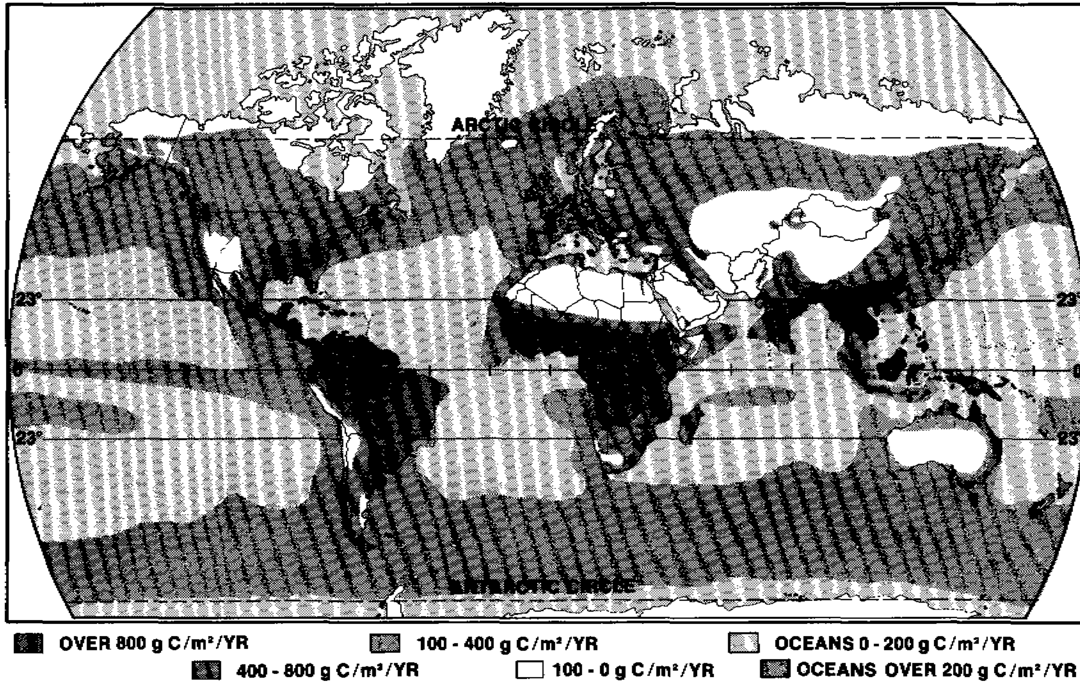


Figure 1. Potential productivity of regions of the earth (after Leith(12).

These are shown on Figure 2 for Africa where next to the Sahara Desert in the north and next to the Namib Desert in the south, desertification is underway because of overgrazing, deforestation, and the desperate search for firewood(14,15). In arid areas reclamation of wastewater for irrigation and waste solids for restoring soil fertility and tilth is a critical design factor. In contrast, waste reclamation in rainy tropical areas includes use of nightsoil as a fertilizer for fish ponds.

Local environmental thresholds include surface water, groundwater and soil properties which establish water supply, borehole, privy, aquaprivy and septic tank system designs(16-20).

Environmental change ranges from gradual long-term continental-scale climatic change to local catastrophic events such as floods, tsunamis, earthquakes, explosive volcanic eruptions, tornadoes and fires. Intermediate-scale events are generally climatic and include drought, seasonal weather patterns, etc. whose return periods are used in designs of larger water supply systems.

Technological Thresholds

Irrigation and Drainage. These are the most ancient hydraulic works. Forbes(9, 21) has given a detailed account which describes construction during the 3rd and 4th millenia for irrigation in Egypt, Mesopotamia, and China. Meanwhile, navigation canals were built around the first cataract of the Nile in 2400 B.C. and between the Nile and Red Sea in 1950 B.C. Drainage structures appeared in Greece in 1400, in Rome during the 6th Century with the Cloaca Maxima, and in the 5th century in the middle East and China.

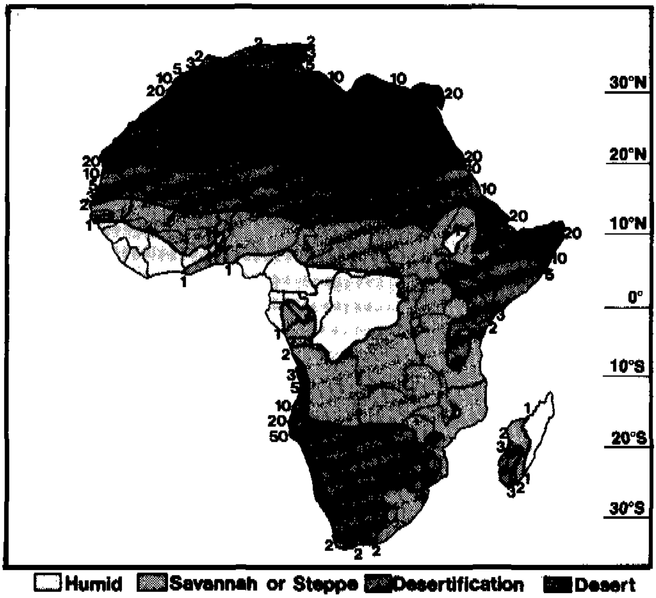


Figure 2. Aridity index for Africa = mean annual net radiation/(mean annual precipitation X latent heat of vaporization), (after D. Henning, Atlas of Climate Aridity Indices, in preparation, 1978 and F.K. Hare, Climate and Desertification, Univ. of Toronto, 1976).

The timing of the early irrigation works resulted from technological, economic, and institutional development in arid lands; they were not a response to the extended periods of dryness identified by Hendrie(22). In contrast, severe drought caused the breakdown of government such as the anarchy following the 6th Dynasty in Egypt in about 2200 B.C., and the concurrent weakening or disappearance of other civilizations around the eastern Mediterranean and Aegean regions(23). A different response was shown by construction throughout the 6th to the 9th Centuries A.D. in northwestern Europe of dikes and artificial mounds which was quite clearly in response to the extended period of wetter and colder climate (the Subatlantic phase) which began about 500 A.D.

Meanwhile, ground-water development for irrigation began in Urartu (Armenia) with qanats adapted from ancient mining methods. These were adits with ventilation shafts constructed in alluvial fans. They were well-developed by the 8th Century in Assyria, Persia, Arabia, India and North Africa(21, 24).

Domestic Water Supply. Structures for domestic water supply appeared somewhat later than those for irrigation. About 2500 B.C. in the Indus Valley, Mesopotamia, and Egypt, hand-dug wells often lined with stone, cisterns for rainwater, plumbing in temples and palaces, and open drains in the streets were built. Shafts and tunnels for gravity flow of spring waters were built throughout Palestine beginning in 1900 B.C. at Gezer and including Hezekiah's 700 B.C. tunnel in Jerusalem.

The Greeks used a qanat-type aqueduct to bring water to Athens in the 6th Century, B.C. and later, at Pergamum constructed a 5 km inverted siphon from settling basins at Hagios Georgos. The siphon carried pressures of up to 21 kg/cm^2 (300 psi). It was later replaced by the Romans with a series of aqueducts(25).

The monumental aqueducts are a hallmark of Roman civilization. In Rome itself, their construction mostly took place between 312 B.C. and 52 A.D. Figure 3 and Table 1 summarize their characteristics. Water service levels which the aqueducts provided are discussed below.

In arid lands, other technologies developed. In the Negev Desert, the Nabateans harvested rainfall by windrowing hillside gravels, thus creating barren strips for rapid runoff into crosschannels leading to farms (note that the windrows collect rainwater, not dew or fog as has been suggested). Modern technologies provide for diking small areas (microcatchments) of 30 m^2 (320 ft^2) area in a region with <10 cm (4 in) rainfall per year, for using plastic sheeting for catchment or for lining underground storage areas, and for drip irrigation(26).

Waste Disposal and Sanitation. Early documentation on sanitation has been previously mentioned(2,4). The earliest records are archaeological, and include plumbing fixtures of the 2500 B.C. temple of Sahuré in Egypt, the 6th Century Cloaca Maxima in Rome, the "great drain" of the Athenian agora of the 5th Century, and the 4th Century cloaca of Alexandria(2,21,27). Many of the ancient sewers and drains, notably the Cloaca Maxima, are still in use.

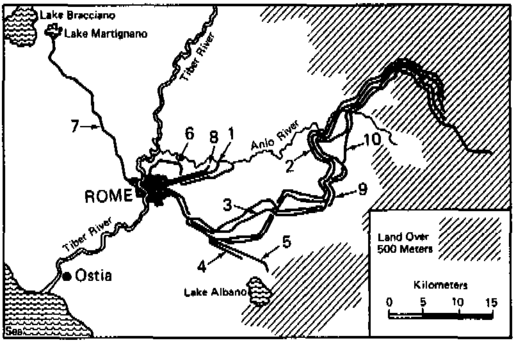


Figure 3. Roman aqueducts (after Forbes(8))

TABLE 1. MAJOR ROMAN AQUEDUCTS (9,19)*

Map No.	Year		Length km	Intake Elev, m	Capacity thousands of m^3/d
1	312 B.C.	Aqua Apia	16.6	40	76
2	272	Anio Vetus	63.6	264	183
3	144	Aqua Marcia	91.7	-	194
4	125	Aqua Tepula	∿6	-	-
5	40	Aqua Julia	22.8	350	50
6	19	Aqua Virgo	20.9	24	104
7	2	Aqua Alsietina	32.8	207	16
8	11 A.D.	Aqua Augusta	40	-	-
9	52 A.D.	Anio Novus	86.8	460	197
10	52 A.D.	Aqua Claudia	68.7	-	191

*Note that the Roman water system was a low pressure, continuous gravity flow, continuous flushing system.

Although sewers continued to be constructed throughout medieval and more recent times, it is mostly during the last 150 years that the idea of water borne sewerage systems has taken hold. This has been best documented by Stainbridge for England(7) and by Hardenburg in his 1942 standard text for the United States(28).

The flushing systems which solve individuals waste problems by giving them to someone else were explicitely recommended in 1842 by Edwin Chadwick in the first official British document dealing with community sanitation(29). Chadwick urged installation of water systems under pressure not so much to provide safe water but to flush the wastes. This says that the householder should have paid more for sewerage than for water supply. That he did not shows that both the supplier and the consumer perceived a need for water but not the need for waste disposal which the water created.

These perceptions became conventional wisdom when John Snow's classic work in 1854 showed the cause-effect relationship between a single polluted well and local incidence of cholera(30). Costs of sewerage systems have accordingly been borne by the community through cross-sectoral transfers of funds. Because it has been cheaper to treat water supplies than to treat wastes, investments in waste treatment have been deferred, so that the streams and lakes which supply the water have become polluted.

Local community perceptions of need still contrast with regional or national ones. In central American villages, a convenient water supply has first priority; sanitation may become important when it is a precondition for getting water or a health clinic. In crowded urban areas, privacy rather than health provides the incentive for sanitation(31). Conventional wisdom about the positive value to women of socializing at the village well or laundry area appears to be a pervasive myth. Women in central America, at least, prefer to draw and use water in private because there are too many quarrels at public taps.

Meanwhile, a series of increasingly complex technologies have been developing for water supply. The former have advanced from settling basins used in ancient Greece to modern water filtration, softening, and chlorination plants. Now we have additional processes which will completely remove the chlorinated hydrocarbons created by chlorinating the water for pathogen control.

On the waste disposal side, on-site disposal advanced succesively to simple dilution in receiving waters, screening plants, primary (sedimentation) plants, secondary (biological) treatment plants, and advanced waste treatment plants using one or more physical, chemical, or biological processes on secondary plant effluent. There are locations where nightsoil is diluted so that the solids can be treated and subsequently reconcentrated by conventional activated sludge methods.

Sludge disposal has become increasingly difficult because of increasing amounts of toxic heavy metals (mercury, cadmium, etc.) and halogenated hydrocarbons (PCB's, PPB's, dioxin, etc.) which enter the system as industrial wastes or as consumer product residuals. Treatment and/or disposal methods currently include pipeline discharge to the ocean,

ocean dumping, landfilling, landspreading, composting, incineration, pyrolysis, the Carver-Greenfield process, and wet oxidation by the Porteous or Zimmerman process. Costs of capital recovery, operation, and maintenance range from $9 to >800 per dry ton.

Imaginative waste disposal schemes abound. One household scale proposal called for electrical incinerating toilets in a southeast Asian city as a substitute for sewers; apparently overlooked was the fact that many of the poorer householders had bootlegged their electrical connections in the first place with some householders electrocuting themselves in the process.

At the other end of the scale, a number of initiatives by U.S. institutions call for using empty supertanker sludge capacity to ship sludge to desert lands in foreign countries. Such schemes have merit only as part of a larger program of utilizing both local and imported wastes for revegetation or harvesting of food, fiber, or fuel. Where dischargers are the main proponents, the schemes would provide primarily for the ultimate in solving waste disposal problems by transportation.

Recent developments in industrial countries indicate that the technological peaks in water and waste treatment may be at hand. Evidence includes water treatment plants which need only two valves to operate and which do not need the conventional pipe galleries and operating consoles(32). Water use has been remarkably reduced in response to the recent California drought where, for example, Marin County residents reduced consumption from about 100 gcd (380 lcd) to 35 gcd (130 lcd). The latter is still comfortably higher than the 20 to 40 lcd needed to realize the health benefits of a safe water supply.

Simpler methods of sewage treatment include ponds and land spreading(33). Sludge treatment by composting in aerated piles which are based on use of 1/3-horsepower electric blowers have been demonstrated in Washington, D.C., Windsor, Ont., Bangor, ME, and Durham, N.H.(34).

These recent developments may support the hope of taxpayers as expressed in California's recent Proposition 13 that less expensive municipal services are possible. However, for lower waste management costs to be realized by municipalities, the determination for greater local participation and self-sufficiency implied by Proposition 13 will need to be implemented by programs in which more careful, less "convenient" practices are followed by the same taxpayers in minimizing the amount of wastes which they turn over to their cities to handle.

Technological options for waste disposal in developing countries are generally expanded by the limited quantities of materials which are thrown away and limited by the resources available to deal with them. In contrast the lack of a materials policy in the United States(35) has created increasingly complex series of technological problems in environmental pollution. These problems have shown no evidence of being amenable to technological solutions which appear viable to corporate planners. There has been increasing political response, the most recent being the Toxic Substances Control Act of 1976 and one which represents only a partial development of a materials policy. Two points can be made. The first is that the political response is just that: it is not an initiative. The second point is that a national materials policy developed with due regard for the National Environmental Policy Act of 1969 is a logical approach to total resource development, use, and conservation. Both points indicate the importance of environmentally comprehensive corporate and government planning.

The most important element in planning is to consider the costs and benefits of conventional and unconventional alternatives. An example drawn from sanitation practices in both developing and industrial countries is summarized in the appendix. Expensive conventional sewerage systems are being introduced into developing countries while examples of mechanized collection from improved bucket and vault systems are found in industrial ones. The former provide greatest convenience at highest cost while the latter provide essential services at low cost and where system capacity closely matches demand. Alternative schemes for incremental upgrading of sanitation services are also presented in the appendix.

Socioeconomic Thresholds. Optimizing responses to the technological, environmental, and financial criteria for water supply and waste disposal systems is within the purview of conventional civil engineering practice. The success or failure of a particular project, however, will depend ultimately upon the willingness of the beneficiary individuals and community to pay for the improved service levels. For example, domestic water service levels of 20 to 40 lcd (say, up to 10 gcd) are needed for health benefits. Higher levels provide for aesthetics or convenience which are logically paid for by the users, although increasing block tariffs can provide for some community support. Meanwhile, community support of the amount for basic needs is reasonable. In any case willingness to pay in either industrial or developing countries requires early involvement of all parties in service-level decisions.

Other economic determinants such as employment opportunity have been discussed by a number of specialists. Probably the most significant statement was made in economic terms by E.F. Shumacher, when he popularized Mohatma Ghandi's dictum that small is beautiful(36). While Shumacher's analysis was directed at development in the third world, there is increasing local, State, Federal, and industrial research institute interest in applying his philosophy in the United States.

A special class of thresholds arises from economic development. Lee(37) has observed that when a country changes from agricultural support of industry to industrial support of agriculture, a number of things happen. For example, the incomes of families on subsistance farms begin to include non-farm income and the farmers lose interest in using nightsoil as fertilizer and switch to chemical fertilizer. Some years later, an environmental constraint may develop. In response to presumably careful observation of soil characteristics and fertility, some farmers in Japan are now going to considerable lengths to obtain nightsoil from urban areas in the face of official and legal opposition in order to restore the tilth of the soil.

Meanwhile, economic analysis in which social costs are included provides the tools by which the tradeoff between efficiency and equality can be resolved for engineering projects(38,39).

Institutional Thresholds. There may be some comfort in observing that people and their institutions do not change much. For example, when Frontinus found that 39% of the water entering the Aqua Claudia shown on Figure 3 was being stolen (a percentage not far from present losses in some water systems), his regulatory and enforcement response was the same as would be required today. The basic problem was that the illegal connections were made in the belief that the water would not be missed from the 1100 lcd (300 gcd) supplied to the city, most of which was used to supply and continuously flush the baths, cisterns, fountains, and other elements of the system(9,10).

From a financial point of view, Rome at the time of its aqueduct and sewer expansion was also enjoying its period of greatest geographic expansion when the institutions for collecting correspondingly increasing revenues were quite efficient. Similarly, the 19th Century development of water and sanitation systems in England occurred during the period of expansion of the British Empire and diversion of revenues from the East India Company to the Crown. It appears that a rapidly expanding economy is a necessary and perhaps a sufficient condition for extravagent use of water to flush wastes.

However, prosperity comes and goes. During periods of increasing competition for funds, the observation of Warford and Julius(40) is more to the point. They suggest that, at least in the field of resource economics, the reverse transfer of technology and economic advice from less developed to the more developed countries may be needed to solve the physical and financial constraints which may be appearing in New York or London. Specifically they note that water conservation will become increasingly necessary and that the most effective ways for doing this include increasing block tariffs so that water used in excess of personal hygiene requirements will be priced at its full marginal costs. The institutions needed for this transformation are not yet in hand, particularly when New York City, for example, does not meter water. However, models either exist or can be developed to meet these appropriate goals. An important constraint is that water conservation, a matter of mounting concern in these days when fewer and fewer people are willing to have someone else's wastes dumped on or near them.

Meanwhile, it is appropriate to examine the philosophy underlying a recent threshold in regulation of waste waters, the Federal Water Pollution Control Act PL 92-500. This law, with its 1977 amendments,

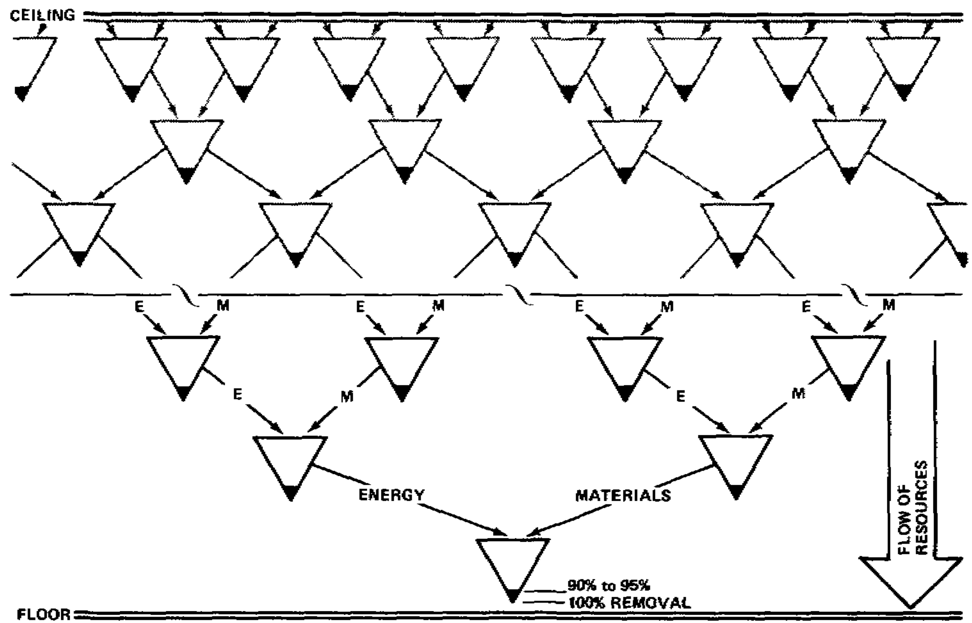


Figure 4. The (almost) zero pollutant discharge model. Emphasis on removal from the waste stream rather than separation at the source creates a unidirectional flow of resources.

is clearly a watershed act of legislation. The expressed goal of this legislation is the total elimination of water pollution. To this end, National Pollutant Discharge Elimination System (NDPES) permits are issued to all public and private dischargers. The zero-discharge criterion for all plants, regardless of location, has been found both administratively and judicially to assure financial and legal equity in pollution control; the law was not intended to assure minimum environmental impacts. Thus a water pollution cure may create a worse air pollution problem(41).

The characteristics of a system requiring 100 percent removals (or for that matter, 90 or 95 percent) are shown on Figure 4. The first-order municipal or industrial discharge is at the first level of the hierarchy. Zero pollutant discharge can be realized with sufficient inputs of energy and materials. Sources of these inputs are themselves (2nd order) dischargers to whom the same criteria apply, with a second generation of energy and material inputs. And so on. The unidirectional flow of energy and materials in a finite world makes the system a technological analog of a pyramid scheme. Like a chain letter, it is liable to collapse at, say, the 3rd or 4th level because of competition for resources. Similarly, reliance on purely financial and technological methods to eliminate pollutants from waste streams becomes unstable.

Summary and Recommendations

The historical and recent case studies, anecdotes, and analyses presented above can be summarized in the general model for technological choice shown in Figure 5. This model was adapted from one developed by Butzer in a study of hydraulic civilization in the Nile Valley(42).

The model shows the role of engineering feasibility studies in determining the independent variables related to people, available technologies, resources, their environmental constraints (on, among other things, locations of facilities) and willingness to pay. After the latter has been determined with the participation of the project beneficiaries, service levels can be established, technologies selected and implemented, and project benefits realized. Note that beneficiaries include regional or national communities or institutions who bear the cost of meeting basic needs and the local communities or households who pay for convenience or luxury.

Each of the independent and dependent variables important in water supply and waste disposal systems includes a number of components. Independent population variables include (1) demography (where thresholds and constraints are determined by population numbers, density, and growth and land tenure), (2) health status or threat with respect to individual diseases (supply of pathogens, latency or route of infection, persistence in environment, infective dose (demand) of pathogens, virulence, and immunity), (3) Culture and social status (social differentiation, decision-making roles, individual and group experience, and taboos). Other variables may be identified for specific cases. Elements listed in parentheses represent thresholds or constraints to be considered.

Resources available for any given project or program include the human ones (managerial, professional, technical, and administrative skills), communications media, transportation networks, energy, food, materials, finance, space, time, and institutions. Available technologies include the whole spectrum of past and present machines, structures, and methods. For water supply they range from buckets to distant impoundments aqueducts, terminal reservoirs and local distribution grids. For excreta disposal, they range from wrap-and-toss to activated sludge followed by sludge pyrolysis. Upgrading existing facilities will often provide the lowest cost improvements to service levels. For this to be realized, engineering fees should be based on time and materials rather than on a percentage of construction costs. In this way, engineers would be paid more for what they do than for what they sell.

Independent environmental variables usually are constraints in technological choice and include climate (distribution of temperature and precipitation, aridity and runoff), geology (topography, structure, soil permeability and fertility, and water table), and hazards (drought, flood, earthquake, vulcanism, fire and, because its effects are much the same as those from other hazards, warfare). Note that public health variables such as distribution of malaria are often fixed by environmental ones. Policy constraints include those fixed by governments which deal with resource allocation, generation of employment, monetary controls, etc.

The deciding independent variable is willingness to pay. Its determination requires active participation by agency officials and project beneficiaries with the engineering, physical, biological and social scientists and the financial, economics, and management specialists who prepare feasibility studies and appraisals.

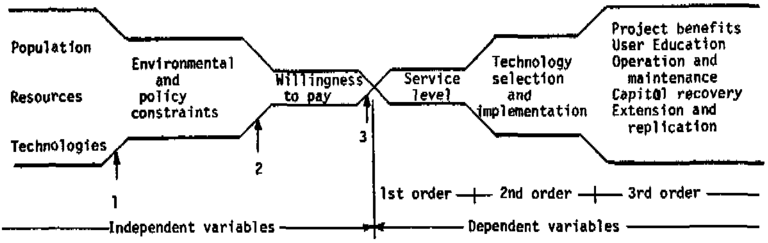


Figure 5. Independent and dependent variables in technological choice. Numbered arrows indicate times for public disclosure and participation in service level decisions.

The dependent variables in Figure 5 follow logically from the determination of willingness to pay. Continued public participation and user satisfaction are essential for capital recovery and in O&M functions and funding.

Recommendations. If water supply and waste disposal project and program planning are to be most effective, the following elements should be considered:

(1) Existing or traditional technologies are often cost-effective and frequently can be transferred or upgraded in stages as funds become available.

(2) The spectrum of environmental, technological, population, and resource variables and their interactions control beneficiaries' willingness to pay. Failure by planners and engineers to identify and design for economic, behavioral and environmental thresholds will almost certainly result in regulatory responses which are inherently inflationary.

REFERENCES

(1) McGarry, M.G. 1977: Appropriate Technology in Civil Engineering. In Environmental Impacts of International Civil Engineering Projects and Practices, Amer. Soc. Civil Engrs, New York, 1978, pp. 202-231.

(2) Lanciani, R.A. 1892: Ancient Rome in the Light of Recent Discoveries, 8th Ed., Boston. See also L. Mumford, 1961: The City in History, Harcourt, Brace & World, New York, pp. 213-221.

(3) Deuteronomy 23:12.

(4) Memford, Lewis, 1961: The City in History, Harcourt, Brace & World, New York.

(5) Socrates. Athenian Constitution, Chap. 50.

(6) King, F.H. 1906: Farmers of 40 Centuries, Toronto.

(7) Stainbridge, H.H. 1976: History of Sewage Treatment in Britain. Inst. of Water Pollution Control, Maidstone, Kent, England.

(8) Gunnerson, C.G., D. Julius and J.M. Kalbermatten, 14 June 1978 (in press) "Alternative Approaches to Sanitation Technology" Proc. Workshop on Water Pollution Problems Arising from Development, International Association for Water Pollution Research.

(9) Forbes, R.S. 1964: Studies in Ancient Technology, 2nd Ed., E.J. Brill, London.

(10) Ashby, Thomas, 1935: The Aqueducts of Ancient Rome, Clarendon Press, Oxford.

(11) Cornell, Nina, 1978: Lectures at Brookings Institution.

(12) Leith, Helmut, 1975: Primary Productivity of the Biosphere, Springer-Verlag, Berlin.

(13) Mara, Duncan, 1976: Sewage Treatment in Hot Climates, Wiley, New York.

(14) Eckholm, Erik and L.R. Brown, 1977: Spreading Desert - The Hand of Man, Paper 13, Worldwatch, Washington.

(15) Eckholm, Erik, 1978: "Firewood - The Poor Man's Burden". International Wildlife, vol 8, no 3, pp.20-26.

(16) Wagner, E.G. and J.N. Lanoix, 1959: Water Supply for Rural Areas and Small Communities, World Health Organization, Geneva.

(17) Wagner, E.G. and J.N. Lanoix 1958: Excreta Disposal for Rural Areas and Small Communities, World Health Organization, Geneva.

(18) Feachem, Richard and Sandy Cairncross, 1978: Small Excreta Disposal Systems, Bull No. 8, Ross Inst. Tropical Hygiene, London.

(19) Cairncross, Sandy and Richard Feachem, 1978: Small Water Supplies, Bull No. 10, Ross Inst. Tropical Hygiene, London.

(20) Winneburger, J.H.T. 1974: Manual of Grey Water Treatment Practice, Ann Arbor Science, Ann Arbor, Michigan.

(21) Forbes, R.J. 1957: "Hydraulics and Sanitation", ch 19 in A History of Technology, Charles Singer, et al, eds Oxford, pp 663-694.

(22) Hendrie, Keith, 1976: "A Chronology of Climatic Variations in and Near the Main Dry Regions." App II in F. Kenneth Hare, Climate and Desertification, Inst. for Environmental Studies, Univ. of Toronto, Canada.

(23) Bell, Barbara, 1975: "The Dark Ages in Ancient History, " Amer. Journal of Archaeology, vol 75, no 1, pp 1-26.

(24) Biswas, A.K. 1970: History of Hydrology, North-Holland Publishing Co., Amsterdam.

(25) Garbrecht, G. u. G. Holtoroff, 1978: "Waserwirt-schaftliche Analagen des antiken Pergamon-die Madradag-Leitung." Mitteilung Heft 37, Leichweiss-Institut für Wasserbau der Technischen Universität, Braunscheweig.

(26) National Academy of Sciences, 1974: More Water for Arid Lands, Washington, D.C.

(27) Der Kleine Pauly: Lexikon der Antike, 1966: Alfred Druckenmüller Verlag, Stuttgart.

(28) Hardenbergh, W.A. 1942: Sewerage and Sewage Treatment, Scanton, PA.

(29) Chadwick, Edwin, 1842: Report on the Sanitary Condition of the Labouring Population in Great Britain. Poor Land Commission. Reprinted, Edinburgh Univ. Press, Edinburgh, 1965.

(30) Snow, John, 1855: On The Mode of Communication of Cholera, London. Reprinted in Snow on Cholera, Oxford University Press, London, 1936.

(31) Buckles, P.K. 1978: "Introduction of Potable Water and Latrines - A Case Study of Two Rural Communities in Guatemala". Consultant's report to the World Bank, Washington, D.C.

(32) MacDonald, D.V., and Lee Streicher, Aug. 1977: Water Treatment Plant is Cost Effective. Public Works.

(33) Process Design Manual for Land Treatment of Municipal Wastewater, 1977: U.S. Environmental Protection Agency, Pub EPA 625/1-77-008, Washington,

(34) Shuval, H.I., C.G. Gunnerson and D. Julius, "Night Soil Composting," P.U. Report No. RES 12, Energy, Water and Telecommunications Department, The World Bank, Washington, D.C. July 1978.

(35) Holt, J.A., Ed. Engineering Implications of Chronic Materials Scarcity, Office of Technology Assessment, 1977, and Materials Policy Handbook, Comm on Sci and Technol, US Congress, Washington, 1977.

(36) Shumacher, E.F. 1971: Small is Beautiful, Harper & Row, New York.

(37) Lee, Robert C.T., Personal communication, 1977.

(38) Okun, A.M. 1975: Equality and Efficiency: The Big Tradeoff. The Brookings Institution, Washington, D.C.

(39) Squire, Lyn and H. vanderTak, 1975: Economic Analysis of Projects. Johns Hopkins University Press, Baltimore.

(40) Warford, J.J. and DeAnne Julius, 1977: "The Multiple Objectives of Water Rate Policy in Less Developed Countries". Water Supply and Management, Vol 1, pp 335-342.

(41) Garber, W.F., G.T. Ohara and S.K. Raksit, 1975: "Energy-Wastewater Treatment and Solids Disposal". J. Environmental Engineering Division, ASCE, Proc Vol 101, No. EE3, pp 319-332.

(42) Butzer, K.W. 1976: Early Hydraulic Civilization in Egypt. University of Chicago Press.

APPENDIX. CLASSIFICATION AND UPGRADING OF SANITATION SYSTEMS

Selection of an appropriate sanitation technology begins with consideration of the physical characteristics of alternative on-site and off-site structures and the ways in which they can be upgraded. Figure A1 presents a generic classification of sanitation systems for developing countries. It includes systems which are in use or which can include the minor modifications shown. It does not include exotic household schemes such as incinerating toilets or recirculating toilets which are too expensive and energy intensive or systems for adding proprietary microbial cultures or enzymes which are unnecessary.

System suitability for household or community excreta disposal and for sullage disposal is indicated. The upgradability index is a measure of technological feasibility of upgrading. The following notes apply to individual technologies.

Sanitation System No.	Remarks
(1)	Overhung latrines are sometimes used in southeast Asia to fertilize fish ponds.
(2)	Variations include Feuillée
(3)	Variations include shallow ($\leq$4m), deep (>4m), wet (where decomposition of solids is most rapid) dry (similar to soakaway), lined, unlined, with or without plinth.
(4)	Reed Odorless Earth Closet; occasional problems from excreta collecting on chute.
(5)	With proper maintenance can provide same health benefits as properly maintained full plumbing and sewerage.

(6) Variations include fixed double superstructure, moveable superstructure placed over several shallow (1/2m) pit sections built in plinth in high water table areas, and provision for separate collection of urine for fertilizer. In all cases, attention must be paid to Carbon: Nitrogen ratio which can be controlled by excluding urine or by adding leaves or straw. Moisture content must also be controlled, usually by adding straw or similar material.

(7) Variations include multrum, modified Roec, Mullbank (heated to 60^{o}C), Sanitherm (heated to 70^{o}C), Mull-toa, Minimus, Utafiti and others. All have same operating requirements as batch composters.

(8) Variations include gravity-operated metal gates which close automatically. Usually 1/2 to 1 1/2 ℓ per flush. Water-seal latrines cannot be used where rocks, mud-balls, maize cobs, etc. are used for anal cleansing.

(9) Almost continual problems of maintaing water seal.

(10) Also known as Chinese 3-compartment septic tank where effluent slurry is used directly on fields.

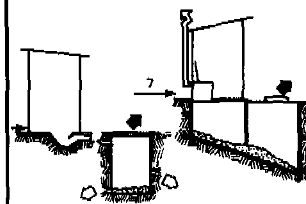
(11) Variations include Botswana Type-B toilet.

(12) Variations include U.S. septic closet, Khatgar latrine.

(13) Conventional high-volume cistern flush.

(14, 15,16) Variations include high volume flush discharging to large-volume soakaway (sumidero) in central America, sometimes installed for temporary use pending sewer connection.

SANITATION SYSTEM			Upgradeability Index	Household	Communal	Sullage	
ON-SITE	DRY	1 Overhung latrine	5	●	●		
		2 Trench latrine	5	●			
		3 Pit latrine	4	●			
		4 R O E C	3	●			
		5 Ventilated improved pit latrine	3	●			
		6 Batch composting latrine	4	●			
		7 Continuous composting latrine	3	●			
	WET	8 PF latrine, soakaway	3	●			
		9 PF latrine, aquaprivy, soakaway	3	●			
		10 PF, septic tank, vault	3	●			
		11 Sullage flush, aquaprivy, soakaway	3	●		●	
		12 Sullage flush, septic tank, soakaway	3	●		●	
		13 Conventional septic tank	2	●		●	→ Same as 12 except conventional cistern flush
ON- or OFF-SITE	WET	14 *Low-volume cistern* flush, soakaway *or* sewer	1	●	●	●	Same as corresponding configuration in 8 to 12 except for elevated cistern with low volume flush
		15 Low-volume cistern flush, aquaprivy, soakaway *or* sewer	1	●	●	●	
		16 Low-volume cistern flush, septic tank, soakaway *or* sewer	1	●	●	●	
OFF-SITE	WET	17 Conventional sewerage	2	●	●	●	→ See standard manuals and texts
	DRY	18 Vault and vacuum truck	1	●	●		
		19 Vault, manual removal, truck or cart	4	●			
		20 Bucket latrine	5	●			
		21 Mechanized bucket latrine	2	●	●		



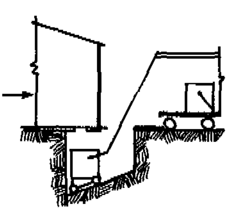
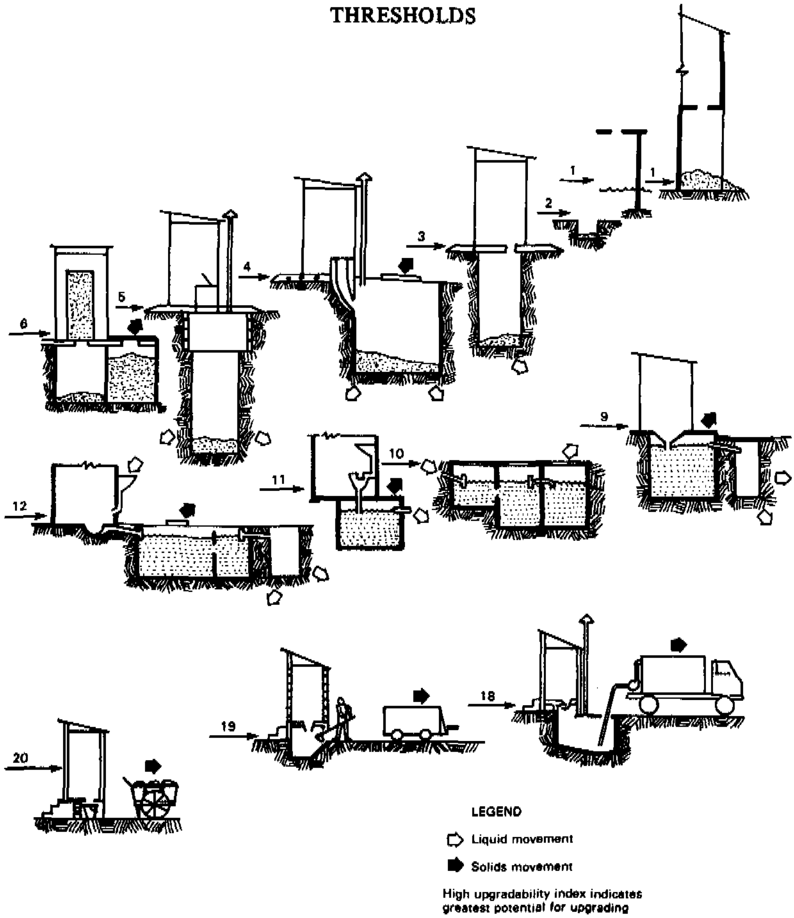


FIGURE A1. GENERIC CLASSIFICATION OF



SANITATION SYSTEMS

(17) 12-20 ℓ per flush

(18,19) Quantity of night soil collected varies with diet and with construction of vault which may lose or gain water from seepage. Maximum solids content for pumping is about 12%.

(20) Crowding and access in some areas are such that improved systems in which buckets are covered, removed, and sterilized are the best alternative.

(21) Variation of pan system developed in Sydney, Australia.

Table A1 lists a number of upgrading sequences in which improvements in health protection, odor, control, convenience, and potential realiability are provided. Note that change from a communal to a household system will ordinarily be preferred by the user because of additional privacy.

Table A1. Examples of Sanitation Upgrading Sequences

Generic Classification	Technology (identification)
Off-site to off-site	Bucket latrine (20) to vault with manual removal (19) to vault with mechanical removal (18)
Off-site to on-site	Bucket latrine (20) to ventilated improved pit latrine (5) or sullage or pour-flush with soakaway (12 or 8)
On-site to on-site	Pit latrine (4) to VIP (5) to PF latrine (8) to aquaprivy with soakaway (9).
On-site to off-site	VIP (5) to vault and vacuum truck (1)
On-site to off-site	LV cistern flush with soakaway (14) to LV cistern flush with sewer (14).

BEHAVIORAL FACTORS IN SELECTION OF TECHNOLOGIES

By Gilbert F. White[1] and Anne U. White[1]

INTRODUCTION

Where a well-known technology is provided to a population that has long been acquainted with its use, the problems of predicting patterns of social behavior are relatively simple. The difficulties arise when a new technology is introduced to a population which is not accustomed to it. It is in these circumstances that there are few if any local lessons from the past to apply, so that it becomes important to rely either on comparable experience elsewhere or analysis based on established relationships of human behavior, preference and motivation. In this paper we point out some of the all too abundant evidence that behavioral factors may be critical to the success of new water and sanitation installations and to their effects on public health, note those situations in which behavioral factors in fact are critical, suggest some of the circumstances which affect the development of new patterns of behavior, and indicate those situations in which a system of community participation may avoid unnecessary expense or failure.

WHEN BEHAVIOR IS IGNORED

Although there have been only a few systematic studies of the circumstances in which new projects have failed because of lack of receptivity by

---

1. Professional Staff, Institute of Behavioral Science, University of Colorado at Boulder.

the user community, the literature is replete with anecdotal evidence of those events: the standpipe in Ethiopia that is destroyed by people living immediately adjacent to it and objecting to noise (1); the new water tap in the Ryūkyū Islands which is accompanied by a towel on which all of the children in the community wipe their hands, thus spreading trachoma (2); the communal latrine in Nigeria that goes unused. There are also the aquaprivies that are first used and then abandoned, and the faucets which are ripped off the pipes and the metal used to fashion ornaments. All of these are examples of misuse of facilities which had been installed with the intention of promoting health and well-being of the users. In each case something went wrong in the design or construction or operation and maintenance.

CRITICAL SITUATIONS

As noted at the outset, there are numerous situations in which attention to behavioral factors may be relatively perfunctory or routine. These are the situations where a population of known ethnic and economic composition is being presented with a new facility of exactly the same type as that which has been used enthusiastically by the same type of population elsewhere. In these circumstances the design engineer can replicate with confidence the design and administrative arrangements in the comparable place and assume that the community acceptance will be equivalent. This degree of confidence is warranted especially where the facilities which are being provided are the sole option open to the user as, for example, where a tap system is provided for a city population which has no other sources of water available from roof or ground. There the users have no choice of the source, although they may exercise some options as to how they misuse the equipment or waste the water.

In other situations the major components of the design situation are: 1) the range of possible physical design, 2) the prevailing behavior patterns of the potential users, 3) the facilities provided for community participation and 4) the nature of educational and informational activities which are carried out by the community authorities. The design includes not only the physical facilities of supply, treatment, transportation and distribution, but the administrative design for means of construction and maintenance, and the arrangement for pricing or other means of payment for the cost of the improvement.

Whatever the technique followed for design of the system, once it is installed people have four choices open to them: 1) they may use the facilities without any significant change in their current behavior patterns as, for example, when a family begins using a private courtyard latrine in preference to a community facility on which it earlier had depended; 2) They may change their behavioral habits in order to make use of the new facility, as when a family collects its excreta daily for collection by a cart rather than depositing it in a nearby field (the change here is to take advantage of a daily collection system); 3) they may misuse the facility, as when they throw garbage into the latrines or let the water from a community faucet run until the temporary supply is exhausted; 4) they may utterly reject the facility, as when they refuse to patronize a community latrine or go for water to a nearby stream in preference to a new well.

People who accept a new facility generally are required to make some kind of adjustment in their behavior pattern. This may involve their payments of charges for use of the facility or for amortization of the investment. Their contribution may have been preceded by contributions of labor before the project begins. They may have to learn to maintain the facility. They must learn to use it with care, and in addition they may be obliged to take action to assure that others do not misuse the facility. Any or all of these adaptations involve a change from their previous pattern but a change which contributes to the desired use of the facility and to its continued upkeep. All such actions may be termed supportive responses.

In contrast, there is an even larger range of destructive responses in which some or all of the members of a community may engage. The most obvious is the complete rejection of the facility and the continued use of whatever arrangements it was intended to replace. These usually have significant implications for health, for expenditures of time and energy for drawing water or disposing of excreta and waste, and for settlement patterns.

Even more common is the user who takes advantage of the facility for some period of time but who refuses to pay for its construction and who either refuses entirely or slowly retreats from payment towards the cost of operation and maintenance. In this situation the system may fall into disrepair without any other overt acts by the users.

Oftentimes, however, the refusal to contribute to operation and maintenance is accompanied by other actions abusing the system. These include failure to maintain a pump, delinquency in providing fuel or chlorine, letting leaking faucets continue to run, breaking off faucets for other uses, and so forth. There may occasionally be overt destructive acts as when some individual or group within a community sabotages the system, or encourages others to do so.

These acts of adaptation for constructive use or of delinquency and destructive use are easy to catalog and classify. It is more difficult to sort out the factors which account for these actions of support, misuse or rejection.

MOTIVATION AND PREFERENCES

From retrospective studies of individual community improvements prepared for the World Bank Research Project on Appropriate Technology for Water Supply and Waste Disposal, from our own field studies, and by inference from what is known about social behavior it is possible to identify at least half a dozen factors which affect the actions of users in the situations noted above. We shall note later on that it is a mistake to think that these can be assumed or defined for any community without some careful investigation of that particular community, and it should be recognized that a factor that may be highly important in one culture may have little or no significance in another. An example is the Swiss road construction firm which had to delay work in an Arab country because toilet facilities for male and female employees had been built next to each other (3).

Individual desire for privacy clearly is a widespread and powerful motivation. People generally prefer to be alone while defecating but this is not always the case, as witness the women in Indian villages who prefer to go to the fields together for defecation so as to have some protection and privacy that might otherwise be interrupted. There is a long and widespread belief that women like to go to common water sources in villages because of the opportunities which the journey and waiting in line provide for them to talk with friends and neighbors. In our investigations we have found that this is largely a myth. Women, given a choice, would prefer to have their own individual source. This does not mean that they necessarily resent meeting others or that they do not gain much in the way of information and companionship from others at the watering place. It does mean that when given an opportunity they will prefer their own source, but then will be obliged to obtain companionship and information through other meeting places and channels in the community. We have found this not only in the individual interviews with water users with a variety of cultural and physical environments in East Africa but in the responses from people questioned in a wide range of other cultures. This is important to recognize because it represents a powerful motive for enlistment of community support for any project which provides some increment in the degree of individual or family privacy in water supply and sanitation facilities. It does not mean, however, that users will automatically adopt such facilities when proferred: they may object to the facilities because of other factors, some of which are ennumerated below.

Individual inability to handle facilities is another factor at work. If the individual cannot easily make the facility work, as in the case of the pump which requires priming, there is a tendency for people to prefer equipment over which they have an easy and complete control, for example, a bucket and rope. If they lack the skill and knowledge to repair the equipment or to make it run properly, they may let it fall into disrepair, not because they are opposed to its use but because they feel inadequate to its use.

Status in the community, and especially the extent of sharing of a prevailing practice with one's neighbors, may be a powerful motivational force. The individuals may be reluctant to experiment with a new device unless there is some support from those who are known innovators in the community.

A sense of community well-being in doing what is expected of the individuals to maintain the wholeness of a community may be a strong motivation quite independent of the desire for maintenance of individual health and privacy. The Guatemala community which acts to make an improvement because it has been accepted as the community goal by the leaders is one example (4). Another example is the Spanish American community in New Mexico where a number of the members contribute to a water supply system because they do not wish to offend neighbors who are deeply committed to such improvements (5). The obverse is the unwillingness of people to cooperate in a project because there are objectionable persons or groups who are identified with them. Such considerations often explain why a community for no reason having to do with the project design or their own preferences rejects an improvement.

A sense of individual health has been shown to be a primary motivation for many users, but this urge always is mediated by the knowledge of the individuals responding to it. They may object to an improved water source or means of excreta disposal because they feel it is unhealthy although the sanitary experts may know the belief is unfounded (6). It is important to find out what they consider to be the criteria of health, as for example, finding out that members of a Filipino community regard intestinal worms as contributing to proper digestion of food.

Related to considerations of individual well-being is a sense of control over one's own activities, that is, the ability of individuals to exercise discrimination with respect to the quality of service which they will enjoy. From our experience, people when given the opportunity will exercise considerable discrimination in choosing water sources and means of excreta disposal. In Indonesia among the 1.3 million families who use community piped water supplies, at least 52 percent make the distinction between potable water for drinking and cooking purposes and unpotable water which they carry from rivers to use for washing and bathing purposes (7). This discrimination may be affected by their prior experience with development activity in the community. In some communities, for example, efforts at sanitation improvement are closely identified with earlier colonial authorities, and participation in the new improvement along the same lines is regarded as some kind of a sacrifice of individual control in the face of forces of colonialism.

Finally, it is important to recognize that every individual family or community has its own sense of priorities as to what needs to be done and as to the acceptable ways of doing it. They may reject a contribution towards a sanitation improvement not because they are against the improvement but because they think they can use their resources to better advantage in building a road or providing an agricultural produce marketing facility. They may use the brass faucet for creating jewelry rather than for water supply because the sense of individual status is overriding in those circumstances.

Other motivations and preferences might be noted, but this is sufficient to indicate their range and their power in influencing the degree of supportive and destructive response to a proposed innovation.

IGNORING COMMUNITY PREFERENCE

Our basic argument is that in most cases where new design facilities fail to gain community acceptance one or more of the factors ennumerated above is at work. It is possible to go into a community and assess why some facilities are abused and others rejected, but one would prefer not to be obliged to make this kind of retrospective appraisal. The effective tactic is to take preferences and motivations into account at the outset as an essential part of the design process. This involves some kind of community participation.

COMMUNITY PARTICIPATION

The appeal for community participation is widespread and frequently voiced, but there needs to be careful definition of what the term means and of specific modes of involving people in the design process. At base, it means providing an opportunity for users of a facility to have a voice in selecting the different elements in the facility. Giving them a voice implies providing for 1) a determination of their current preferences, 2) an estimate of their current ability to meet their perceived needs, 3) an estimate of their capacity to adapt to new facilities, and 4) an assessment of the likelihood of their maintaining the system in the face of changing preferences over time. It should not be expected that preferences and motivations would remain uniformly the same: they may be expected to change. It is possible to anticipate some of this change by analogy with other communities that have gone through social change as well as by reference to underlying preferences that are strong but are not satisfied by existing community arrangements.

OPPORTUNITIES FOR COMMUNITY PARTICIPATION IN DECISION-MAKING

The artistically difficult and trying judgment to make is to determine at what time to involve citizens in community decision-making about the new facility. Ideally, the initiation of any preliminary exploration of improvements would come in response to community initiative, with the community leaders asking for technical assistance in achieving a need which they already have recognized and which they are determined to satisfy. This rarely is the case because communities may have difficulty initially perceiving their needs, may place sanitation in very low priority at the outset, and may be unaware of steps which they could take to enlist technical assistance.

Figure 1 suggests the way in which various components of the decision situation may interrelate over time. In its condition of imposed design one sees community behavior patterns as remaining untapped by the engineers who proceed to make a preliminary and final design, then to construct the project and impose it on the community. Depending on the character of the prior studies or sheer luck the response may range from complete acceptance, to acceptance with a significant adaptation of behavior, to rejection. Many existing water supply systems fit into this pattern.

Where there is participation, the community may, as shown in the second case in Figure I, have a role in first deciding whether or not a project will be applied for, and then at three separate times in making its views known. Prospective users would have opportunity for a reaction to preliminary designs, for another reaction to final designs, and for participation in necessary preparation actions before the final design gets translated through construction into reality.

A third possibility is where the design is modified through community participation as above, but where the community comes to understand that it cannot achieve its ends within the limits of available funds and customary behavior. In this case the community decides that modifying its behavior patterns could allow it to achieve its ends, and that it is willing to make the necessary modifications.

An extreme example of behavior modification comes in the fourth case where a constructing authority, having designed a project, sounds out community preferences and in the light of this undertakes a program of community education to attempt to assure that the community will be prepared to modify its behavior so as to use the facility once constructed.

<u>Components in dealing with behavioral factors:</u>

Design: D_P – preliminary; D_F – final
Behavioral pattern: B_1 – *original;* B_2 – *modified*
Involvement: I_1 – community needs & promotion; I_2 – community preferences; I_3 community consultation; I_4 community education
Application decision: A
Construction decision: C

<u>Inter–action points</u>

Before applying for project
Design of project – Preliminary
Response to preliminary design
Design – Final
Response to final design
Response to construction

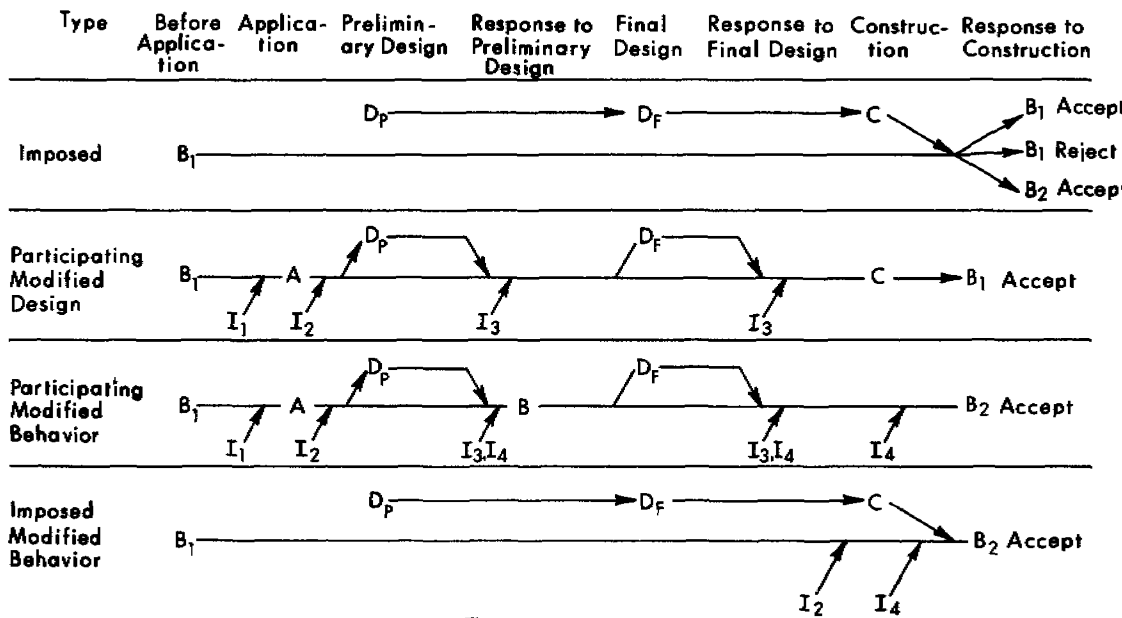


Fig. 1 Typical Flows

Health centers, or special programs like vaccination, have often taken this approach.

In brief, there are five major points at which local people can have a significant role in decision-making with respect to a new project. First, they may join in deciding whether or not to apply for a project and thereby to make at least a preliminary canvass of their own sense of priorities about the needs of the project and their own judgment as to their capacity to marshall the money, labor and local management facilities to carry it out.

Second, they may participate in the selection of technologies in relation to their preferences subject always to the constraints of available money and labor to carry out the undertaking.

Third, they may influence the pricing policies, including the provisions that are made for funding operation and maintenance. There has been a good deal of evidence that demand is relatively price-inelastic for such facilities, but also that any drastic increase in prices is likely to arouse substantial resistance over the short term. Local people can indicate what kinds of pricing schedules appeal to them, what modes of billing and payment are best adapted to their income flows, and what means of handling money or other contributions of labor are most acceptable to them.

Fourth, the local management policies may be most effective if adjusted at the outset to the modes of action within the community, including their formal governmental structure as well as the community networks through which information is transmitted and through which people join in arriving at judgments about priorities.

Finally, local people can be of assistance in monitoring the construction and operation and maintenance of a project, thereby influencing the degree to which it ultimately meets its stated aims.

DEVICES FOR ASSURING COMMUNITY PARTICIPATION

In another paper (8), we have discussed some of the tools that can be employed in carrying out a project so as to assure effective community participation at the points noted above. Here we recapitulate in capsule form what seem to be the chief devices that are readily available to engineers who are interested in avoiding failure or inefficiency in use. These involve chiefly two types of community survey, community consultation, and provision for continued monitoring.

The most informal sort of community survey is one in which an individual or team enters a community and carries on a series of informal discussions with selected members of the community and its official leaders and with people who are well acquainted with the community. This can yield preliminary judgments as to how the community views its own health needs and how it places these in priorities among other community needs, as to its facility for mobilizing some kind of community action, and as to the assortment of existing networks of communication and power. The informal survey may lead to the judgment that the time is not right for further action, it may suggest that a substantial amount of community education and promotional work may be necessary before further action can be taken, or it may suggest that the time is right for a more intensive kind of community survey looking to assistance in the design of a project.

Always related to the preliminary inquiry is the assembly of whatever information of a statistical or anecdotal type is available about the community from people who have worked in it. Oftentimes a story or chance conversation may give insight as to forces which are at work in the community, but they never should be accepted as fully descriptive of the community situation. That can only be determined with any great confidence by a more careful sample of the total population.

It is not necessary to carry out an elaborate census of members of the community. A carefully selected sample stratified according to differences in income and in social groups may reveal a good deal about the range of motivations and preferences within the community and about the significant social networks.

From the results of the survey of as few as 30 families in a large village it is possible to compose a relatively accurate profile of the community preferences of the type shown in Figures 2 and 3. This indicates that the two communities illustrated, one in Haiti and one in the Sudan, have major differences in their views of their health conditions, the need for improvements, and their willingness to participate.

The survey may also reveal significant relationships between social factors and likely participation in the project. Thus, one finds that in community A there is strong preference for using an individual source of water supply whereas community B is less concerned about this factor. One might also learn that it is the people without latrines and of the middle and upper income groups who are most likely to support an improvement for excreta disposal in community C.

The results of such community survey can be invaluable to the designer in understanding the current practices and information level of the community, its preferences regarding services, and how these might be met by design or how they would require modification through education and consultation if an effective design is to be accepted.

Community consultation may be expected to follow whenever there is a preliminary design for review with community leaders. Here it is helpful to recognize that the designated or elected leaders of a community are not always those who are most sensitive to or acquainted with the wishes or capacities of its people. Indeed, it may happen that particular officials, persons or cadres of people in a community are so unpopular

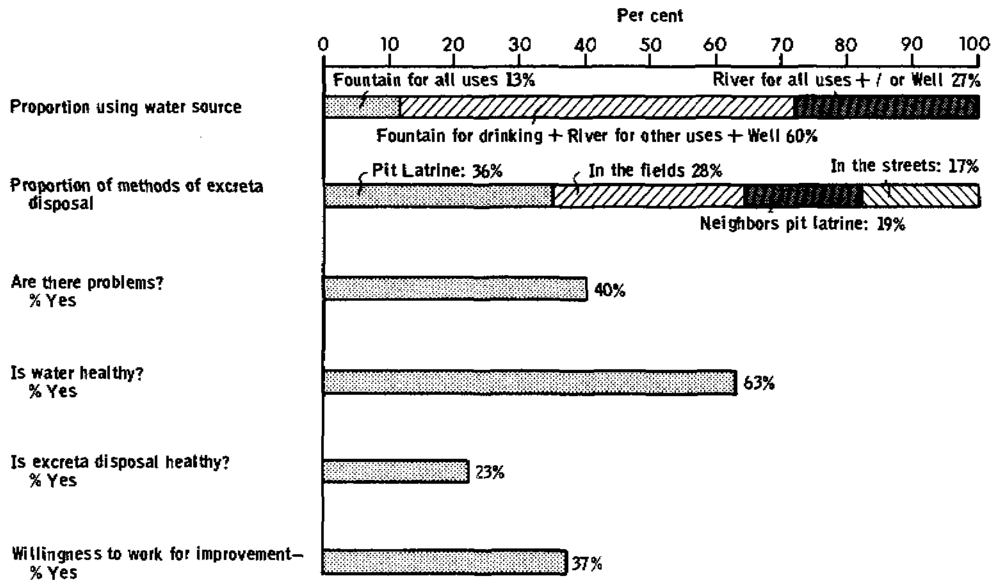


Fig. 2 Example of profile based upon community survey: Haiti, village with unpiped water. N = 30.

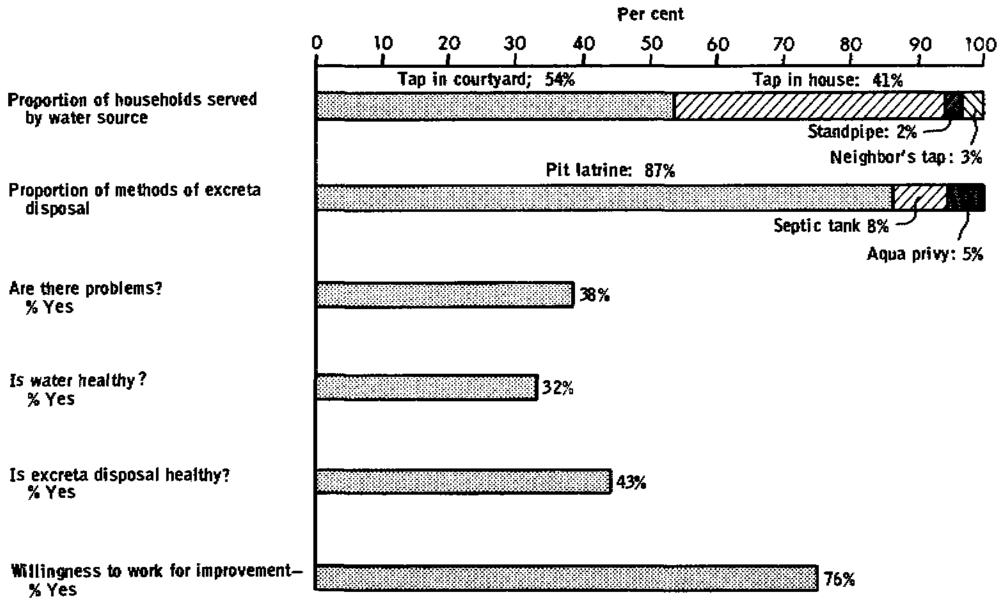


Fig. 3. Example of profile based upon community survey: Sudan, village with piped and unpiped water. N=37.

that associating a project with them is assurance of failure at the outset. It is important to determine who are the significant people in the community in terms of passing on information and deriving community judgments. From this information which will have been learned in the community survey it is possible to see to it that representatives of those groups are drawn into consultation about the preliminary design and, subsequently, consultation about the final design.

It usually is desirable to present two or more but not more than five alternative choices to the community. It will be practicable to know what kinds of information will be significant for the people in terms of their previously expressed preferences and their indications of how much they would be willing to pay or contribute and with what groups they would be willing to work. The people need to be drawn into the system of consultation if they are to act on the commitments made by representatives selected as having responsibility. It is essential to learn what kinds of concrete evidence they would like about the alternatives available: what it is that they need to see or touch or have more details about before they can make any judgment about it. It also is important for them to be entirely cognizant of what the costs will be in terms of time, money, or labor.

One of the cautions, beyond those noted above, which needs to be in the minds of those who participate in community consultation is that where there seems to be an important discrepancy between preferences and design it is wise to be cautious about the extent to which the gap can be bridged by information or educational activities. Oftentimes an easy solution is to suggest that printed materials will be distributed or a motion picture will be shown or that community leaders will be informed about reasons for a design feature which is out of harmony with the community preferences. Such educational programs sometimes work, but often they do not, and it is desirable to be skeptical about the extent to which they will be effective in a particular community. Suggestions of the care and continuity needed to make such programs effective can be found in some manuals such as those for rural health education in Kenya (9), or for the promotion of rural water projects in Columbia (10).

There is no sure panacea or set of dos and don'ts for examining these factors affecting behavioral patterns. Unwillingness to make the effort no doubt is the most common cause of failure of community water supply and sanitation improvements. Rarely would one encounter a situation in which all of the factors noted above would be significant or in which all of the steps noted above would be required in order to achieve a thoroughly effective community plan of improvement. The essential feature of the approach is to assume at the outset that factors affecting behavioral patterns need to be taken into account, that community participation can assist in determining them, and that such effort will generate greater assurance that the completed project will serve its intended aims.

ABSTRACT: Behavioral factors such as motivation and preference may be critical to the success of new water supply and sanitation projects and their effect on public health. Once the facilities are installed, the users have the options of adopting the new facilities without significant change in behavior patterns, changing behavior patterns to use the facilities, misusing the facilities, or rejecting their use entirely. If it is assumed at the onset of a project that behavioral factors should be taken into account, community participation can assist in determining these. Local people can have a significant role in decision-making at five points in the development of a new project: the decision to apply for assistance in terms of their own priorities, the selection of technologies within the constraints of available money and labor, the selection of pricing policies adapted to their needs, the incorporation of local modes of action into management policies and information transfer activities, and in monitoring methods. Tools for community participation include two types of surveys, community consultation, and provisions for continuing monitoring.

KEY WORDS: Water supply, public health, sanitation, community participation, decision-making, technology, choice.

REFERENCES

1. Beyene, Atnafe, "Planning Considerations for Rural Water Supply in Developing Countries", unpublished paper prepared for the Ethiopian Water Resources Authority, 1978.

2. Marshall, Carter L., "Some Exercises in Social Ecology: Health, Disease and Modernization in the Ryukyu Islands", in The Careless Technology, ed. M. T. Farvar and J. P. Milton. New York: Natural History Press, 1972, pp. 5-18.

3. van Wijk, Christine, -Sybesma, Bibliography on Extension and Community Participation in Community Water Supply and Sanitation, Preliminary Draft, WHO International Reference Centre for Community Water Supply, 197803, The Hague, n.d., p. 41.

4. Miller, Frank A. and Cynthia A. Cone, "Latrines in Yaluc: A Twenty-Year Perspective," paper prepared for the Appropriate Technology for Water Supply and Waste Disposal Research Project, World Bank, 1978.

5. Olinger, Colleen E., "Domestic Water Use in the Española Valley, New Mexico: A Study of Resource Decision-Making," Master of Arts thesis, Geography, University of Chicago, Chicago, Illinois, 1970.

6. White, Gilbert F., David J. Bradley, and Anne U. White, Drawers of Water: Domestic Water Use in East Africa, Chicago: University of Chicago Press, 1972.

7. Indonesia, Biro Pusat Statistik, Household Condition in Indonesia, September-December, 1976, 1976 National Labor Force Survey, NEI 78, 1978.

8. White, Anne U. and Gilbert F. "Community Assessment of Water Supply and Sanitation Options for Developing Countries," paper prepared for the Appropriate Technology for Water and Waste Disposal in Developing Countries Research Project, World Bank, 1978.

9. Scotney, Norman, Health Education, African Medical and Research Foundation, Rural Health Series 3, Nairobi, Kenya, 1976. This manual for rural health workers indicates an approach of listening and learning versus directive teaching in the rural health field.

10. Manual de Procedimientos en Promocion Communitaria para el Programa Nacional de Saneamiento Basico Rural, Instituto Nacional Para Programas Especiales de Salud, S.B.R. no. 0072, Bogata, Columbia, 1975.

ECONOMIC INCENTIVES AND APPROPRIATE TECHNOLOGY IN VILLAGE WATER SUPPLY

by
Robert J. Saunders and Jeremy J. Warford[1/]

Introduction

This paper discusses the use of appropriate technology in the village water supply sector in developing countries. Appropriate technology may be defined to cover both the technical alternatives of supplying a given standard of service, as well as appropriate variations in service levels or standards.

The appropriate means of constructing a village water supply scheme can be defined as the least social cost means of doing so, where costs are defined as the opportunity costs to the economy as a whole rather than simply the financial costs to the village water supply authority. Shadow prices for labor, capital, and foreign exchange should therefore be used in determining whether or not a project makes use of appropriate technology.

Inappropriateness in designing and constructing village water supply schemes may be symptomatic of a number of things:

(a) lack of knowledge of the socially cheapest means of providing acceptable water;

(b) pressure from foreign suppliers, contracting firms, consultants, as well as from local engineers who are trained in sophisticated technologies; and

(c) the system of incentives.

1/ The authors are respectively Chief, Telecommunications Division and Economic Adviser, Energy, Water and Telecommunications Department, the World Bank. The opinions expressed in this paper should not necessarily be taken as reflecting the views of the World Bank. The authors acknowledge helpful comments from Pat Rosenfield.

We argue that (a) and (b) are overemphasized by the appropriate technology industry, to the virtual exclusion of (c). We also argue that development of new appropriate technologies or the adaptation of existing ones in new areas is likely to be futile if there is an inadequate system of incentives—of rewards and penalties--which continues to make it financially worthwhile for decision makers in the village water supply area to employ inappropriate technologies.

Framework for Analysis

The basic concepts involved in choosing what technology is, in fact, appropriate involve nothing more than cost-benefit analysis using shadow prices. The main thrust of the "appropriate technology" issue must then be related to market distortions.[2/] This thrust is essentially one of trying to ensure that costs and price signals which reflect the real values of physical and human resources are communicated to government decision makers so that there is a coincidence of private and social interests. If correct signals are given, appropriate least cost technology solutions will be derived naturally through the behavior of consumers, entrepreneurs and sector and project managers rather than by the activities of a few select outsiders who represent a self-appointed "appropriate technology" industry. There is no need for a somewhat paternalistic approach in which those with responsibility for water supply are advised of what is in their own best interests, when a properly shadow priced national or local market place which reflects the real costs facing society does it for them.

2/ For a number of years agencies such as the World Bank have advocated economic analysis in which the shadow pricing of both benefits and costs help outline a least cost (appropriate) solution to projects. See L. Squire and H. van der Tak, 1975, Economic Analysis of Projects, The Johns Hopkins University Press, Baltimore and London.

In the frequent instances where explicit consideration of social, cultural, religious or other factors are necessary, it is conceptually straightforward to fit these into the analysis. For example, when considering cultural criteria, the decision maker should take into account how needs, such as drinking water, have traditionally been supplied, how religious taboos might influence the use of technology, and how educational levels and social needs might assist or detract from project implementation and operation. With such a general understanding, explicit or implicit attempts can then be made to shadow value appropriately both the costs and benefits of the different approaches (levels of technology) to the problem. The basic point, however, is that the appropriate technology issue involves nothing that is conceptually new; it simply involves increased emphasis on attempting to convince planners and policy makers in the short run to compensate for, and in the long run to eliminate, market distortions so that proper investment decisions will be made at all levels of the economy.

Incentives at the Project or Program Level

The first step in determining what is the appropriate level of technology for any community water supply project or program is to estimate the real resource costs of supply. The real costs of using any type or level of technology need to be measured so that (a) real costs can be compared with expected benefits to determine whether the investment should be made; (b) real costs of alternative project designs can be compared to minimize project costs; and (c) project beneficiaries can (when possible) be asked to pay the real resource costs of their consumption and thus provide guidance about optimum investment levels.

Real resource costs can be measured relatively easily in a more developed economy where market prices or close proxies for market prices exist.

In developing countries, however, some levels of technology entail the use of goods for which there are no markets (e.g., use of bamboo poles for water pipes). In addition, although foreign exchange costs might not be a serious measurement problem for much of the proposed simple or low-level technology, it is important to compare investment costs in providing, for example, bamboo poles as opposed to cement which might require cement factory construction which in time provides additional employment. This kind of comparison requires estimating real resource costs for several different technologies. Other examples abound. It is well known, for example, that the lack of use of shadow pricing in rural water supply programs, where there is unemployment, overvalued local currencies, and development capital available at low rates, can easily lead to distortions in choosing between labor- and capital-intensive methods of construction and operations, and therefore to distortions in choosing among alternative projects.[3/]

The Importance of Benefits as Well as Costs

A further problem with most appropriate technology literature is that it is generally concerned with the cost side of investment; but the benefit side is equally important. Different technical solutions where there are variations in the methods of construction, or in design standards, can imply variations in the standard of service. In particular, a lower-cost method frequently provides an "inferior" service, i.e.,

- house connections versus public standposts;
- waterborne sewage versus pit latrines; and
- protected water supply versus open dug well.

3/ Robert J. Saunders and Jeremy J. Warford, 1976, Village Water Supply: Economics and Policy in the Developing World, The Johns Hopkins University Press, Baltimore and London, pp. 131-135.

These differences in service standards introduce a practical problem that is all too familiar to practitioners of cost-benefit analysis: while estimates of costs can usually be made with relative confidence, estimates of benefits are much more difficult to make. However, an attempt must be made to estimate, using shadow prices, both costs and benefits of alternatives, including either explicitly weighting income distribution effects, or at least specifying them.

In addition to concern about estimating the scarcity prices for different levels of technology and about measuring the first round benefits of different levels of service, the dynamic implications of project alternatives must also be evaluated. Development project objectives usually include the improvement of well being (including real income levels) for project beneficiaries. The use of "low level" technology might in the longer term, however, impede reaching this objective by limiting the rate of development of technological skills. Also, management skills in the local population might, over time, be inversely correlated with the simplicity of the local systems which they must help operate. A more detailed economic evaluation of the opportunity costs of investment for different levels of technology would enable explicit consideration of such possibilities.

Of course, in a practical sense the problems of benefit measurement are in many instances too great to overcome; macroeconomic, financial, and political obstacles may be too great. This means that a second-best alternative has to be found, since it is apparent that simply developing a different technology is not likely to be sufficient.

Village Level Resources

On the village level the second-best alternative must, in some cases, deal with poorer, smaller, scattered communities, in which there is little chance in the foreseeable future of any capital being available for local or foreign services. While some demonstration projects may be useful, it is likely that the main constraint to the improvement of water supply is simply that people are unaware of the benefits. This might be remedied by an intensive education program. Public health workers could be hired to spread the word about the benefits of improved water supply and sanitation, and could be rewarded by a bonus system based on the extent to which communities take certain measures to improve their water supplies. We would argue that if the benefits are perceived, local communities will typically be able to allocate resources to build schemes in the most efficient manner; the low opportunity cost of unemployed labor, for example, will be quite clear at the local level and since the communities tend to be self-contained economic entities, distortions in the capital market tend to be less relevant. Self-help may be the answer--there is no other way, by definition, in this case.[4/]

Central Government Resources

There are, of course, cases where resources will be available to enable the central government to play a more active role: where there is the possibility of capital infusion from the center. In such cases, at the national level the objective in improving resource allocation in community water supply programs should be two-fold: (a) to acquaint national decision makers who make policies at the macroeconomic level with the benefits of attempting to eliminate market distortions and (b) to encourage national decision makers to make the sometimes politically difficult decisions which are involved in allowing market prices to

---

4/ For an example of the importance of self-help see: Julian Bharier, "Improving Rural Water Supply in Malawi," Finance and Development, September 1978, pp. 34-36.

reflect real resource costs to society (in some cases tariffs might be higher, etc.). If this were done, the selection of appropriate technology would be greatly simplified because costs of community water supply systems, including equipment components and labor, would reflect the opportunity costs of investment in these systems. The critical decisions about the specific technology used could be made by "locals" who presumably are in the best position to judge what is best for themselves.

National fiscal policy and fiscal programs should aim to ensure that prices reflect real costs to society and that benefits are distributed in an equitable fashion. Examples of specific national level programs which might be considered are (a) allowing the foreign exchange rate to reflect free market levels, (b) making sure development funds are on-loaned to project agencies at non-subsidized interest rates which reflect the opportunity cost of capital[5/] (lower subsidized rates encourage the substitution of capital for labor), and (c) modifying national minimum wage laws which in many instances can cause a significant substitution of capital for labor. With regard to the latter, the course of action might be the elimination of minimum wage laws, accompanied by a national wage subsidy program whereby workers receive payments directly from government to make up for the difference in wages which they currently receive for work, and the minimum which they would have received under the old wage laws.[6/]

5/ The World Bank has long advocated such market-rate lending policies. See for example, Robert McNamara, 1973, Address to the Board of Governors, Nairobi, Kenya, p. 20.

6/ A somewhat similar proposal has been put forth by the US Agency for International Development in their Congressional proposal, Proposal for a Program in Appropriate Technology, July 27, 1976, prepared by Agency for International Development for House Committee on International Relations, US Government Printing Office, Washington, DC, pp. 29-30.

The point is that national policy makers must take final responsibility for market distortions. And if the appropriate level of technology is to be used in all sectors of the economy, and over time, it is at a national macroeconomic level where the first steps must be taken to assure that the price signals given to producers throughout the economy reflect the real resource costs to society.

Conclusion

The conclusion is therefore that in many less developed countries the incentive system is distorted by central government macroeconomic and other global policies, although it tends to be less of a problem in poorer, widely dispersed, and largely subsistent communities. In such poorer communities, the stimulation of demand for improved water supply is therefore first priority; and local ingenuity then has to be relied upon to produce it. There is no reason to suppose that inappropriate technology would result. The educational role is of paramount importance, mainly with regard to benefits, but also in some cases on introducing appropriate technology.

An educational role is also important where government agencies are involved. Agencies need to understand the significance of shadow pricing. The discipline of this exercise in determining least cost solutions and improving project design would then be directly passed on to construction industry, which would be confronted with a more correct system of incentives. While reform of pricing and other economic incentive systems is tremendously difficult, such a step will normally be essential if appropriate technologies are in fact to be adopted by those responsible for investment decision-making in the rural water supply area.

TRADITION AND INNOVATION IN WATER USE AND RECLAMATION

by

Saul Arlosoroff 1/

SHORT HISTORICAL BACKGROUND

Ancient Irrigation Practices

Of all the arts of food production, none is older or more important than irrigation. Historical and archaeological findings show that irrigation played a major role in the development of ancient civilizations. Large, arid and semi-arid countries owed their existence to knowledge of irrigation practices.

The ancient and traditional water raisers are based on the lifting of buckets or on Archimedes' screw. There are several variations of the bucket-lifting devices including (1) a shaduf consisting of a bucket attached to one end of a balanced cross-bar that is tilted, immersing the bucket into water, and then lifted and rotated for emptying into a higher, water-conveying canal; (2) a rotating wheel with buckets attached, where the lifting force is provided by animal or motor; (3) waterwheels too, in which a stream moves a primary (large) wheel that, in turning, collects water which is lifted through a series of buckets attached to a smaller wheel; and (4) more sophisticated devices based on Archimedes' screw where water is 'trapped' on, and made to move up an inclined plane rotated by hand or animal, is released at the upper level. Many variations of these devices are still used today all over the world.

The earliest method of irrigation consists of diverting a water stream at the head of a field into furrows or borders and allowing it to flow downgrade. In all gravity systems, water infiltrates into soil in

1/ Director, MAOT, Ltd., Consulting Engineers, 64 Hey Beyiar, P. O. Box 21577, Tel Aviv, Israel.

traversing furrows, borders or basins. By subsequent ponding and lateral water movement, soils are filled to their water holding capacity to a depth depending on the amount of water applied, duration and rate of stream flow, gradient, and soil structure and texture. The overall conveyance and application efficiency of the irrigation water is the ratio between the amount of water that actually wets the crops' root zone and the amount of water released from the source.

Surface-irrigation practices have progressed little throughout thousands of years. Many current irrigation practices are almost identical to those used in ancient times. Improvements have been in the use of concrete instead of masonry, more sophisticated gates or measuring devices, better canal linings. Although recent years have seen the introduction of control and automation, the basic failing of surface irrigation is the low product value per unit of water.

The Effect of the Industrial Revolution on Irrigation Practices

The Industrial Revolution, resulting from the development of machinery operated by man-made power, had marked effect on approaches to irrigation and on methods of water application, but traditional forms of irrigation are still common over most of the world. Invention of the steam and internal combustion engines, iron and steel castings, and machine tooling all effected advances in irrigation practices.

Engines, the prime movers of equipment in industry, are used mainly to operate pumps in irrigation. They supply power for lifting and delivering irrigation water. Their reliability and ease of operation have revolutionized irrigation, for with advances in metallurgy and pipe and fitting fabrication, it is now possible to convey water under pressure and against gravity to elevations hitherto not irrigable by gravity.

The 20th century development of simple electric motors, centrifugal pumps and pressure pipes have closed the cycle of needs so that there are possibilities of developing new forms of water application.

SURFACE IRRIGATION AND ITS APPLICATION

For open-ditch conveyance and gravity irrigation, often less than half of the water released reaches the crop. Conveyance and distribution losses must be considered in planning and operating projects. Further, erosion, salination and water-logging occur, degrading land productivity and water quality. Gravity-irrigation projects operated by skilled personnel afford higher efficiencies.

When efficiencies are low, more water is needed, making necessary larger storage and channel capacities and extensive drainage systems. All these need heavy capital investment. The end result of low efficiencies is thus inflated per unit-area project costs that reduce overall project returns and impair project feasibility.

Compared with pressure irrigation, gravity or surface irrigation has the following disadvantages:
(1) More water is needed per unit area; it is almost impossible to apply small amounts of water so that product per unit of water is consequently low; (2) water may accumulate in the subsoil, causing water-logging and eventual salination; (3) land preparation is costly and time consuming; scraping and grading demand skill, and may temporarily reduce soil fertility; (4) heads of water must be adjusted constantly to give the uniform water coverage needed for high yields; and (5) costly drainage may be needed.

The advantages in using gravity irrigation are (1) lower initial capital costs are required; (2) it is assumed (but not proven) that the farm community is better able to operate forms of gravity irrigation than more sophisticated methods; and (3) there are options open for improvement by various devices.

There is need for improvement in irrigation projects. Highly efficient surface-irrigation projects can be found in few countries. Their efficiency is based on a combination of (1) suitable size and location of dams; (2) skilled, mechanized land grading and preparation; (3) concrete, asphalt or plastic lining of main and secondary canals and use of large diameter pipelines; (4) automatic control of flow in pumping plants, primary and secondary canals, and lateral furrows, and onto and in the fields; and (5) computer scheduling and timing of water application.

Potential Improvements of Surface Irrigation

There is a great potential for improving gravity irrigation, and capital thus invested would increase marginal outputs and lessen dependency on manpower.

In the last few decades, many automatic labour and water saving devices have been developed for irrigation-system water control. An automatic surface irrigation system would sense the need for irrigation; turn on the water, whether from on-farm or canal-allocation source; accept this water and correctly apportion it to field or border; and turn off the water and reset itself in preparation for the next irrigation.

The deterioration of soil and water quality caused by irrigation can best be controlled on the croplands where the water or effluents are applied.

The two points of control are (1) the call period, the maximum interval a farmer can expect to wait for his water delivery after placing an order with the canal operator, enables ditch riders to plan in advance and program it, and (2) regulation of irrigation timing and amounts based on improved evapotranspiration models for various crops and soils.

SPRINKLER IRRIGATION AND ITS APPLICATIONS

Lifting of water by contrifugal pumps and the use of electric motors, diesels, metals and plastics for pipe fabrication fostered the development of sprinkler irrigation.

In sprinkler irrigation, water is delivered under pressure through ordinary pressure mains into systems of fixed and/or portable, light, quick-coupling irrigation lines on which sprinklers are mounted at regular intervals. Sprinklers have been designed for operation under various pressures, spacings, and sizes, and to give various distribution rates and patterns. This rain-like irrigation system is thus adaptable to a wide range of agricultural conditions.

The change from surface to sprinkler irrigation provided a break-through in irrigation concepts, giving the irrigation engineer, agronomist and farmer almost complete control over delivery of water to the soil reservoir. Mechanized sprinkling systems have almost completely solved the labour-problem.

Since up- and down-stream conditions vary, the need for regulating flow so as to maintain correct pressure in the laterals at the sprinkler nozzles was soon realized, and pressure and flow devices were developed to achieve constant flow at correct pressures. Regulated sprinkling allows all sprinklers to release the same flow irrespective of topography or position. This means that as the supply pressure changes, a high system distribution coefficient and minimization of evaporation and run-off losses and soil salination are maintained.

Specific Applications

The specific applications of sprinkler irrigation exploit the very manner in which water is applied, and their potential for spread and further development is extremely high.

Application of Chemicals. A most important and growing development is the use of sprinkler systems to apply fertilizers and other chemicals during irrigation. The operational savings possible are clear, but costs and/or damages of over-and under-applications of the chemicals motivate the precise water metering and thus the chemical application.

Aeration Irrigation by Sprinklers. Frequent, low-discharge, high-pressure sprinkling contains a new irrigation concept, "Aeration Irrigation". High soil moisture is maintained in the top third of the root-zone, soil structure is greatly improved, soil compaction is prevented and of particular importance, air is entrained and carried in the soil with the water.

Frost Protection or Crop Cooling by Sprinklers. The idea of providing frost or cold protection by sprinkling is not new, but when it can be controlled electronically, and automation provides swift water distribution and application throughout the system, its costs is compensated by higher crop yields. A simple fertilizer feed tank can be turned into a water heater by adding an oil or gas burner. Demands from irrigation systems used for cooling are usually less rigid than those demanded for frost protection, both need either permanent or solid-set systems for good results.

Sprinkler Irrigation Techniques - Present and Future

The most promising techniques for sprinkler irrigation include the following:

Orchards. Using a combination of stationary mainlines with portable aluminum pipes, water is applied through impact sprinklers mounted on aluminum pipes which can be shifted manually. Application rates are controlled by discharge regulators, sector sprinklers, and automatic metering valves.

With semi-portable irrigation, light flexible polyethylene or PVC lines are permanently connected to the main line, work is lightened, leakages are prevented and surface run-off reduced. Several variations exist which enable irrigation in the daytime and in windy conditions. All the automation options can be installed.

Permanent-set sprinkling is used when frequent or irregular irrigation is required to save both labor and water. Automatic metering valves, discharge regulators, or electronic controls are provided with such systems.

Field Crops. A combination of stationary mainlines and portable aluminum pipes is widely used on smaller family farms. Connected to stationary mainlines, the portable irrigation lines are aluminum pipes used to irrigate successive parts of a field by dismantling and reassembling. The aluminum pipe is light but durable and has good water-carrying capacity; the system is simple to use and relatively cheap, but moving the pipes is one of the heaviest of farm tasks. If lines must be shifted frequently, workers are needed, mostly at awkward hours.

The tractor drag-line (two-line method) is used for large plots of field crops, and for supplementary irrigation of wheat. Tractor-dragged lines reduce labour input significantly. The method has enabled many farms to extend their irrigated area. The equipment can be dismounted fairly rapidly to permit spraying and other operations. The equipment may be used 2-4 times a year on different crops or locations, thus optimizing the capital input.

Equipment is relatively simple and cheap. A high degree of irrigation efficiency can be attained. A recent development enables the farmer to shift secondary lines as well as the laterals.

Stationary (solid-set) systems, using aluminum pipes are placed before germination and, depending on the crop, removed before or after harvest. This method is widely used for intensive cropping. With pregermination irrigation, schedules can be closely followed and water contents necessary for proper plant growth maintained, particularly in warmer or drier periods. Wear and tear on lines is elimated. The major drawback is, of course, the high initial investment in equipment. On some farms, the equipment is used three or four times per year: solid set for spring and fall vegetables, and dragged in summer and winter for cotton and wheat.

DRIP (TRICKLE) IRRIGATION AND ITS APPLICATIONS

Drip irrigation, perhaps is the most important development in the world of irrigation since the impact sprinkler in the beginning of the century. It is out of its infancy and begun to spread in the world. Even in countries where it has spread relatively quickly, the state-of-the-art is not fully established. Even though most experience has been with the irrigation of vegetables and vines in either arid or temperate zones, the results are applicable to other areas and crops.

Crop response and moisture tension are strongly related. In many cases yields increase markedly as irrigation intervals shorten, and in highly arid or semi-arid conditions and on coarse soils, one or more daily irrigations give highest yields. Frequent irrigation keeps soil moisture tensions low, so that the crop is able to withstand the high evaporative stress of arid conditions and the high osmotic tensions of saline waters. No adverse effects of poor aeration have been observed.

Under dripping, net yields values are considerably enhanced: earlier and high yields and thus higher produce value, with lower production costs since early irrigation gnerally reduces the amount of water, pesticide and other treatments.

The commercial use of this irrigation method has aroused much interest throughout the world. Developed to a commercial level in the 60's in Israel, it spread very rapidly. Worldwide, about 100,000 hectares (250,000) acres are presently drip irrigated.

Predictions of 1 million ha (2.5 million ac) under drip irrigation by the next decade in the world could be met if there is a breaktrough with expendable laterals into field crops.

Advantages

Advantages of drip irrigation include (1) marked increased in crop yields, quality and timing, product value per unit water; (2) crop growth in saline and/or warm conditions that could not be obtained with other irrigation methods; (3) low energy requirements.

Drip irrigation is not a high-frequency form of furrow irrigation. It differs principally in the following features:

(1) It requires none of the grading skills normally associated with furrow irrigation.

(2) There is no surface flow of water along furrows and hence neither soil erosion nor tail-water loss.

(3) Since the emitters discharge water by dripping along the length of plant rows, each releasing approximately the same amount of water and fertilizers because pressure losses along the laterals are minimal, water distribution is fully controlled and highly uniform.

(4) There are essentially no evaporation losses.

(5) The system is easily adaptable to any type of control or simple automation.

Problems and Requirements

Conditions which must be fulfilled if drip irrigation systems are to realize their full potential include the following:

(1) They must be designed very carefully to suit local conditions by experts familiar with the systems and what they can offer.

(2) It is essential to clean the water. Most failures are due to inadequate filtering. Many types of filters have been developed whether for surface water or sewage effluents.

(3) Water application should be based on careful observations, and should be frequent and light.

(4) In some conditions, leaching may be needed. Salts tend to concentrate at the perimeter of the soil volume wetted by each emitter, and light rain may leach them into the root zone.

The use of dripping for field row-crops may completely revolutionize irrigation. The important development is so recent, that another 3-5 years will be needed before conclusions can be drawn. Until now, drip systems have been limited to vegetables, fruits, and vines; their feasibility, if proved for cotton, maize, sorghum, sugarbeet, potatoes, sugarcane, etc., would open vast areas to regulated water application.

The two main lines already available for field row-crops include; cheap, either laser-perforated or 'sweating', low-pressure, plastic lines used for after one season and costlier, laser-perforated medium-pressure, plastic lines, for 3-5 seasons. The first published result, indicate that for cotton and maize, significant increases in crop yield per unit of water were obtained.

AUTOMATION OF IRRIGATION

Labor shortages, increasing costs, rising food prices and decreasing water quality, inevitably result in the development of automated irrigation systems.

Small-scale Partial Automation

The more important examples include semi-automatic or automatic cut-off pump starters. The former, generally installed in small pumping stations, stop and start pumps automatically. Re-starting is done by manual re-set or automatically. The automatic metering valve is a very simple instrument to deliver any prescribed volume of water, preventing excess discharge due to pressure fluctuations or forgetfulness. Both devices increase the efficiency of water use; they do not eliminate manual labour, but do save it.

Large-scale Automation

Large-scale, more sophisticated forms of automation are to be found throughout irrigation projects, where computers and electronic controls are used in operating dams, large pumping stations, canal flows, etc. More important is the recent move of electronics into the farm plots.

Larger automated systems have three sets of components: (1) the sensing devices; (2) the relay equipment; and (3) the data-processing and control equipment.

In most present irrigation projects decisions are arbitrary and unscientific. Heavy expenses of investment and maintenance of automated systems can only be justified if they raise irrigation efficiency.

Computers in Irrigation

Examples of current use of computers in irrigation as planning aids and controllers include computer derived graphs and tables to aid the networks designer; on-line computers are available for network simulation to aid irrigation disigners and universities and firms offer computer programs to optimize the operation of established and new irrigation systems.

A Computer-controlled Irrigation System. Since the early seventies, the first completely automated, computer-controlled pressure irrigation systems have been in operation. Water applications are based on considerations of water availability, climatic conditions, soil properties and marketing factors.

The automatic irrigation control system developed recently in Israel consists of three major elements:

(1) A master control station, containing the computer and its peripheral display and control equipment:

(2) The field units, which execute instructions relayed from the master control and which, in turn relay status reports and field measurements to it, and

(3) A single, buried 3-wire cable, or radio-link, which transfers instructions to the field units, and relays the status and field reports to the master control.

In comparison with a conventional sprinkled system, the system achieves water saving (10-20%), manpower saving (20-30%), centralized control, optimization of network capacity (reduction of 10-15% in price), improved crop yield (5-20%), quick adaptation to changing climatic and system conditions, automatic record-keeping, and a reduction of investments.

The system's program permits the operator to specify a variety of operating conditions and priorities. Thus, the schedule for opening and/or closing valves can accord with either water quantity, time or both. The operator may assign priorities according to water pressure, time wind, or temperature.

<u>Mechanization of Irrigation Systems</u>.

The methods mainly relate to sprinkling equipment. All aim at various optimizations between cost of capital, cost of labor and irrigation efficiency where the common denominator is the trend to mechanized transport of a sprinkling system. Recent developments aim at dragging or towing dripping lines. Most of the mechanized systems are high-energy consumers.

"Boom-Type" System (Overhead rotating boom, etc.) Here, the two laterals or booms, supported by a tower and cable structure are made to rotate by the jet action of the many nozzles on the boom arms about a point at which a main-line supplies water. The boom is usually transported by a trailer that may also be used to transport the pipe sections used to lengthen (or shorten) the booms.

"Self-Propelled" or Pivot Systems (Pivot, Tri-matic, Squr-matic, etc.) Here a single, sprinkler-carrying pipe is supported by a series of wheeled towers; in that the whole structure moves or pivots about a point that connects with the principal supply line. Propulsion and/or irrigation are hydraulically and/or electrically controlled.

These units can be completely automated with regard to water and chemical applications, and are able to supply water at a wide range of rates. The pivot systems can irrigate parts of the circle covered, can reverse directions, and with radii of between 120 and 400 m, can irrigate areas of 6 to and 80 ha (15 to 200 ac).

"Skid or Wheeled Giant Sprinkler" Equipment (Winchable, Automatic "Big-Guns") Here giant sprinklers or 'irrigation guns' are supported on skids or wheels for manual or mechanical transport. They are generally mounted in tandem, 60-72 m (200-240 ft.) apart and linked to the main supply pipe. They can be moved to the right or left of the supply pipe either by operating a pulley attached to the main pipe, or manually. Because it can provide high hourly water rates, this system is mainly suitable for highly permeable soils.

RECLAMATION OF SEWAGE-EFFLUENTS FOR IRRIGATION PURPOSES

Both raw and treated sewage have been used for many decades in countries throughout the world. However, the scientific aspects of reclamation have received attention only during recent years.

Israel, a water scarce country, has adopted a national policy for reuse of almost all sewage effluents, after secondary treatment, as a major water source for irrigation. In many cases, this replaces good potable water to be supplied for the expanding domestic water demand. Such a policy forces all involved to enhance research and experimental works in order to clarify all relevant aspects of effluent irrigation. The following chapter contains data from few important works in this field.

Use of Sewage Effluents for Irrigation

The advantages in the use of treated sewage for irrigation are (1) a low-cost source of water, (2) an effective use of plant nutrients contained in wastewater, and (3) provision of additional treatment before being recharged to groundwater reservoirs.

The principles of wastewate utilization discussed here are based on Israeli experience and on some investigations reported in the literature.

The responsibility for wastewater treatment lies with the discharger, whether municipality, industrial plant, or agricultural settlement. Wastewater, even after some degree of treatment can cause the pollution of waterways, rivers, and groundwater reservoirs. The quantity of wastewater in Israel ranges between 70 and 300 litres per capita per day, and is usually the cheapest water resource in arid areas. In some cases it is considered that the treatment of wastewater is required in any case, mainly because of pollution and health regulations. Significantly, effluent standards required for irrigation quality are less strict than those standards for other disposal methods. The cost of water should therefore be based only on the cost to transport it to the irrigated plots.

Another advantage is the addition of nutrient elements required by growing crops. The application of wastewater in irrigation brings about the renovation of the percolating water through the soil profile, especially in the presence of growing plants. The root zone is considered a "living filter". It has been found (3,4) that phosphate is reduced in the percolating water by reaction within the soil, and nitrogen is reduced due to uptake by plants. In this manner groundwater pollution and eutrophication of streams and lakes by wastewater are drastically decreased.

Possible disadvantages in the Use of Treated Wastewater for Irrigation. Disadvantages of wastewater utilization for agriculture include the following:

(1) Wastewater supply is continuous. However, the demand is variable. Therefore, there is a need for effluent storage to provide for operational and seasonal fluctuations. Treated wastewater is being stored for operational purposes together with water from other sources, thus increasing the total quantity of water to be applied in irrigation and improving its quality. In small treatment plants, oxidation ponds also serve as storage ponds. Another solution is to recharge groundwater, which may be reused for irrigation by pumping. Groundwater replenishment causes a significant improvement in the wastewater quality as percolation through the soil reduces undesirable suspended and soluble constituents by absorption or precipitation, including a reduction in pathogens.

(2) Most of the suspended solids in raw wastewater are moved by proper treatment. However, some solids may be found in the effluent. Effective filtration and the use of large nozzles are then required to avoid plugging in sprinkler or drip

irrigation systems. In the irrigation season the addition of organic matter may reach 1500 kg/ha (1300 lb/ac). Such a quantity may bring about changes in the physical properties of the soil. This may be favorable in sandy soil; in heavy soils however, it may cause partial clogging and a decrease in the rate of infiltration. Breaking up the surface crust and ploughing the deeper layers are sometimes necessary to improve infiltration rates.

(3) Treated domestic effluent, and certainly industrial wastes, may contain soluble consitutuents at concentrations toxic to plants. Domestic use results in increases of 50 to 100 mg/l choride and sodium ions which may concentrate in the root zone and harm sensitive crops. Sodium causes of deflocculation of clay particles which results in unfavorable soil structure. This then decreases water and air permeability. A concentration of 1 ppm boron may harm sensitive crops. Some industries may discharge heavy metals at concentrations toxic to plants or animals feeding on plant material. Other wastes may contain organic compounds such as organic acids and phenols, that may restrict biological activity in the root zone. Mixing of such wastewater with other wastes may dilute the toxic constitutents to acceptable concentrations. In some cases such wastes have to be separated to avoid their utilization in irrigation.

(4) Wastewater may contain pathogenic bacteria, parasite eggs, cysts, and viruses which are carried by human excreta. Treatment brings about a decrease in their number up to 99.9%. The remaining concentration still exceeds 10^3/100 ml. (The standard allowed for coliform bacteria in wastewater used for irrigating

certain edible crops is sometimes set at 100/100 ml or less). Irrigation with effluent in Israel is usually restricted to fiber and other industrial crops, seed crops, ornamentals, and sometimes fodder crops and vegetables that are consumed only after cooking.

The Value of Fertilizers in the Effluent

Review of recent research conducted in Israel on utilization of domestic secondary effluents as a source of fertilizers for agricultural crops reveals the following:

Nitrogen. Effluents contain relatively large quantities of nitrogen which is usually added artifically to the irrigation water and/or directly applied to the soil. In domestic secondary effluents nitrogen appears mainly in the form of ammonia, with some as organic nitrogen, and very little as nitrite or nitrate. The common values in Israel for total nitrogen in the effluents is between 30 to 60 mg/l, 60 to 70% of which is ammonia nitrogen.

Recent studies and surveys made in Israel attest that the nitrogen in the effluents is available to the plants. Thus, for example, a team from the Soil and Water Research Center found that irrigation of cotton with secondary effluents with a 50-70 mg/l level of total nitrogen, with no addition of nitrogen fertilizer, resulted in a gross yield level higher than that in plots irrigated with water from the national water carrier with an addition of a pure nitrogen fertilizer in rates of 60, 120, and 180 kg per ha (50, 100, and 150 lb/ac). Plants irrigated by effluents showed faster growth than those in plots irrigated with water from the national carrier with regular nitrogen fertilizer.

The team studied this aspect in a further research in Zoraa, and it was found that with an irrigation regime of 3 effluent applications per season yielded 4500 kg/ha (4000 lb/ac) of cotton: and 4 irrigations yielded 5000 kg/ha (4400 lb/ac) as compared to 4400 kg/ha (3900 lb/ac) cotton irrigated with water from the national carrier, with full nitrogen fertilization. The yield level was obtained with effluents and no additional fertilization. The various irrigation treatments were applied with the same water quantity, 350 mm (14 in). The team is now studying a regime of 5 as against 4 irrigation per season.

The same team studied the effects of effluent irrigation on Rhodes grass. It was found that Rhodes grass yields were high in all the plots where effluents supplied all the nitrogen. In plots irrigated with well-water, high yields were obtained only when a regular level of nitrogen fertilization was applied. Here too, water management is very important. Increasing frequency of effluent irrigation from once to twice a week increased the level of dry matter and increased the utilization efficiency of nitrogen provided by the effluents. A significant finding was that no high concentration of nitrates were found in the soil profile below 2 m (6 ft.). This was a result of the high absorption of nitrogen by this crop, and of the denitrification encouraged by the root respiration and the organic matter in the effluent.

Current experience in Israel showed that Rhodes grass can be grown on effluents alone, with no fertilizer addition, and the regular yields and above obtained. It is also valuable, as aforementioned, as a good nitrogen absorber, preventing infiltration of nitrates into groundwater. According to the works of the Israel Field and Extension Service, the values of nitrogen in citrus leaves on effluent irrigated plots are higher than in plots irrigated by regular water plus regular fertilization. This fact points to the possibility of greater savings in fertilizers in citrus groves.

Phosphorus. Phosphorus concentrations of approximately 20 mg/l are found in domestic secondary effluents in Israel. It is considered available to plants and is therefore as a fertilizer according to the works and results by the field and extension service.

The effluent research team of Soil and Water Research Center found that phosphorus values in cotton reached an average of 22 kg/ha (20 lb/ac) irrigated by fresh water, and 31 kg/ha (27 lb/ac) in plots where effluents were provided. The effect of effluent irrigation on phosphorus absorption is therefore obvious. The same team also found that the phosphorus valued in Rhodes grass were 67-80 kg/ha (59-71 lb/ac). The highest phosphorus yields were in plots irrigated twice a week.

According to findings in Israel 1000 mm/ha (16 in/ac) of effluent containing a concentration of 11 mg/l total phosphorus are equivalent to 11 kg (24 lb) enriched superphosphate. As domestic wastewater contains 7-20 mg/l total phosphorus, the quantity of phosphorus applied through effluents in regular quantities of irrigation is sufficient for the various pasture crops but not field crops such as cereals or sugarbeets. The conclusion is that significant quantities of phospheric fertilizer can be saved.

Potassium. Potassium is a vital element to plants and is given to them as fertilizer. Its concentration is secondary effluents in Israel is 15-30 mg/l. The effluent research team of the Soil and Water Research Center found that effluents with a concentration of 14-19 mg/l potassium added to cotton fields in 335 cm per ha, was 60 kg/ha (53 lb/ac) potassium. This quantity is small compared to the plants' consumption. The proportion of potassium in effluent irrigated plants was not different from that of plants irrigated with water from the national carrier, and ranged between 80-190 kg potassium/ha, (70-170 lb/ac) and for one fruit was 140-270 kg potassium/ha (120-240 lb/ac). The research team found that in Rhodes grass the quantity of potassium contributed by the effluent in one growth season was 160 kg/ha (140 lb/ac) with a potassium concentration of 15-22 mb/l - just a little over the potassium absorbed in plants increased parallel to the increase of nitrogen application. According to Hershkovitz and others, urban wastewaters contain 20-60 mg/l K_2O and 400-500 m^3/ha of effluent will add 4 kg potassium sulphate per ha.

Research and Field Experiments with Irrigation of Corn (Maize) with Sewage

In this experiment (4) corn was chosen as an indicator plant because (1) it is used for fodder and as a supplement to Rhodes grass (forage), and it is used as an alternative crop for utilization of wastewater on farms; (2) corn is grown in many places around the world, so that information on wastewater irrigation of corn can be widely applied; and (3) being a row crop, corn is suitable for drip irrigation, and to a certain degree represents other row crops.

The Source of Water. The source of water used for irrigation is sewage from a school, including laundry, swimming pool, and rinsing water from the cowshed. Cow manure and urine from the cowshed do not reach the sewage system. The sewage reaches two interconnected open earth ponds, with no control over the distribution of water between them. In these ponds, the sewage undergoes primary treatment by the settlement of solids. These ponds serve as oxidation ponds, but the detention time of the wastewater in ponds is insufficient and there is no control over the oxidation process so that the experiments deal with extremely overloaded lagoon effluents.

Results. The yield response was similar to that of other experiments, and under conditions of a constant pan coefficient during the entire period of growth. It seems that water application of 50% of class "A" pan evaporation is sufficient in order to obtain maximum yield under the experiment's conditions. In this experiment water was not a limiting factor. It should not be concluded, however, that the maximum yield obtainable was obtained. It is quite possible that another restricting factor operated here, and had it been removed, the water would have resulted in additional yield.

It was shown that in this experiment that a good yield was obtained in a shorter than the usual growth season, with 60-70 percent of the regular water quantity. Even assuming that the soil was wet at the beginning of the experiment, and taking into account only the irrigation following the saturation irrigation, the water consumption in this experiment was only 80% of the regular quantity. It is also clear that the wastewater provided sufficient phosphorus and potassium to supplement the reduced quantity of application. The basic fertilizer in the wastewater provided the equivalent of 680 kg ammonia sulphate per ha only, (600 lb/ac) instead of the conventional 1000-1200 kg per ha (880-1100 lb/ac). It was found that controlled operation of sewage treatment including sedimentation, oxidation pond (or other secondary treatment) and effluent filtering is essential. The operation should include skimming algal mats from the oxidation pond, and two-stage filtering with screens and gravel filters. The 80-mesh screen filters must be equipped with automatic cleaning and flushing arrangements, and each main filter should be connected to an additional filtering device for cases of failures. Frequent backwashing of the gravel filter is required, as is flushing of the emitters in the irrigation system. Note that the number of cloggings in the last emitters in a 100 m line was greater than in a 50 m line of emitters.

The conclusion from the experiment is that irrigating corn with low quality effluents proved successful, reaching above average levels of yield with 70-80% of the nominal water quantities and 60% of the nominal fertilizer quantity.

Public Health Aspects. In 1972-78 experimental irrigation of vegetables with the effluents from Eilat, Israel was conducted. Here, the control plots were irrigated with National Carrier water by ridge-and-furrow methods and experimental plots used drip irrigation of effluents from overloaded oxidation ponds. The faecal coliform contamination of vegetables drip-irrigated with effluents in plots that were not covered with plastic mulch was 38 times higher than on vegetables in the control plots (6). On the other hand the contamination level in vegetables drip-irrigated with effluents was only 10 times higher in a mulch covered plot, and only 5 times higher in a covered plot with underground irrigation as compared to the control plots.

Results of field experiments in which large quantities of virus were applied support these findings, i.e. that adjustments in the drip irrigation method, such as irrigation under plastic mulch or underground irrigation, significantly reduce the contamination of effluents irrigated crops, even under extreme conditions of massive artificial contamination of the effluents by microorganisms.

All the agrotechnique operations that were tried, and particularly covering the soil with mulch, are common and accepted practices in Israeli agriculture, and the practical advantages of using these methods for effluent irrigation is therefore obvious. This is true for both the experiments in Eilat and at the "Green Farm" in Ramat, Hashason.

Climatic conditions and soil properties should be taken into account as important factors affecting the residue of microorganisms in the soil and in vegetables, even on a high level of contamination. It should however be borne in mind that even under difficult conditions pathogenic microorganism can survive a long time in the soil.

The use of large doses of enteric organisms to study public health aspects resulting from effluent irrigation should also be noted. Such an approach enables a sensitive examination for comparing various effluent irrigation methods under extreme contaminating conditions. This method can also serve to examine the existing microbiological standards for effluents intended for irrigation.

Safe use of effluents for agriculture is a welcome goal in arid areas, and many countries aim to reach it. This research indicates that the drip irrigation methods with plastic mulch cover of the soil may significantly reduce the microbal contamination of the vegetables up to satisfactory limits.

Limited data revealed that under certain conditions when the root or the stalk are contaminated, animal virus penetration into the stalk or the leaves through the conduction system of the plant is possible. The significance of these limited experiments is not yet clear.

Taking into consideration the findings concerning the advantages of agrotechnique methods such as drip irrigation and soil covering, the researchers recommended that the drip method be used as a alternative for effluent irrigation, provided that effluents used for irrigation will meet the standards and other permit conditions of the Ministry of Health. Another advantage of using effluents by the drip method is the prevention of airborne aerosols formation. Recent research indicates a small likelihood of transporting pathogenic microorganism through the air as a result of sprinkling effluents (7).

Despite the above said the drip irrigation method with effluents, it is important to remember in estimating the health hazard that a very small number of viruses is enough to cause infection to man.

In order to estimate the health hazard it is important to take into account such contamination, even if it is small, with due regard for the prevailing conditions in the country or region. It should be stressed that all the research and conclusions discussed in this paper reflect both health conditions and human sensitivity in Israel.

REFERENCES

1. Israel, a Model of a Country's Efficient Water Development and Utilization - S. Arlosoroff (March 1977, U. N. World Water Conference).

2. Irrigation Forecasts and Trends - S. Arlosoroff (F.A.O. - European Region Rome, May 1976).

3. The Use of Waste-water for Agricultural Irrigation - J. Noy, A. Feinmesser (Water renovation and Re-use, 1977, Academic Press).

4. Drip Irrigation of Maize with Secondary Effluent Research Results - E. Rawitz, Z. Katain, Z. Friedman (Hebrew University, Water Commission, March 1978).

5. Secondary Effluents as a Fertilizer Source - A summary of research and field experiments by A. Burshtein (Water Commission, Israel, January 1978).

6. Microbial pollution of Vegetables Drip Irrigated with Sewage Effluents - D. Goldberg, B. Fattal, E. Katzenelson, H. Shuval, A. Sadovsky, M. Ozrad (Hebrew University, Eilat County Council, June 1978 - preliminary publication).

7. Air-Dispersion of Viruses and Bacteria as a Result of Secondary Effluents Sprinkling - B. Teltch, E. Katzenelson, H. Shuval, (Hebrew University, November 1976.

SIMPLIFIED WATER TREATMENT PLANT DESIGN

By R. L. White[1] and N. L. Presecan[2]

INTRODUCTION

The building of succeeding generations of sophisticated water and waste water plants, usually containing complex and energy intensive equipment, has been one of the marks of technically advancing countries. And yet, one of the most common problems is that adequately trained personnel are unavailable to operate these plants and another is that the equipment is in many cases costly to install and to maintain. In addition, even with competent operators, too often plants; another is that the equipment is in many to operate because of the wide variety of options provided to the operator.

The purpose of this paper is to describe one approach to plant simplification, in this case a water treatment, without penalizing cost, throughput or effluent quality. In fact in the ensuing discussion it will be demonstrated that increased return on investment results when simplification occurs.

In most areas of the world, water may be made potable by removing turbidity and phytoplankton and disinfecting against pathogenic organisms. In terms of treatment, this simply translates to the successive unit processes of chemical coagulation, sedimentation, filtration, and chlorination. Although some water supplies must be treated to remove dissolved minerals for health or aesthetic reasons, by far the majority of the world's water supplies may be rendered potable by the treatment processes mentioned above.

---

[1]President, Engineering-Science, Inc., Arcadia, California

[2]Senior Vice President and Chief Engineer, Engineering-Science, Inc., Arcadia, California

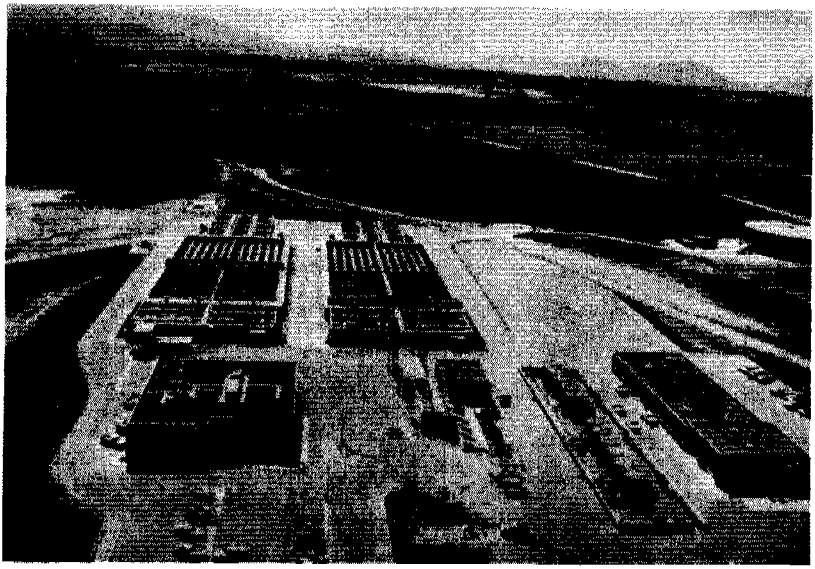
Recent parallel and independent efforts by the Metropolitan Water District of Southern California (MWD) and by Engineering-Science, Inc. (ES) have been directed toward the design of water treatment plants which use less energy, have lower construction costs, require less space, and require less experienced manpower because of the elimination of equipment and elaborate control systems. These plants are modern and utilize the most recent technical developments in the treatment processes and yet by virtue of these improvements, are both simple and effective.

This paper describes in detail ES' recent design of the Oceanside, California, water treatment plant which employs the principles enumerated above. The concepts basically parallel MWD's pioneering Robert A. Skinner filtration plant, which has been in operation in Riverside County, California, since 1976. The Skinner plant treats Colorado River water with alum and is characterized by high process flow rates of 85 $m^3/m^2 \cdot d$ for the dual media filters. The filtration system is unique because of the elimination of pipe galleries and extensive piping, valving and control consoles. The filtration system is of a simple design and was developed by Mr. Lee Streicher when he served as Water Purification Engineer with MWD.

Because of success at the Skinner plant with high process flow rates and the simplified filter design concept, MWD subsequently constructed a second plant, the Henery J. Mills Filtration Plant, with 285,000 m^3/d capacity which was recently completed and will soon be placed into operation.

GENERAL DESCRIPTION

The water treatment plant designed by ES for the City of Oceanside, California, is a 65,000 m^3/d complete treatment facility consisting of chemical (flash) mixing, flocculation, sedimentation, filtration, and disinfection. Backwash water from the filters is recovered and returned to the



ROBERT A. SKINNER 570,000 m^3/d FILTRATION PLANT
Courtesy of Metropolitan
Water District of Southern California

plant headworks for retreatment. Sludge from the sedimentation process and the backwash recovery system is dewatered on sludge drying beds. The supernatant and underdrain flow from the drying beds are also returned to the plant for retreatment.

The chemical systems include both essential chemicals for turbidity removal and disinfection and desirable chemicals for aesthetic purposes. The essential chemicals include chlorine for disinfection, alum and polymers for coagulation and sedimentation, and sodium hydroxide for pH adjustment. The chemical facilities also include activated carbon and potassium permanganate for aesthetic purposes of controlling color, taste, and odor. The selection of these facilities for Oceanside was based upon considerable experience with the water, and the availability of the chemicals. It must be recognized that the chemical types and availability vary throughout the world and that design criteria must be adjusted to reflect this change.

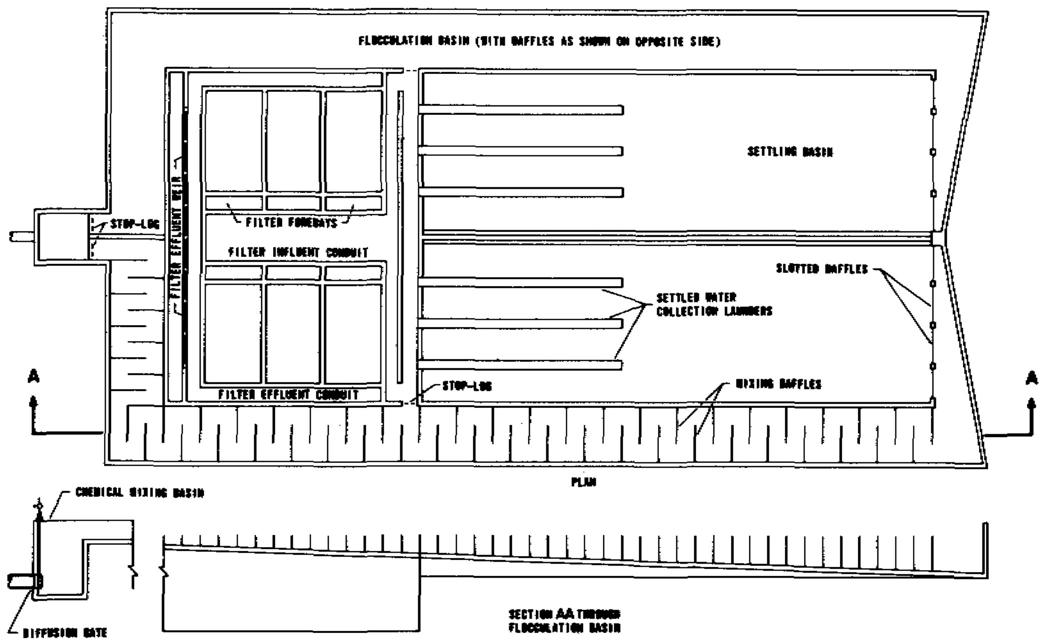
The design criteria for the plant processes are summarized in Table 1 and the processes described in greater detail in the following sections. A general plant layout is shown in Figure 1 and the plant process diagram is presented in Figure 2.

CHEMICAL MIXING BASINS

Coagulation occurs in the chemical mixing basin and is usually achieved by power input into the water by means of mechanical agitators. The rate of power input is measured by the mean velocity gradient, G, sec^{-1}. For chemical mixing, mean velocity gradients of 500 to 1,000 sec^{-1} are common.

The primary source of water for the Oceanside Water Plant is the San Diego County water Authority's No. 2 Aqueduct System which can provide an estimated residual hydraulic head at the plant headworks of approximately 12 meters of water. This residual pressure or energy will be used for

OCEANSIDE WATER TREATMENT PLANT CONCEPTUAL DESIGN



PROCESS FLOW DIAGRAM

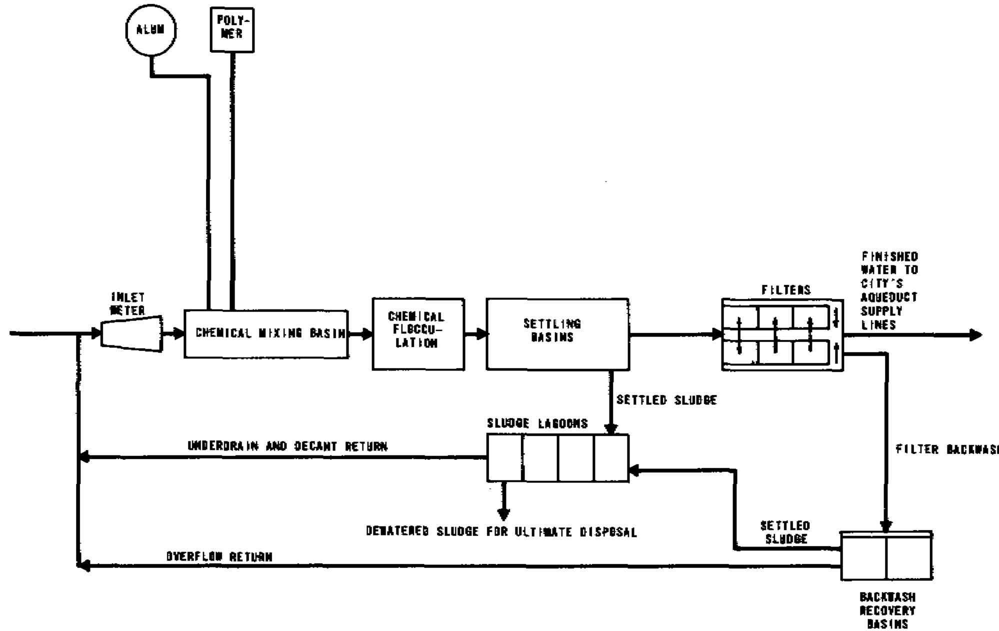
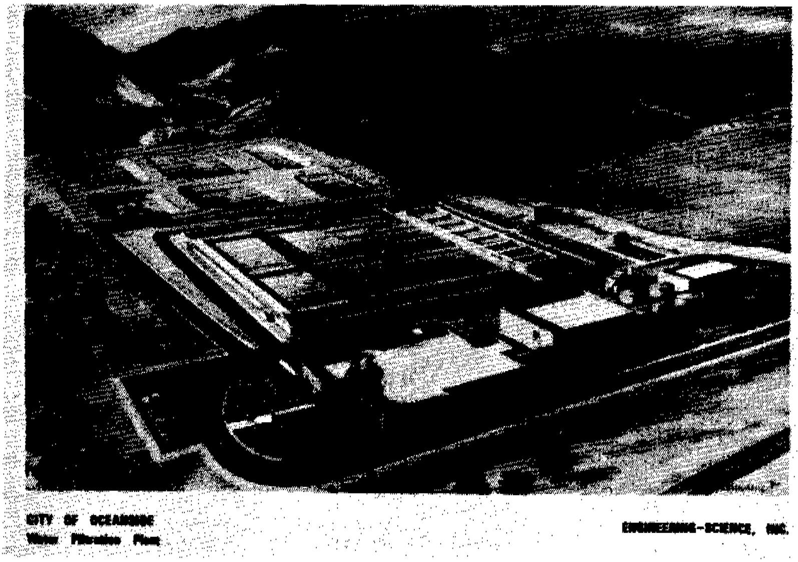


FIGURE 2



CITY OF OCEANSIDE
WATER FILTRATION PLANT

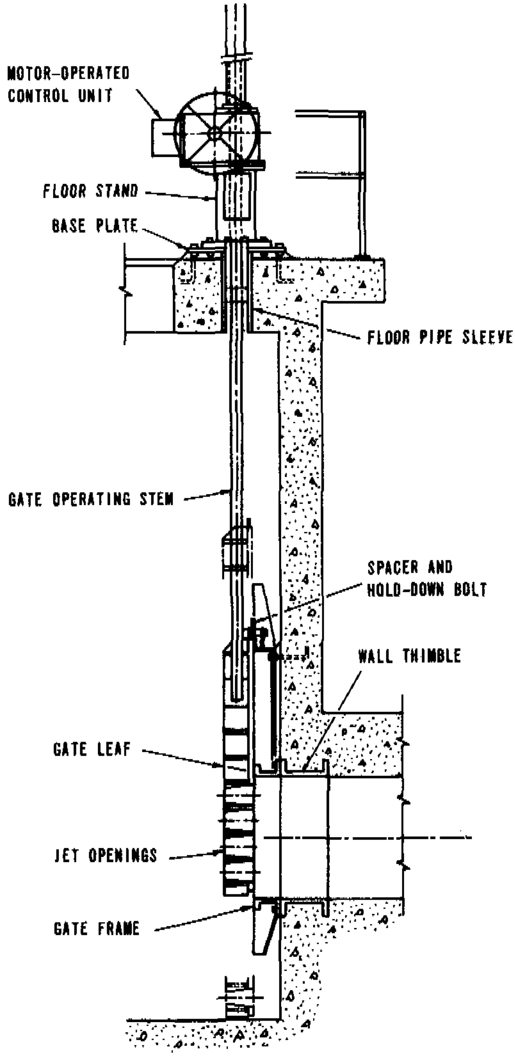
TABLE 1.—Oceanside Design Criteria

Item (1)	Criteria (2)	
Capacity	65,000 CMD (maximum)	(17 mgd)
Chemical Mixing Basin		
Number	One	
Dentention Time	15 seconds	
Flocculation Basins		
Number	2	
Detention Time	25 minutes	
Settling Basins		
Number	2	
Detention Time	60 minutes	
Hydraulic Loading	56 $m^3/m^2 \cdot d$	(1,370 gpd/sf)
Filters		
Number	8	
Type	Dual Media	
Filtration Rate	12 $m^3/m^2 \cdot h$	(5 gpm/sf)
Headloss	2 to 2.5 meters	(6 to 8 feet
Backwash Rate	37 to 49 $m^3/m^2 \cdot d$	(15 to 20 gpm/sf)

chemical mixing, thereby reducing the power requirements and operating costs. The velocity gradient of G = 1,000 sec^{-1} should be maintained in the mixing chamber for approximately 15 seconds. This condition will be developed with a multijet diffuser gate using 4.5 to 7.5 meters of available head. The multijet diffuser gate was originally developed by the MWD and has been used in their plant designs. The device is similar to a slide gate with multiple diffuser ports which convert the entrance pressure into turbulent energy for mixing. A sectional view of the diffuser gate is shown in Figure 3.

Where sufficient head is not available, the more conventional means of a mechanical mixer would be employed in the same basin.

MULTIJET SLIDE GATE



FLOCCULATION BASINS

Concern for the energy situation, along with the high cost of mechanical equipment, led ES to evaluate the possibility of tapered high energy flocculation without mechanical mixers. Although baffled flocculation basins have been widely used in the past, they are rarely considered today in the design of new plants because the mixing energy in conventional baffled basins is directly related to the water velocity through the basins, and insufficient energy is available for satisfactory flocculation at low flow rates through the conventional around-the-end or over-and-under baffles. Baffled basins seldom short-circuit and never have mechanical difficulties, which are often the problems in basins with mechanical flocculators.

To develop tapered-energy flocculation with an overall higher-energy input at reduced flow rates, a modified flocculation basin design and baffle system was developed. The floor of the mixing basin is sloped to maintain minimum water depth immediately beyond the chemical mixing basin, gradually varying to a maximum water depth at the end of the flocculation basin. Therefore, the velocity is highest when the water first enters the flocculation basin and gradually decreases as the water progresses through the basin. Since the headloss around the baffles, and thus the energy input, is directly related to the velocity, this design results in a tapered-energy application during flocculation. The shallow depth of water at the beginning of the mixing basin promotes relatively high velocities and energy input in the early stages of flocculation even at reduced flow rates, mitigating the main problem with conventional baffled mixing basins. The overall reduced level of energy applied at lower flows is counter-balanced by the increased time of flocculation.

To facilitate the design of the baffle flocculation basins, ES has developed a computer program which analyzes the number, size and spacing of the baffles. The computer program calculates the velocity gradient profile through the basin, the total detention time and the GT value for the entire basin over a range of different flow rates. For the Oceanside design, the mean velocity gradient varies through flocculation basins from 200 sec $^{-1}$ to 8 sec $^{-1}$ at a flow of 15,000 m^3/d per basin. The variation of the velocity gradient along the flocculation basin for both flow rates is presented in Figure 4.

The desirable values of GT (a dimensionless product of the velocity gradient and time) are in the range of 30,000 and 150,000. The value of GT in the tapered flocculator over a flow range of 30,000 to 65,000 m^3/d is approximately 65,000 to 80,000 with both lines in service. The effect of the variation in flow on GT through this system is presented in Figure 5.

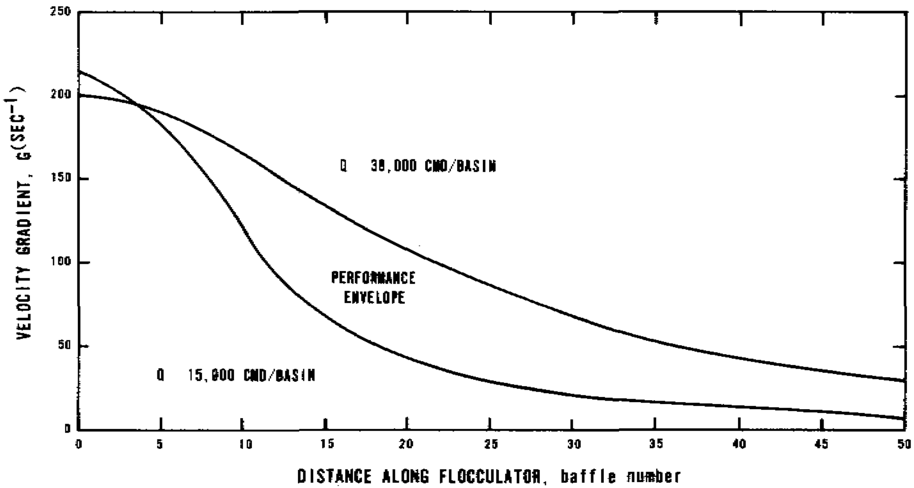
As a means of field adjusting the theoretical to the practical, the mean velocity gradient can be varied for a given flow rate by respacing or reducing the number of baffles.

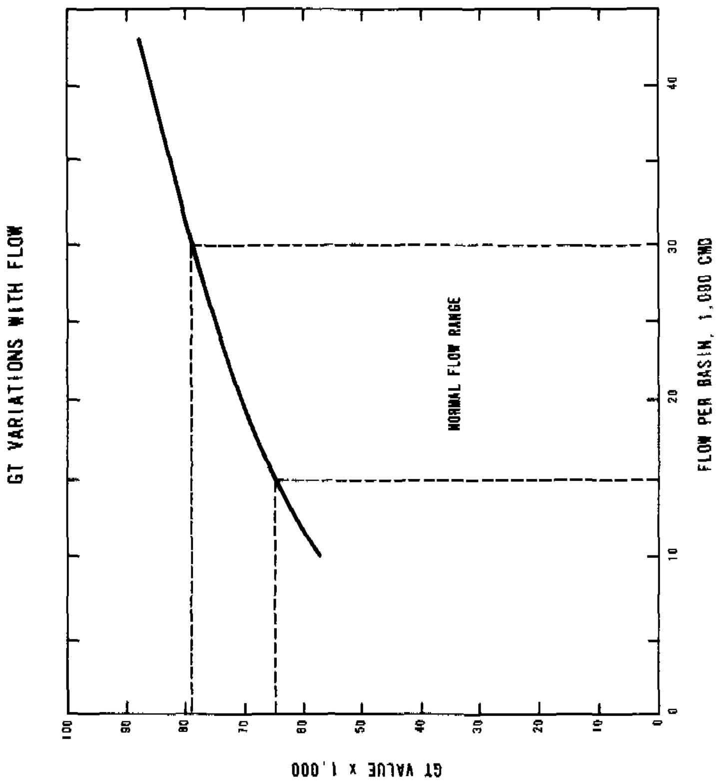
SETTLING BASINS

The removal of coagulated solids is accomplished in two rectangular sedimentation basins. The basins are designed for an overflow rate of approximately 57 $m^3/m^2 \cdot d$ and a detention time of 60 minutes. The maximum horizontal velocity through these basins is estimated at 0.76 m/min.

It is not uncommon practice, where low solids waters result in low sludge production, to design sedimentation basins without sludge collecting devices. In these instances, deeper basins are required and a basin must periodically be taken out of service for cleaning. Although this approach certainly results in plant simplification, there are additional cost

MEAN VELOCITY GRADIENT VARIATIONS WITH FLOW




GT VARIATIONS WITH FLOW
NORMAL FLOW RANGE
FLOW PER BASIN, 1,000 CMD
0
10
20
30
40
GT VALUE x 1,000
0
10
20
30
40
50
60
70
80
90
100

trade-offs required and potential for producing soluable odor and taste producing compounds which must be considered in design.

The sludge collection system for Oceanside consists of a traveling bridge suction lift unit. This unit differs from conventional suction-type units in several respects. Most conventional units have scraper blades, usually set to form a V with a vertical suction pipe at the base of the V to draw up the accumulated sludge. The ES design provides a horizontal pipe header that spans the width of the path to be cleaned. By means of wheels mounted at each end of the header, it travels 1.25 centimeters above the floor of the basin as the traveling bridge moves forward. Each header has ten 2 centimeter holes evenly spaced along the bottom and a fabric-reinforced rubber squeegee attached to the back. The squeegee sweeps the floor and helps to concentrate the sludge under the header as the bridge moves forward. It also prevents the water in the cleaned area behind the squeegee from being drawn into the suction header. For this reason, sludge withdrawn from the settling basin may contain up to 5 percent solids.

The settling basins are divided by low concrete curbs into longitudinal strips. The traveling bridge has steel frames suspended from it and each frame supports a suction-header assembly to sweep within longitudinal strips in the basin floor. Small pumps or air lift device on the bridge, provide the suction to draw the sludge up to a pipe under the bridge. Sludge is discharged into a longitudinal trough along the top of the basin side wall and is conveyed to the sludge drying beds. Depending on the raw water quality and the subsequent amount of settled matter, the collection unit may be operated as frequently as once per day or as infrequently as once per week.

In addition to more effective sludge removal, several other factors led to the decision to use the traveling bridge and suction system rather than the flight or rotary sludge collectors. One of these was the elimination of the tunnels and pump rooms beneath the basins, which are required with other types of collectors and add substantially to concrete substructure cost. Also, the hinges on the support frames for the suction headers permit any header to be raised out of the water on to a working platform for servicing while the basin remains in operation.

FILTERS

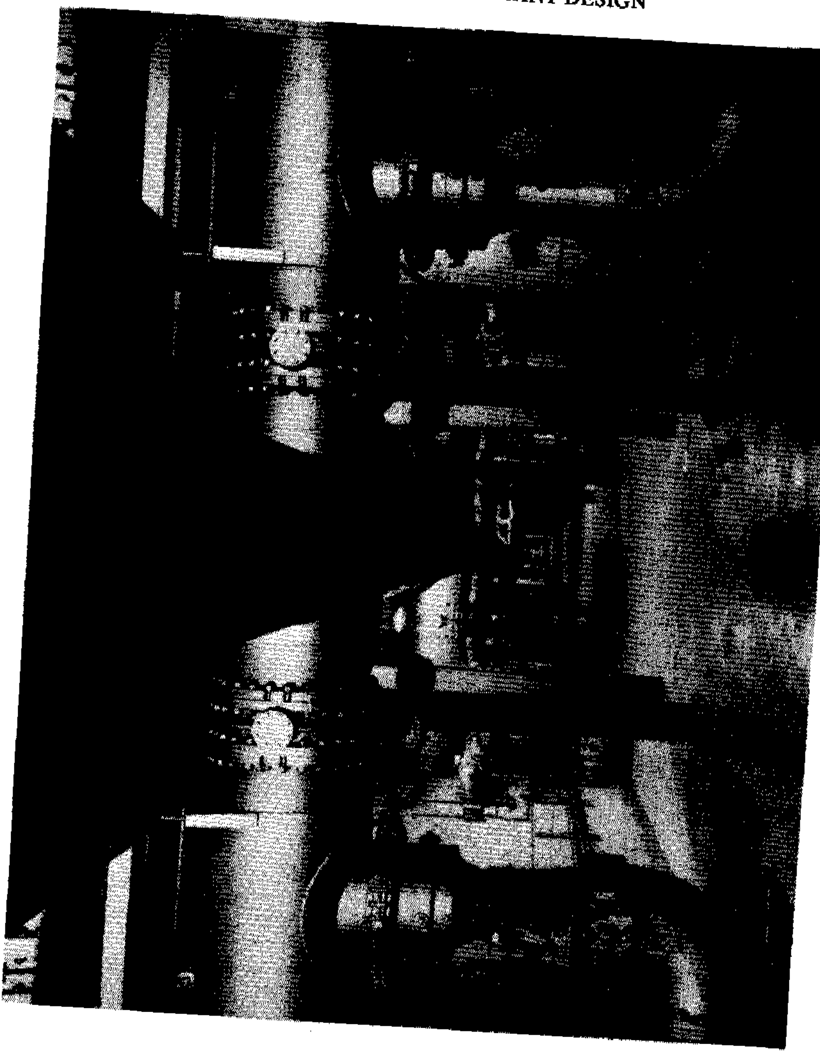
Following coagulation and sedimentation, the water is processed through eight dual media filters consisting of 45 centimeters of gravel, 25 centimeters of sand and 45 centimeters of anthracite coal. The gravel, of a specified transition gradation, will be placed in four layers above the top of the filter underdrain system. The sand will have an effective size of 0.43 to 0.50 millimeters and a uniformity coefficient of not more than 1.65. The anthracite coal will have an effective size of 1.0 to 1.1 millimeters and a uniformity coefficient of not more than 1.65. Media of this type is capable of maintaining surface loading rates of 12 to 15 $m^3/m^2 \cdot hr$. Depending on the raw water quality and the resulting solids loading on the filter greater filtration rates can be economically achieved. The design rate for the Oceanside project is 12 $m^3/m^2 \cdot hr$ under maximum flow conditions or 14 $m^3/m^2 \cdot hr$ under maximum flow conditions and one filter being backwashed.

In many countries, the use of dual media using anthracite coal is not practical owing to local non-availability and/or reluctance to import. Therefore, the selection of criteria must be compatible with the media available. Efforts have been made to develop dual medias from other material which have the appropriate relative specific gravities and can be produced in the appropriate sizes. Designing effective media can only be accomplished by knowing the nature of the particles to be removed and the range of solids loadings. The best means of obtaining these data are through pilot studies of the waters to be treated.

Traditionally, large filter plants are designed with below-grade pipe galleries, access tunnels and pump pits containing elaborate piping, valving and controlling systems. Figure 6 presents a view of such a typical pipe gallery containing extensive piping and valves which not only must be maintained, but also represent a major portion of the filtration system complexity and cost. Further, maintenance of such pipe galleries adds to the housekeeping duties at a plant. One of the objectives in simplifying the plant was to eliminate these pipe galleries by revising the filter operation and backwashing.

The method of operation of the new Oceanside filter units can be seen from Figure 7. From the filter forebay, the water enters the unit through the backwash troughs (the forebay drain valve is closed). At the start of filtration, the water in the filters is at the level controlled by the effluent weir since the effluent conduit is common to all filters. The water slowly rises above that level to reflect the initial head loss at the start of operation and continues to rise as head loss builds up during filtration.

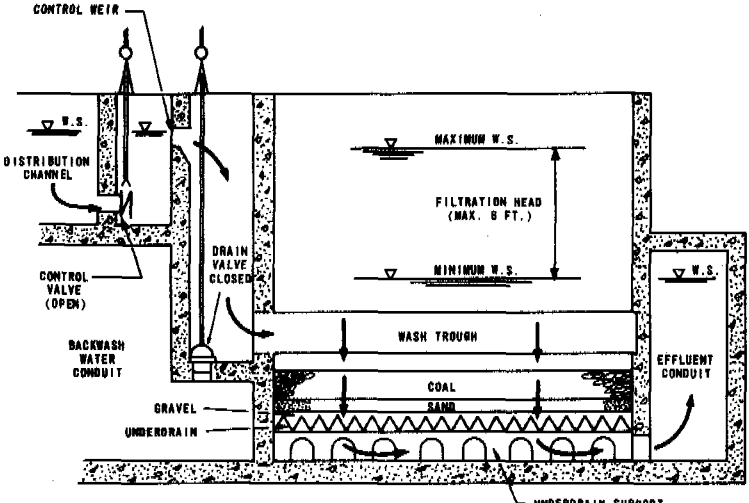
At no time during filtration is the water level in the bed or in the effluent conduit below the level of spill over the effluent weir. Hence, negative head cannot be developed in or below the filter. When the level in the bed rises to a point that is preset for maximum desirable head loss, a level sensor initiates closure of the filter-influent valve. When the



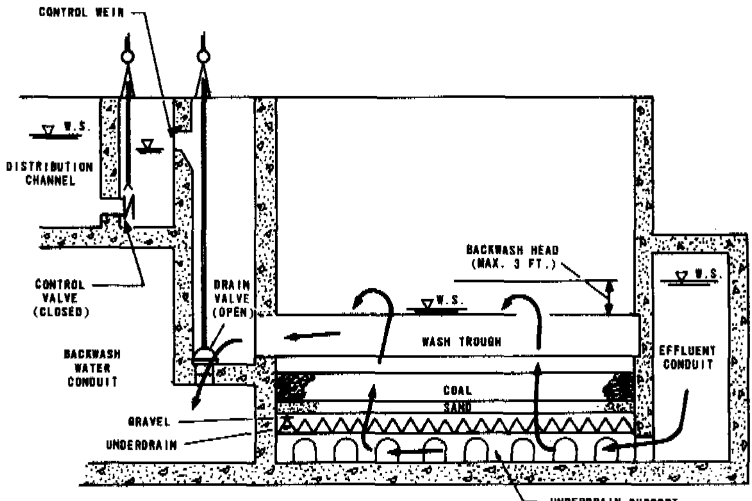
TYPICAL FILTER PIPE GALLERY

FIGURE 7

FILTER OPERATION



FILTRATION MODE



BACKWASH MODE

inflow is shut off, the water in the bed slowly drains down as filtration continues until it reaches its initial level established by the effluent weir. The drain gate in the filter forebay is then opened, permitting the water in the bed to drain to the level of the backwash trough lip. As the water level in the filter falls below the level in the filtered-water conduit, a reverse flow of filtered water upward into the bed gradually builds up until the maximum backwash rate is attained when the water in the filter unit reaches the overflow level into the backwash troughs. The difference in water levels is then at a maximum.

With this design, a backwash rate of 49 $m^3/m^2 \cdot hr$ can be attained with approximately 0.75 meters of head. To compensate for changes in water density with temperature and to provide flexibility in adjustment of backwash rates, the plant filter-effluent weirs are adjustable to provide a range of 0.5 to 1.0 meters of head for backwash. These low heads are adequate because the plenum under the filter is large enough and velocities are low enough that variations in head within the plenum are less than 4 centimeters, thereby eliminating the need for high head loss through the underdrain in order to ensure uniform distribution of backwash water.

The required backwash water is derived from the plant effluent conduit; therefore, the plant discharge is reduced when backwash is required. A discharge replenishment tank or clear well may be needed to balance the plant discharge with the water system demand. As an example of this condition with respect to the Oceanside Water Treatment Plant, at a maximum backwash rate of 49 $m^3/m^2 \cdot hr$, the minimum plant flow required to backwash one filter is 32,000 m^3/d. Below a minimum plant flow of 32,000 m^3/d, a backwash storage tank is required to supplement the water produced by the filters in service and thus supply a backwash rate of 49 $m^3/m^2 \cdot hr$; however, backwash rates as low as 37 $m^3/m^2 \cdot hr$ may also be adequate. The anticipated flow range for the Oceanside plant is 30,000 m^3/d to 65,000 m^3/d; therefore, a backwash tank is not necessary. Should the operational mode change in time and lower plant flows are desired, an area of the plant site has been allocated for the addition of a supplemental backwash water tank.

INSTRUMENTATION

The degree of instrumentation sophistication depends upon the capabilities of the owner to operate and maintain equipment. It is obvious from the simplicity of the plant described above that there are minimal requirements for instrumenting the plant. The only necessary instrumentation is influent flow monitoring, chemical feed measure, and an annunciator panel for failure alarms. Because the filters are constant rate increasing head, backwash can be initiated upon visual observation of excessive head loss. The backwash operation can be easily accomplished manually.

To facilitate the operation of the Oceanside plant and to minimize the manpower requirements, advanced instrumentation and control techniques were designed into the project. Instrumentation and control devices will permit the plant to operate unattended for at least two shifts per day. During the work shift two or three operators will be needed. The major automatic process control functions are contained in the main control panel (MCP) in the control building. Local control panels are located at the chemical mixing systems, the traveling bridge sludge collector, filters, and pumping stations.

Plant influent, recovered backwash and sludge conveyance water and plant effluent flows will be metered and transmitted to the MCP for recording, totalizing and for control of the chemical systems. Individual flow meters will be used for measuring and control of the distribution of the various chemicals such as polymers, chlorine and sodium hydroxide which can be injected at multiple and variable points in the process. The filter surface washwater will be metered for adjusting flow rates.

The plant influent and effluent will be monitored by automatic analyzers for chlorine residual, turbidity, and pH. The turbidity of recovered backwash water, which is returned for treatment, will also be analyzed. These analyses will be transmitted to the MCP for recording, control and initiating alarms (audible and visible) when parameters deviate from desired values.

One of the principal automatic operations is the backwashing of filters. The filter backwashing system is capable of automatic or semi-automatic control from the MCP or manual control from local panels at the filters. For automatic control the initiation of a backwash cycle is controlled by head loss with a time clock override. The semi-automatic cycle will be initiated manually by pushbutton on the MCP. The automatic and semi-automatic cycles are identical except for the manner of initiation. The automatic cycle will be controlled on the basis of timers, and level sensors. Valve and gate position switches and flow switches will be used to interlock the sequence to prevent and detect faulty operation. Backwash operation at the local filter panels will require all manual control of gates, valves, and surface-wash pumps. Indicating lights controlled by limit switches will show valve and pump status. All valves and gates are motor operated.

PLANT COSTS

Considerable cost savings may be realized with this simplified water treatment plant concept. Lower operation and maintenance cost can be achieved due to reduced power requirements and minimum manpower expenditures.

It was estimated that the annual cost savings for the baffled flocculators would be approximately $8,000 (March 1977 dollars) for the Oceanside Project. The analysis is based upon the construction cost amortized at 7 percent for 30 years plus the annual power and personnel requirements.

The construction costs can also be significantly reduced due to the elimination of pump galleries, pipe galleries, piping, valves and fittings. A conventional water plant for Oceanside was expected to cost between $4,500,000 to $5,000,000. The plant, as designed and described above, has an estimated construction cost of $3,700,000 which includes some unusually heavy site earthwork. It is obvious that the cost differential becomes increasingly greater with larger sized plants. Other cost examples which use the above concepts for sedimentation and filtration are the Robert A. Skinner Filtration Plant at $13,000,000 for 570,000 m^3/d capacity and the Henery J. Mills Filtration Plant at $8,600,000 for 285,000 m^3/d capacity. Conventional plants of these capacities would have cost $20,000 and $16,500,000, respectively.

SUMMARY

Treatment plants to produce potable water need not be elaborate and complicated nor must they be conservatively designed to old fashion and costly standards. Innovative and simple and economical plant design has been successfully demonstrated to the satisfaction of health authorities. New plants, particularly in the Western United States, are trending to these modern principles.

INTERMEDIATE SERVICE LEVELS IN WATER DISTRIBUTION

BY

Donald T. Lauria, M. ASCE, Peter J. Kolsky, M. ASCE,
and Richard N. Middleton 1/

ABSTRACT

This paper is concerned with design standards for secondary water distribution networks. Based on field studies in the Middle East and Africa, mathematical equations were developed for predicting the length and mean diameter of network piping for given values of system variables. When coupled with local cost data, these equations enable prediction of system costs for a variety of design conditions, and hence provide a basis for decisions on design variables such as the spacing of public standposts, minimum network pressure and per capita flow.

INTRODUCTION

Recent estimates suggest that less than 300 million of the two billion people in the developing countries have access to adequate supplies of safe water and to proper sanitation (Ref. 1). Rapid growth of slums and squatter settlements in urban areas of the developing world present an urgent problem. The water and sewer systems of cities, already operating under considerable difficulties, are rarely expanded to meet even the most modest requirements of slum and squatter districts, whose inhabitants therefore resort to polluted water sources or to water purchased from vendors at rates much higher than those charged by the water utility.

One problem in extending water supply to cover these densely-populated and rather unstructured poor districts has been that designers tend to size the secondary distribution networks by 'rule of thumb' methods rather than on the basis of rigorous analyses. The resulting designs are generally oversized and expensive so that many are never actually constructed. Also, many designers are inclined to consider why networks designed to serve house connections or yard faucets rather than simplier systems with public standposts, which are more affordable.

1/ Respectively, Professor, University of North Carolina, Chapel Hill, N.C. 27514; Senior Engineer, Camp, Dresser and McKee, Boston, Mass. 02108 and Senior Sanitary Engineer, The World Bank, Washington, D.C. 20433.

The work reported herein represents the views of the authors and not necessarily those of the World Bank, nor does the Bank accept responsibility for its accuracy of completeness.

Recognizing the need for more appropriate water supply technology in developing countries, especially for migrants and the very poor, the World Bank commissioned a study in 1975 of minimum design standards. With principal emphasis on secondary distribution systems rather than major facilities like supply, transmission and primary networks, the Bank raised such basic questions as: How many persons should be served by each standpost? what is optimal standpost spacing? how much flow should be supplied per capita? what minimum pressure should be maintained in networks? and what is minimum acceptable pipe diameter?

As in the case of all standards, answers to these questions can in principle be obtained by benefit-cost analysis. Consider optimal standpost spacing, for example; it would be necessary on one hand to know the benefits associated with different spacings (they are essentially related to health and convenience), and on the other, information would be needed on costs (construction, operation, wastewater disposal, etc.). The optimal spacing, which maximizes net benefits, is that for which the marginal costs and benefits are equal.

In practice, one is most unlikely to be able to quantify the benefits of such things as the *spacing of standposts*, the amount of water supplied per capita, and minimum network pressure is not good. Consequently, this study focused primarily on costs, with the understanding that if accurate functions were available, they could be used to determine the incremental expense of changing such things as the per capita flow from, say 25 liters per capita per day (lcd) to 50 lcd, or average standpost spacing from, say, 100 meters (m) to 60 m. Then it could be left to the user to decide whether his perception of the benefits associated with incremental changes was worth the cost.

The study methodology for developing cost functions comprised the following steps:

(1) identify the decision variables (i.e., those items for which standards are sought);
(2) select a variety of study regions in cities throughout the world;

(3) design secondary systems for the selected regions using different values for the decision variables;

(4) determine the quantities of materials and the costs of the alternative designs; and

(5) determine the association between costs and decision variables to obtain the desired function.

A different approach for developing cost functions might have been to find areas for which water systems had already been designed, and then to analyze them in light of their stated criteria. Although this approach was considered, it was rejected when trial studies in Latin America indicated that the actual capability of systems did not in fact match the standards nominally used for their design (i.e., that the systems were overdesigned and hence technologically inefficient). Consequently, the former methodology was adopted.

DECISION VARIABLES

The principal decision variables over which the designer of secondary water distribution networks has control include (1) the quantity of water supplied per person, (2) the number of standposts to be employed, and (3) the minimum allowable pressure to be maintained in the system. If the standposts are more or less evenly spaced (as assumed in this study), the number N provided for a given area A is approximately related to the maximum walking distance R for carrying water by the relationship

$$R = 56\sqrt{A/N} \qquad (1)$$

where R, the radius of the circular area served by each standpost, is in m and A is in hectares (ha).

Clearly, the designer has control over numerous additional variables (for example, pipe materials, type of joints and the details of standpost design such as the number of faucets), most of which were either ignored or set to specific values in this study. It was assumed that systems would be continuously under pressure (although this is not true in many developing countries). Fire protection was ignored. In general, branched rather than looped systems were used in order to minimize pipe length, and the shortest possible routes were selected for connecting standposts to the primary distribution mains. The study included several designs that employed private yard or patio faucets instead of public standposts. In these cases, the need to serve each house required that looped rather than branched layouts be used. Throughout the study, alternative designs for given regions were homogeneous, in that a standpost network was never incorporated into a system wih yard faucets, nor were two or more standpost spacings used in the same system.

STUDY REGIONS

The principal work in this investigation was done for two squatter areas in the Middle East and one in West Africa. Each of the areas was visited and outline designs prepared in the field, with detailed study being carried out on return. The Middle Eastern city has a population of 135,000 which is growing about 7% per year, divided more or less evenly between migration and natural increase. Most of the city obtains water from private wells, many of which are polluted. Vendors play a prominent role in distribution; they charge the equivalent of about US$8 per cubic meter ($m^3$). The ground water table is falling at the rate of 3 m/yr. A new well field, treatment plant and primary distribution system are under construction.

The first study region (Zone 1) has an area of 29 ha and population of 10,000. The houses are large, with 4 to 6 storeys; the average number per ha is 52. The streets are narrow, winding, and cannot easily accommodate vehicles. The terrain is flat. Zone 2 in the Middle East has 40 ha and 4,000 persons. The houses are lower and often detached; streets are wider and better able to handle traffic. The average housing density is 20/ha.

The population of the city in West Africa is about 200,000 with annual growth exceeding 6%. Nearly half the population is in squatter areas. The nonsquatters obtain water from a municipal system in quite good condition. About 35% have house connections and draw an average of 70 lcd; the remainder rely on standposts and vendors, using an average of less than 10 lcd. Vendors charge a price equivalent to US$3 per m^3. The study region in this city (Zone 3) has 185 ha. It is a squatter neighborhood that is being upgraded and is expected to have a population of 22,200 residing in nearly 3,000 single-story dwellings. The terrain is flat.

SYSTEM DESIGN

The work of generating alternative designs began with Zone 1. Three different spacings were selected: standposts with approximate maximum walking distances of 100 m and 50 m, and individual courtyard connections. With A = 29 and R = 100, the number of standposts N from Eq. 1 (after rounding to the nearest whole value) is 9; similarly, with R = 50, N = 36. In fact, after the site inspection, it was decided to use only 33 standposts for this spacing which resulted in maximum walking distance slightly in excess of 50 m. In the case of courtyard faucets, a total of 1,475 were required which, from Eq. 1, implies a maximum carrying distance of about 8 m.

Three levels of average per capita consumption were selected for standpost service: 20, 50 and 100 lcd. The upper values are unrealistically large, but they were desirable from the standpoint of this exercise which sought water system characteristics and costs over a wide range of flows. For yard faucets, only 50 and 100 lcd were considered. A peak factor of 3 was assumed for network design.

The secondary network for Zone 1 was connected at only a single point to the primary main which supplies the area. The minimum head at the point of connection under peak demand conditions is known to be about 25 m. It was first assumed that the minimum allowable pressure at standposts and yard faucets was 15 m, thus allowing a head loss of 10 m across the distribution network. Later, the minimum pressure was raised to 20 m, reducing the available loss to only 5 m.

The design conditions in Zone 2 were similar to those in Zone 1. Maximum walking distances of about 100 m and 50 m were selected which resulted in 11 and 43 standposts, respectively. A total of 785 yard faucets were required which implied a walking distance of about 13 m. As before, average flows of 20, 50 and 100 lcd (corresponding to peaks of 60, 150 and 300 lcd) were used for standposts, with the smaller value deleted for courtyard connections. Maximum allowable head losses of 10 m and 5 m were assumed. For Zone 3, the design criteria were modified to reflect its lower economic status. Three standpost spacings with maximum walking distances between about 140 m and 240 m were investigated; courtyard faucets were not considered. Four different system head losses in the range of 5 m to 17.5 m were analyzed. Average flows in the range of 10 to 100 lcd were selected; systems were designed, however, for maximum demands assuming a peaking factor of 3. The basic data and design standards for all three zones are summarized in Table 1.

TABLE 1

Design Criteria

Zone	1	2	3
Population	10,000	4,000	22,200
Area, ha	29	40	185
Houses	1,475	785	3,000
Population/ha	350	100	120
Houses/ha	52	20	16
Standposts (N)	9, 33, 1475	11, 43, 785	10, 17, 31
Service Radius (R)	101, 52, 8	107, 54, 13	241, 185, 137
Persons/Standpost	1110, 303, 7	364, 93, 5	2220, 1306, 716
Average Flow, lcd	20, 50, 100	20, 50, 100	10, 20, 30, 40, 50, 100
Peaking Factor	3	3	3
Head Loss, m	10, 5	10, 5	17.5, 15, 10, 5

After selecting design conditions and making network layouts to minimize pipe length, it was necessary to determine pipe diameters such that standards were satisfied at minimum cost. Analysis of construction bid prices showed that network piping costs (in 1976 US$ per m of length) could be approximated by (5 + 0.24 D) and (0.20 D) in the Middle East and West Africa, respectively, where D is diameter in millimeters (mm). Using these cost functions and a linear programming approach described by Robinson and Austin (2), optimal designs were obtained for the branched standpost systems in all three zones. In the case of networks for courtyard connections in Zones 1 and 2 where looped rather than branched systems were needed, the linear programming model could not be used because the flow in each pipe was unknown. Consequently, it was necessary to employ a simulation model which produced near-optimal designs; a computer program by Epp and Fowler (3) was used.

MATHEMATICAL MODELS

A total of 10 different designs in Zone 1, 10 in Zone 2, and 24 in Zone 3 were generated. Data were tabulated for each design giving the average flow Q in lcd, the number of standposts or courtyard connections N, maximum allowable head loss across the system H in m, area of the region served A in ha, total population P, total pipe length L in m, average pipe diameter $\bar{D}$ in mm, total pipe cost Cp and total system cost C_T. Average diameter for each design was obtained from the relationship.

$$\bar{D} = \sum_i L_i D_i / \sum_i L_i \qquad (2)$$

where D_i and L_i are the diameter and length of the ith piece of pipe in the network and the summation is made over all pipes.

The data for all 44 designs were pooled (i.e., no distinction was made between branched and looped systems), and mathematical models were fitted to them using linear regression analyses. The first model was for total pipe length L which was assumed to depend on only two variables, the area of the region served A and the total number of standposts or yard faucets N. The resulting equation is

$$L = 90\ N^{0.4}\ A^{0.6} \quad (3)$$

for which $R^2 = 0.98$. The fit of the model is excellent, with F (2, 6) = 129; the partial F values for N and A are 162 and 97, respectively.

Next, a model was fitted to the data with mean diameter $\bar{D}$ as the dependent variable and the number of standposts or yard hydrants N, total population P, area A, average design flow Q, and system head loss H as the independent variables. The relationship is

$$\bar{D} = 3.9\ N^{-0.17}\ P^{0.22}\ A^{0.10}\ Q^{0.38}\ H^{-0.23} \quad (4)$$

Although Q is average design flow, this equation implicitly incorporates a peaking factor of 3. The statistics for Eq. 4 are $R^2 = 0.99$ and F (5,38) = 570, with partial F's from 100 to 1400; the fit is excellent. Deleting H from the model results in

$$\bar{D} = 3.6\ N^{-0.17}\ P^{0.21}\ A^{0.05}\ Q^{0.37} \quad (5)$$

and deleting both H and A yields

$$\bar{D} = 4.5\ (P/N)^{0.21}\ Q^{0.39} \quad (6)$$

with R^2 values of about 0.95 and 0.9, respectively.

From the above expressions, it is possible to estimate total pipe length and mean diameter for selected values of the decision variables N, Q and H, and given values of A and P for the area served. Suppose now that the unit cost per m of length is known for pipe diameter $\bar{D}$. Multiplying this price by total length L from Eq. 3 results in an estimate of total pipe cost for the network. In mathematical symbols, let C(D) be the known cost per m of length for pipe with diameter D. Then total piping cost Cp is

$$Cp = 90\ N^{0.4}\ A^{0.6}\ C(\bar{D}) \quad (7)$$

where $\bar{D}$ is obtained from Eq. 4, 5 or 6. If the unit cost of standposts or hydrants C_s is also known, the total system cost C_T can be estimated from the following

$$C_T = 90\ N^{0.4}\ A^{0.6}\ C(\bar{D}) + NC_s \quad (8)$$

Thus eq. 8 is the desired function that relates the total system cost to the decision variables N, Q and H.

APPLICATIONS

Before illustrating the application of the above equations to design, it is useful to make some observations about them. Note in Eq. 3 that for a given area, length increases as the number of standposts increases, but at a decreasing rate; the function is concave. Doubling the number of standposts, which is equivalent to halving the number of persons served per standpost, increases total pipe length about 30%. Note that N can be replaced by the maximum carrying distance R through use of Eq. 1. Making this substitution in Eq. 3, it follows that reducing the carrying distance by half increases network length about 70%.

All three equations for $\overline{D}$ give approximately equal results. Eq. 6 is particularly attractive because of its simplicity; it says that mean diameter depends on only two variables, the number of persons per standpost P/N and average design flow Q, $\overline{D}$ is fairly insensitive to both variables. A fivefold increase in flow, for example, increases mean diameter only about 85%, assuming P/N is constant. Similarly, for a given flow a fourfold increase in P/N results in only about 30% increase in mean diameter. These relationships allow one to explore the optimum diameter of pipes to be laid when a subsequent improvement in service (such as upgrading from public standposts to house connections) is anticipated. Of course the length of pipe is greater for networks with house connections than with standposts; the tradeoff can be seen by examining how L in Eq. 3 increases and $\overline{D}$ in Eq. 6 decreases as N increases.

Because Eq. 6 has only two independent variables, a graph can be made indicating the tradeoff between P/N and Q for networks of given mean diameter, as shown in Fig. 1. A network with $\overline{D}$ = 25 mm has capacity to deliver average flow of about 25 lcd at a peak factor of 3 to 10 persons per connection. Increasing $\overline{D}$ to 50 mm enables the supply to be increased to 150 lcd under the same conditions. A network of this same average size can also deliver 40 lcd average flow to 100 persons per standpost or 20 lcd to 350 persons per standpost.

Eqs. 4, 5 and 6 only describe mean diameter, but an analysis was also made of the variation in pipe diameter as a function of length. For the branched systems, about 20% of total pipe length had diameters equal to or less than 0.65 $\overline{D}$, and 20% of the total pipe length had diameters equal to or greater than 1.35 $\overline{D}$. For looped networks, the equivalent diameters at the 20 percentiles were 0.45 $\overline{D}$ and 1.55 $\overline{D}$, respectively.

In the remainder of this section, only Eq. 6 is used for estimating $\bar{D}$. Assume that pipe cost per m of length C(D) can be estimated by

$$C(D) = 0.2\ D^{0.9} \qquad (9)$$

where D is diameter in mm. Further assume that the unit costs of standposts and yard faucets C_s are 500 and 100, respectively. Suppose a population P of 30,000 living in an area A of 100 ha is to be served by standposts. If the designer wants to limit maximum walking distance R to about 100 m, the number of required standposts from Eq. 1 is 30. The estimated length of the network from Eq. 3 is 5600 m, and average diameter from Eq. 6 is 47 mm, assuming an average per capita flow of 10 lcd. By Eq. 9, the cost per m of length for pipe of this size is 6.40 from which it follows that total pipe cost Cp is about 35,000. Adding the cost of 30 standposts (15,000) gives a total cost C_T of about 50,600 and per capita cost of 1.69.

Assume now that the designer wants to consider a somewhat better level of service. Instead of 1000 persons per standpost, 500 is preferred, with average per capita flow of 25 lcd at a peak factor of 3. The maximum walking distance for this number of standposts would be reduced from about 100 m to 70 m. Repeating the calculations, L = 7300, $\bar{D}$ = 58, C(58) = 7.73, Cp = 56,700 and C_T = 86,700. Thus the improved service has an incremental cost of 36,100 which is about 1.20 per capita. Whether the increase in benefits is worth this cost is for the designer to decide. If indeed the users are willing to pay at least this amount for the extra convenience and additional flow, then a still higher level of service should be considered, the iterations ending at the point where incremental benefits are judged to be just equal to incremental cost.

In the above example, the total per capita cost is 2.89. Suppose, however, that the total expenditure cannot exceed 75,000, which is equivalent to 2.50 per capita. To design within this constraint, assume that an average flow is selected, say, 25 lcd. The problem, then, is to decide the number of standposts N. From Eq. 3, the length of the network is 1426 $N^{0.4}$, from Eq. 6 the average pipe diameter is 138 $N^{-.21}$, and from Eq. 9, the unit cost of this size pipe is 16.8 $N^{-.19}$. The resulting expression for total cost is 23,957 $N^{0.21}$ + 500 N = 75,000 which yields N = 45. With this number of standposts, each will serve 670 persons on the average and will have a maximum carrying distance of 84 m.

A difficult problem for designers is to make an initial choice of pipe sizes to serve standpost networks that must later be compatible with the piping required for house connections after upgrading (compatibility here implies the use of pipe with sufficiently large diameter to avoid uneconomical replacement as the area switches from standposts to connections). One approach to design is the following. Suppose the area in the previous examples includes 3000 houses for which the ultimate target design flow is 200 lcd. From Eq. 3, the final length of pipe in the network is 35,100 m, with mean diameter 58 mm from Eq. 6. Assuming the standpost system is to be initially designed for an average per capita flow of 25 lcd, the number of standposts that can be supported by a network with the identical mean diameter of 58 mm can be found from Eq. 6 to be about 60, and maximum walking distance is about 70 m. From Eq. 3, the length of the initial standpost system is 7300 m. Thus it appears that if the initial system is designed for $N = 60$ and $Q = 25$, it will consist of pipe that will be essentially of the same diameter as that required to supply 200 lcd average flow after upgrading to house connections. However, care should be taken in designing the detailed layout of the initial system to ensure that the distribution of pipe sizes is compatible not only with the standpost layout but also with the eventual demands of the system with house connections.

In the previous example, it might be asked whether it would be preferable to initially construct only the 7300 m of pipe required for the standpost system, with network extensions to be made as required, or to build the ultimate system initially using only standposts that are gradually replaced by house connections. Assume for simplicity that the two alternatives are (1) construct in year zero 7300 m of $\overline{D} = 58$ mm pipe plus 60 standposts; expand the system to total network length of 35,100 m with $\overline{D} = 58$ mm and construct 3000 house connections in year 5 and (2) construct 35,100 m of $\overline{D} = 58$ mm pipe plus 60 standposts in year 0; construct 3000 house connections in year 5. Because the identical facilities are required with both alternatives, the total construction cost is the same at 601,300. However, the present value costs of the alternatives are different because of differences in staging. With a discount rate of 6% per year, the present value costs of alternatives 1 and 2 are 471,200 and 525,500, respectively. With higher discount rates (appropriate to reflect the opportunity cost of capital in developing countries), the difference is even more marked. Thus it is more expensive to initially build the ultimate system and have it remain partially idle than to more carefully match the supply of facilities with the demand that is made upon them. Although this example is overly simplistic, its result will generally hold true.

This section has included illustrative examples of some of the uses that might be made of the equations resulting from this research. For a more complete description, the reader is referred to Ref. 4.

SUBSEQUENT STUDIES

Eqs. 3, 4, 5 and 6 are calibrated predictive models for network pipe length and mean diameter; they constitute the principal findings of this report. Although they are based on 44 separate case studies, their accuracy remains uncertain until they are verified by additional field investigations. Such studies were made in three different cities during 1978; two of the cities are in the western Pacific and one in Latin America.

A total of 28 separate network designs were prepared for five study zones in the three cities. As in the original work in the Middle East and West Africa, the networks were designed for alternative standpost spacings (including courtyard connections), per capita flows, and system pressure losses. As before, branched networks were designed using linear programming techniques, and looped systems were designed using Fowler's program.

The area of the study zones ranged from 4.5 to 30 ha, and the population densities were between 150 and 1000 persons/ha. Zones in the Pacific were relatively flat like those in the original work, but the Latin American city was hilly with a maximum difference in ground elevation of about 80 m. As in the original work, data were obtained on total pipe length L and mean diameter $\bar{D}$ for each of the 28 different networks. Comparisons were then made between these observations and predictions of length and mean diameter using Eqs. 3 through 6. If the agreement between predictions and observations was good, the predictive model was considered to be verified; otherwise, modification was assumed to be necessary.

Work to verify Eq. 3 for total pipe length revealed dificiencies in this model. When applied to branched (standpost) networks, predictions were consistently less than observations; the error ranged from about 7 to 20%. However, when Eq. 3 was used for looped networks, predictions were always greater than observations by a substantial margin; errors ranged from 20 to 50%.

Eq. 3 most likely fails to apply to both branched and looped networks because it does not consider street configuration. In the case of branched networks, pipe length can be shortened as the number of streets in a given area increases because of more available paths for routing. In the case of looped networks, however, an increase in the number of streets has the opposite effect of lengthening the network. Unless street pattern is therefore taken into account, it is unlikely that a single equation can apply to both types of systems (an attempt to represent street configuration in the model by the number of blocks in an area was not satisfactory).

The data for all the branched networks including both original and subsequent studies were pooled and used for revising the length model. The resulting equation (with $R^2 = 0.90$) is

$$L = 82\ N^{0.55}\ A^{0.49} \qquad (10)$$

Similarly, the entire set of data for looped designs were pooled from which the following model (with $R^2 = 0.97$) was obtained

$$L = 105\ N^{0.32}\ A^{0.63} \qquad (11)$$

Hence it is recommended that Eqs. 10 and 11 (other than Eq. 3) be used for predicting the length of branched and looped networks, respectively.

Work to verify the predictive models for mean diameter was quite satisfactory; predictions using Eqs. 4, 5 and 6 agreed very well with observations for the 28 case studies. Hence, the original mean diameter models can be expected to produce accurate results.

Eqs. 7 and 8 should be modified to include Eq. 10 in the case of branched networks and Eq. 11 for looped systems. The respective equations for total piping cost Cp are

$$Cp = 82\ N^{0.55}\ A^{0.49}\ C(\bar{D}) \qquad (12)$$

$$Cp = 105\ N^{0.32}\ A^{0.63}\ C(\bar{D}) \qquad (13)$$

Total network costs C_T can be obtained by adding the cost of distribution devices NC_s to these expressions, as in Eq. 8.

FUTURE STUDIES

The boundaries of this research need to be expanded to take account not only of secondary networks, but of primary facilities including source works, treatment plants, transmission mains and the primary distribution system. A complementary project is needed for the associated wastewater systems. Optimal design procedures are needed for appropriate wastewater technology in developing countries. Specifically, improved methods must be developed for designing sewers, especially the type that can be used with aqua privies or septic tank effluents, where flows are low and settleable solids are not a serious problem. Just as initial standpost networks must be compatible with final house networks, initial sewers for collecting aqua privy overflows must be compatible with final collection systems that will provide ultimate service. Work on these problems has begun by first of all developing an optimization model for the design of sewers. Application of this model to sanitation design in Latin America is currently underway. Also, work is proceeding there on a more general analysis of the effects on primary water facilities.

Acknowledgements

The authors wish to acknowledge the support and encouragement of John Kalbermatten of the Work Bank and the technical assistance of Paul Hebert of the University of North Carolina at Chapel Hill.

REFERENCES

1. Saunders, R. J. and J. J. Warford, Village Water Supply: Economics and Policy in the Developing World, The Baltimore, 1976.
2. Robinson, R. B. and T. A. Austin, Cost optimization of rural water systems, J. Hydraulics Division, Am. Soc. Civil Engrs., FY8, pp. 1119-1134, 1976
3. Lauria, D. T., P. J. Kolsky and R. N. Middleton, Design of low-cost water distribution systems, P. U. Report No. RES 11, Energy, Water and Telecomcunications Dept., World Bank, Washington, D.C., 1977.

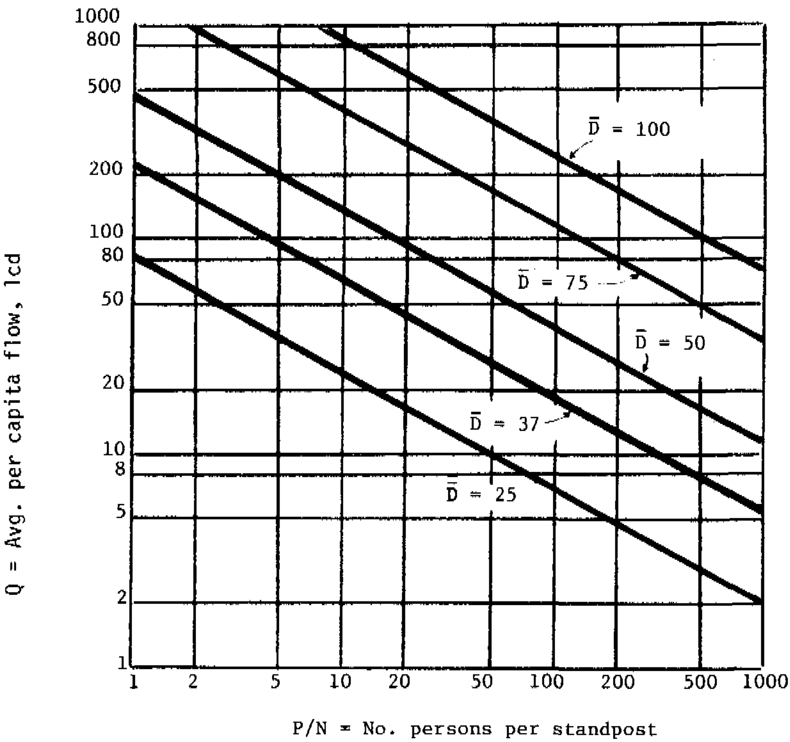


FIG. 1. Per capita flow vs. persons per standpost for various pipe sizes.

Plate III. Nightsoil removal in Seoul, Korea. There are many urban areas where access and other constraints make bucket systems for nightsoil removal the most appropriate sanitation system.

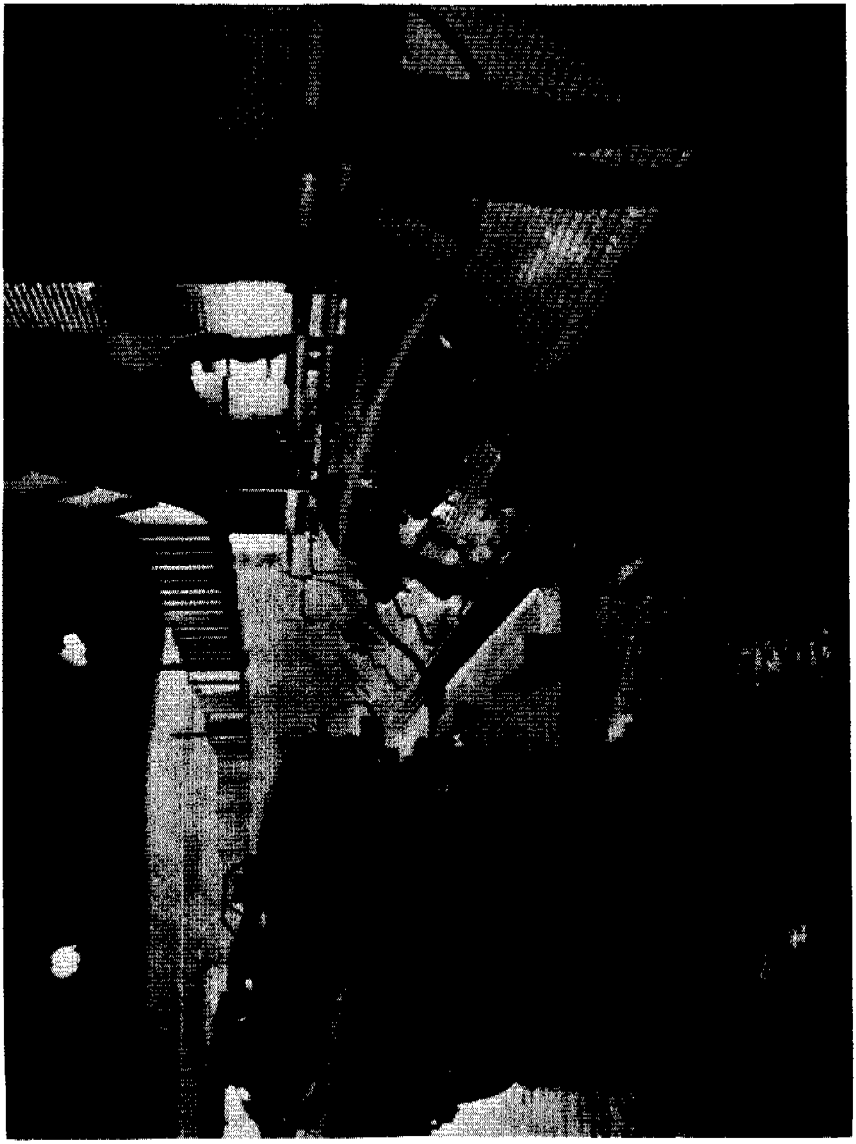


Plate IV. Nightsoil transfer station in Kyoto, Japan. Advanced technologies contribute to maintaining a high level of household and community sanitation.

INTERMEDIATE SERVICE LEVELS IN SANITATION SYSTEMS

by

John M. Kalbermatten and DeAnne S. Julius 1/

ABSTRACT

The major alternatives to sewerage are described and their potential for application in developing countries is explored. The reasons why conventional engineering practices have led to the selection of inappropriate technologies are examined. A least-cost comparison is made between sewerage and staged sanitation schemes, and recommendations for improved sanitation planning are presented.

INTRODUCTION

Sanitation is defined as the promotion of hygiene and prevention of disease by maintenance of sanitary conditions. For purposes of this discussion we limit sanitation to the adequate disposal of human waste in less developed countries (LDCs), although some brief references are made to water supply and sullage disposal insofar as they affect sanitary removal of human wastes.

It is clear that appropriate waste disposal by itself is not sufficient to provide adequate sanitation. Sufficient quantities of safe water are essential for human health, and other inputs such as medical care and health education are often required. The provision of water or waste

---

1/ Water and Wastes Adviser and Project Economist, respectively, The World Bank, Washington, D.C. 20433. The views expressed in this paper are those of the authors and should not be attributed to the World Bank or any of its affiliates.

disposal facilities to users who lack a clear understanding or knowledge of the importance of personal hygiene is, at best, partially effective and, in the worst case, useless. The emphasis of this paper is on human waste disposal, simply because that field has received insufficient attention in the past, and ideas and solutions have been stereotyped by experience in industrial countries which have little relevance to the needs and constraints of LDCs.

In the industrialized western countries, the standard solution for the sanitary disposal of human excreta is waterborne sewerage. The flush toilet is regarded as the ultimate and essential ingredient to an adequate solution to our waste disposal problems. Little thought is given to the fact that this method is designed not to maximize health benefits but to provide user convenience and environmental protection; two very important objectives in developed countries but with limited constituencies in LDCs. In fact, the flush toilet and associated sewer system is the result of slow progress over decades, even centuries. The cost of achieving the present standard of convenience is substantial.

The problem of LDCs is one familiar to most of us: high expectations coupled with limited resources. The decision-making elite would like to achieve the standards of convenience observed in industrialized countries. However, given the backlog in service and the massive size of sewerage investments, they do not have the funds to realize that goal. Sewerage could be provided for a few, but at the expense of the vast majority of their populations. Therefore, an investigation of other solutions to satisfy the health requirements of human waste disposal at a lower cost.

is urgently required. Any such solution, though of primary importance to LDCs, could also benefit inhabitants of industrialized countries not yet "blessed" with waterborne sewerage or those who find the ever increasing cost of cleaning up surface water polluted by sewage too great a burden.

HISTORICAL DEVELOPMENT

In Deuteronomy 23:12,13 the Lord instructed the Israelites to keep their camps clean: "You must have a latrine outside the camp, and go out to this; and you must have a mattock among your equipment and with this mattock, when you go outside to ease yourself, you must dig a hole and cover your excrement." Since Deuteronomy contains some of the oldest writings of the Bible, we can assume that appropriate waste disposal was of concern wherever people congregated, even in antiquity. It is interesting to note that there is no reference in the Bible to the need for clean water.

The latrine was probably the earliest attempt to increase user convenience associated with waste disposal, although not necessarily to reduce the health hazard 1/. The latrine provides privacy not available in the field; it reduces or eliminates the need to travel long distances to find privacy. If properly designed and maintained, is a perfectly acceptable method of human waste disposal. The majority of the people in rural areas of LDCs today use it in one form or another, and many people from industrialized countries still remember it from their childhood. In fact, in many rural areas the latrine presents the most cost-effective solution for the safe disposal of human waste.

As the population in the cities increased and land became more densely populated there was less room for backyard latrines. In addition, the development of municipal water systems required the disposal of increasing amounts of

---

1/ When properly designed and fitted with a ventilation pipe the latrine can also fulfill stringent requirements for pathogen destruction. (Ref. 1) water. Latrines gave way to bucket cartage or public latrines with waste being collected and discharged into nearby water courses; sullage water was usually discharged to open drainage ditches or the street. Obviously, as more wastes were generated very unsanitary conditions resulted, leading eventually to water closets and the discharge from them to storm drains and nearby water courses. As population increased further and water consumption rose, treatment of the discharged waste had to be instituted in order to reduce the massive pollution of receiving waters which had arisen from indiscriminate discharge. This ultimately lead to the separate sanitary and storm water systems we know today. Now some professionals are beginning to consider treating storm water because even rain water receives enough pollution from roofs, streets and other paved surfaces to become a substantial source of contamination. Due to industrialization, there are demands for more and more sophisticated treatment processes to protect our water resources. 1/

It is clear that we have reached the present stage of sanitation technology by a process of devising a solution to a problem created by a previous solution which eliminated a previous problem. For example, the present concern about organic chlorine compounds in drinking water is a result of chlorination of waste water and industrial effluents in order to disinfect the discharge before it enters the receiving waters. The disinfection technology was the response to the problem of health hazards created by the discharge of effluents.

This cause and response relationship can be extended all the way back to the change from dry to waterborne waste disposal. Unfortunately, neither then nor at any time since was a thorough examination undertaken to determine

---

1/ For more detailed discussion of the history of waste disposal technologies see Ref. 2.

whether waterborne waste disposal was the best solution. This may be because its consequences were not adequately foreseen. However, it is entirely possible that at some stage in the future we will find that we took the wrong fork in the road where the waterborne system and the dry system separated. It is clear that every time a new technology has been developed in order to solve the problems of another technology it has been the least-cost solution in the engineering sense. However, had a full economic evaluation been undertaken which included indirect as well as direct costs and which properly valued inputs at their opportunity costs rather than their market prices, the result might have been quite different 1/. Given the massive sewerage investments which now exist in the industrialized countries, it is probably too late for any major change in direction unless a definite correlation between some of the modern illnesses and sophisticated waste disposal and water treatment practices can be established, a development which is entirely within the realm of possibilities.

On the other hand, LDCs have waste disposal problems whose solution, in a majority of cases, has not yet been pre-empted by past commitments. They do not have the time it took the West to progress from the latrine to the present system. They also do not have the funds to do in one step what industrialized countries had decades, even centuries, to accomplish. In short, not only have *the opportunity but the obligation exists to take another look at existing waste* disposal practices, an opportunity which is of vital importance to the people in developing countries. For if a less expensive method to solve the waste disposal problem cannot be found, many people will be condemned to live their lives in unsatisfactory sanitary conditions.

1/ Since the basic tools of economic evaluation of projects have only been developed in the last 30 years, of course, it is unfair to criticize this aspect of engineering decisions made before that time.

LESS DEVELOPED COUNTRIES

To understand the magnitude of the problem, it is only necessary to look at some of the data collected by the World Health Organization in preparation for the United Nations Water Conference which took place in Mar del Plata in the Spring of 1977. Those figures show that at the present time only 32% of the population in LDCs have adequate sanitation services; that is, about 630 million out of 1.7 billion people. Population growth will add another 700 million people in the 1980s. In other words, between now and 1990 nearly two billion people will have to be provided with some means of sanitation if the goals of the Drinking Water Decade; i.e., adequate water supply and sanitation for all people; are to be achieved. A similar number of people will require water supply by the same date. It is at least of some consolation that water supply technology is better understood, and interest in water supply is substantially greater than in sanitation.

One of the fundamental problems in any attempt to provide the necessary sanitation services is the cost involved. Very general estimates based on existing per capita costs indicate that up to $60 million would be required to provide water supply for everyone and anywhere from $300 to $600 billion would be needed for sanitation services.1/ Per capita investment cost for sewerage ranges from $150 to $650, which is totally beyond the ability of the beneficiary to pay. It should be remembered that some one billion of these unserved people have per capita incomes of less than US$200 per year, with more than half of them below US$100 per year.

In addition to the technical task of developing or adapting lower cost technologies, the social and cultural aspects of waste disposal must be considered. Often there are strong social and religious taboos about

1/ The lower figure assumes technologies other than sewerage are used.

particular methods of waste disposal and personal hygiene which may preclude certain solutions. At the very least, education to enhance people's understanding of the value and the methods of waste disposal is necessary. In order to have the desired health impacts, sanitation technologies must not conflict with the natural preferences of the intended beneficiaries. For example, where water is a religious requirement for anal cleansing there is no sense in providing dry pit latrines. In some areas the feeling of being outdoors is desired; in other areas, privacy is of utmost importance. The construction of the privy or toilet enclosure will have to reflect these preferences.

The first question to be answered in evaluating sanitation technology for developing country application is whether feasible alternatives other than sewerage exist. Clearly, resources to serve all of the people of the developing world with sewer systems are not now available and probably will not be generated in the foreseeable future, as governments have other investment priorities. A look at alternatives in an attempt to improve the acceptability and the performance of some traditional but frequently abandoned technologies is therefore relevant.

ALTERNATIVES

On-Site Disposal

The latrine and its various modifications are probably the most widely used excreta disposal system in developing countries, especially in rural and semi-rural areas. They can be constructed by the user with very little outside help and few purchased materials. They are usually the least-cost method for the disposal of human waste.

In its *simplest form* the latrine is merely a hole in the ground into which excreta falls directly. It has been modified to improve convenience and eliminate some of the shortcomings of the open pit. One example of such an improvement is the pour flush squat plate or bowl, which not only increases user convenience but also prevents access by flies and insects and eliminates odors. Where the dry latrine is used, the design has been improved by including vent pipes which eliminate odor and substantially reduce fly breeding. Another improvement is to offset the seat or slab from the pit which permits the eventual removal of pit contents without disturbing the superstructure. The superstructure can be built to reflect the preferences of the owner and his ability to finance a simple or a more elaborate housing.

In rural areas it is the practice to abandon the latrine once the pit is about two thirds full, dig another one, place the existing superstructure on the new pit or build a new superstructure. In more densely populated areas where room for this multiple pit digging is not available, an offset pit latrine can be built. However, whenever this type of latrine is employed (often with pour flush squat plates or bowls) a community organization is required to empty the pit at intervals frequent enough to prevent filling up and possible spilling of the pit contents. Although no single design can be used universally, such latrines are adaptable to various conditions of environment, soil, and groundwater, by incorporating appropriate design modifications.

Composting toilets differ from the latrine in that they actively treat the excreta (i.e., kill pathogenic organisms) within the unit. Their operation requires considerable care because the composting process is sensitive to the amount of carbonaceous matter, such as kitchen wastes or grass cuttings, added. The process is also sensitive to moisture levels and thus water cannot be used for flushing. Composting toilets can be either continuous or batch process types; an example of the former is the well-known Swedish Clivus - Maltrum. The best known of the latter type is the Vietnamese double-vault latrine.

In the double-vault latrine one of the vaults is in use while the waste material in the other (which is sealed) undergoes composting. After a period of one year or so, the sealed vault is emptied while the first vault is sealed to allow the material to compost. The batch type latrines are more appropriate for use in LDCs because they are simpler to operate than the continuous type. The latter requires careful control of waste composition and frequent removal of the composted material in order to keep the process going. In addition, because the length of the composting period determines disease vector die-off, process control for the batch type composting latrine is less important than for the continuous process composting latrine with its shorter residence time.

Aquaprivies and flush toilets with septic tanks are another on-site disposal method. The aquaprivy is a vault on which either the pour or cistern type squat plate or bowl is placed with the water in the tank forming the water seal. The septic tank consists of a tank anywhere on the lot connected to a cistern flush or squat plate toilet with inverted siphon seal. As the description indicates, the aquaprivy can function with the very small amount of water needed to maintain the water seal. If the water consumption is elevated, as in the use of a cistern flush appliance, then the aquaprivy can be equipped with an overflow pipe to a soakage pit or drain field similar to the ones used to dispose of septic tank effluent. While the pit privy and the composting toilets can be adapted for use in almost any environment with or without water supply facilities, aquaprivies and flush toilets with septic tanks require water and depend on facilities to dispose of excess water. However, they do represent the increased convenience of a waterborne system without requiring the massive investments of off-site sewerage. They also represent improved insect and odor control; but in contrast to the pit privy and its various modifications, they require regular desludging, i.e., an institution to collect and dispose of the sludge.

Proprietary toilets are mentioned only for completeness as there seems to be little scope for their application to benefit the poorer population in LDCs. They might be of some use, however, in areas where public sewers are not available and home owners can afford to make the necessary investments to have the amenity of a more sophisticated system. Examples of proprietary toilets are recirculating toilets based on an oil-flush system with a separation of excreta and oil in an on-site separating unit and subsequent recirculating of oil to the system, and a toilet based on the destruction of feacal matter by the use of an electric burner or heating element.

Off-Site Disposal

The cartage system, which consists either of a bucket or vault latrine with collection at regular, short intervals and disposal by dumping or treatment, is in wide use in LDCs. The former is probably the oldest off-site disposal system known and is still used where the ability to maintain vacuum trucks and vehicles needed for the emptying of vaults is not available. There is no question that the bucket system is the least sanitary of the two cartage systems, and there is little possibility of improving the handling of buckets sufficiently to make this a satisfactory long term solution.

On the other hand, emptying of vaults by means of vacuum trucks is a satisfactory method, and possibly the least-cost off-site method of waste disposal for the near and medium term as long as local competence in maintaining and operating the necessary mobile equipment can be developed. The mobile equipment need not be sophisticated. A hand operated pump and donkey drawn wagon can be used as an intermediate step towards a vacuum truck.

Off-site disposal requires treatment of the disposed material to prevent public health hazards and pollution of the environment. The most commonly used method of treating nightoil is anaerobic digestion either in a conventional sewage treatment plant (where nightsoil collection exists in parallel with a sewer system) or by separate anaerobic digestors designed for nightsoil treatment. Digestors can be designed to recover the methane gas produced in the digestion process if the sale of this gas would contribute towards the cost of operation of the nightsoil collection and disposal system. Another promising treatment method is composting of nightsoil.

Pour flush latrines with small bore sewers combine the advantage of the pour flush latrine - the waterborne system with little water consumption - and the convenience of disposing of human waste through a sewerage system. Pour flush latrines with smallbore sewers represent an upgrading of the simple waterseal latrine with a soakaway. The addition of sewers usually is required when water consumption reaches a level which no longer permits the disposal of effluent through soakaways. Because no solids are discharged from the latrines to the sewers, the pipes can be much smaller and are therefore less costly

Because no solids are carried, the number of manholes can be reduced and the maintenance of grades is less critical. In general, the operation and maintenance of the small bore system is simpler than that of solids-carrying sewer system. On the other hand, latrine desludging program is necessary to avoid clogging problems if solids overflow into the sewers. Treatment of the effluent carried by the small sewer system is usually by stabilization ponds.

Communal toilets are an attempt to provide amenities to the low income population without constructing a large sewer system and individual house connections. Proper selection of communal toilet sites and the construction of a system to carry waste away from them can provide sanitation facilities for a large number of people. Unfortunately, in many societies communal facilities are not socially acceptable. Success with this type of waste disposal thus has been relatively rare. Nevertheless, where there are no low-cost alternatives due to environmental conditions, communal facilities combined with health education to overcome user resistance can constitute an important intermediate step in the development of a full sewerage system.

THE SELECTION PROCESS

Given the range of alternative waste disposal technologies available, the problem becomes one of selection. Evidently the method of selection which will yield the "right" solution is not an obvious one, however, since very few (if any) technologies other than conventional sewerage have been "selected" over the past 20 years. Yet the number of sewerage master plans gathering dust on shelves in citeis in LDCs with desperate waste disposal problems indicates that sewerage is not always appropriate. Even in a few of those cities where sewerage systems have been built, the number of house connections lags far behind the projected demand, and as a result both technical and financial problems abound. Apparently, the analysis which showed sewerage to be the least-cost solution omitted several important parameters.

In our experience there are four factors which, singly or in combination, account for the bias in conventional feasibility studies toward sewerage. The first, and probably most widespread, is that alternatives other than sewerage were not included in the investigation. Often this is not the fault of the engineering firm, but can be traced to the drafters of the terms of reference which specify that a sewerage system shall be designed for the city in question. The bias exists in the minds of planning officials and aid agencies, perhaps understandably since they are not expected to have a wide technical knowledge of the field. Due in part to the current fashion of searching for "appropriate technology" 1/ in the fields of agriculture and industry in LDCs, this bias is rapidly disappearing in the international aid community. Sewerage feasibility studies recently commissioned for two of the largest cities in Southeast Asia have included in the terms of reference the development of sanitation programs for those portions of the population who cannot afford to pay the full cost of sewerage.

A second factor which has led to the selection of wrong alternatives is that many least-cost analyses are based on financial rather than economic criteria. Thus they select the alternative which will be the cheapest (in present value terms) for the utility given prevailing interest rates and foreign exchange provisions, and often ignoring those costs borne by others, including the householder. An economic comparison would include all costs

---

1/ The publication of E.F. Schumaker's book, Small is Beautiful, in 1973 was, perhaps, most responsible for promoting this idea and giving it widespread visibility.

necessary for the system's proper functioning, and would value all inputs at their opportunity cost to the economy rather than their financial cost to the utility. Thus, for example, the fact that sewerage systems require 20-40 liters of flushing water per person per day would be reflected by including the long-run cost of producing that water (including capacity expansion costs, properly discounted) in the cost of the sewerage system. The fact that sewerage systems are generally subject to economies of scale and therefore are not fully utilized until five to ten years after completion would be reflected by calculating the per capita cost of sewerage not on the basis of the design population but on the basis of the present value of the population actually served over time. Just as costs incurred in future years are less expensive than those incurred today, so benefits received in future years are less valuable.

The reasons that sewerage benefits from financial rather than economic costing are that it is relatively capital intensive (and financial interest rates are generally below the opportunity cost of capital), it is relatively import intensive (and foreign exchange is often officially undervalued), it has a very high cost to the householder in terms of the plumbing and internal facilities needed, it has relatively high water requirements (and even where this is included in an analysis water is almost always priced below its long-run production cost), and it possesses larger economies of scale than most non-conventional waste disposal systems. With more and more consulting firms incorporating economic analysis (in fact, rather than in name only) into their feasibility studies, one can hope these factors will be more fairly reflected in the future, and alternatives which are truly least-cost will be selected.

A third problem is the failure to incorporate social factors into the design and selection process. When working in a familiar and homogenous social environment such as the United States this is done almost automatically since the engineer himself is generally a part of that social fabric. However, in developing countries it is necessary to make a real effort to discover the users' current practices and preferences in order to satisfy them at the least cost. Habits and ideas regarding human waste disposal are highly variable across cultures and are not easily discerned by the casual visitor. There are many examples where cultural misunderstandings have led to non-use or misuse of new technologies, not only for waste disposal but also for agricultural innovations, birth control campaigns, etc. The social dimension of technology design should not be regarded as an appendage to the technical and economic analyses, but as an integral part. Factors such as the color or location of a latrine may have little technical import and yet be crucial to the acceptance and use of the facility. In one new African community, the engineer designed the bathroom to be in the front of the houses so that the connection to the sewer would be as short as possible. However, the people were unwilling to change their traditional practice of having the latrine in the back of the house, away from the view of passersby. Had this been discovered before the sewers were laid, they could have been placed between the adjoining backyards of the houses for little additional cost. However, the users were not consulted during the design process and by the time they discovered the plans and complained, the sewers were already in place, and the house connections had to be modified at extra expense. Thus the system which eventually was built was not the least-cost solution.

A final factor which creates a bias toward sewerage is the method of tying consultant fees to the cost of the system designed, for example, through percentage of construction cost payments. It probably takes much more engineering effort and ingenuity to design non-conventional solutions than to use the company's computer programs to optimize sewerage options. Yet, because non-conventional alternatives are much cheaper than sewerage, the engineer would get paid less. This is clearly an inequitable situation and one that is fortunately being changed as more and more countries are turning to contracts where fees reflect the engineers' actual costs.

The problems with the technology selection process which have created a bias in favor of sewerage are gradually being overcome. In addition, the increasing interest in appropriate technology in other fields has stimulated related work in sanitation. A technical bibliography has just been published to review the state-of-the-art in low-cost sanitation (Ref.3). An important new work on the health aspects of excreta and sullage management with special reference to non-conventional options is now under review and should bepublished next year (Ref.1). One of the interesting conclusions of that study is that a properly designed and located pit latrine is just as effective (and sometimes more so) in pathogen destruction as a sewerage system. In fact, most of the nonconventional options can be designed to provide the potential for full health benefits. As with sewerage, the realization of those benefits will hinge upon proper user education and maintenance.

THE DYNAMIC SOLUTION

Incremental Improvements

Asking people to forego the possibility of having the convenience of a sewer system, even if they do not expect to have one until far in the future, is clearly not realistic. Not providing for a reasonable degree of sanitation immediately is also unacceptable if we are serious about people's health needs and improvement in their living conditions. A solution must be found which eliminates this"either/or" proposition. Fortunately, a variety of such solutions already exist.

If we examine how waterborne sewerage came about, two facts stand out. First, waste disposal went through many stages before sewerage. Second, existing systems were improved and new solutions invented whenever the old solution was no longer satisfactory. Whether or not waterborne sewerage is the best solution for human waste disposal problems is for the purposes of this discussion, irrelevant. What is important to remember is that sewerage was not a grand design implemented in one giant step but the end result of progressively more and more sophisticated solutions. Surely what took industrialized countries over a hundred years to achive in a close matching of needs and the economic capacity to take care of them cannot be expected of LDCs with limited resources in a short time. With the benefit of hindsight it should be possible to correct not only some of the shortcomings of more primitive waste disposal practices, as discussed in earlier paragraphs, but to develop a sanitation system which can be improved to reflect user requirements and the economic capacity to pay for improvements.

Staged sanitation systems should reflect not only the capacity of users to afford the facilities provided, but also their cultural environment and technical competence. Clearly, if sanitation facilities are to be used, consumer preferences and the customs of personal hygiene must be considered. In fact, staged sanitation might be more successful than the installation of sewerage since it can give the user a chance to progress as he sees fit, to whatever level of convenience he desires, and at his own speed. There is also no need for a commitment to reach a given stage at a given time.

A staged system can be chosen which reflects the user's growing level of technical experience as well as his cultural preferences. Construction of some sanitation facilities can be very simple and easily mastered by a homeowner. Operation and maintenance of on-site facilities may also be very simple; and off-site facilities, when they are needed, can be designed for operators with minimal technical expertise.

Sample Staged Solutions

To demonstrate the feasibility of using a staged sanitation system, three possible schemes are described, and costs are calculated for one of them and compared to those of sewerage. Each scheme could be started at any stage or terminated at any stage, depending on the desires of the users. For simplicity it is assumed that each stage would remain in service for ten years, after which either the next stage would be added or the existing facility would be replaced or repaired. The schemes described could be varied substantially without adding greatly to the cost. For example, to a standard pit privy with a pour flush a vault could be added if housing density increases or soil becomes clogged. Similarly, a composting toilet which already has a water tight vault, could be converted into an aquaprivy or pour flush privy with a vault.

I. The Waterless Latrine Scheme. The initial installation would consist of an offset pit or vault latrine with the vault extending outside the latrine housing to permit easy emptying. Emptying would be required every five years. This stage would last until the community water supply was upgraded from communal standpipes or wells to yard hydrants. With increased water availabili the dry latirne would be converted to a pour flush latrine by adding a squat plate or bowl with inverted siphon or aquaprivy waterseal. A baffle and overflow pipe would also be added to the vault to carry the overflow liquid to a soakage pit or drain field. Annual collection of accumulated sludge would be required along with a facility to compost or digest it. The third stage would begin when the water supply service is upgraded to house connections and a large quantity of sullage water has to be disposed of. At this point a small diameter sewer system would be constructed to accept the overflow from the vaults (replacing the drain fields). This solution would permit the use of cistern flush toilets to replace the bucket flush if desired. Annual collection of sludge would still be required.

II. The Pour Flush Latrine Scheme. The initial installation would be a pour flush latrine with a vault emptied by vacuum truck at one month intervals. The collected nightsoil would be composted, digested, or treated in stabilization ponds. As the water supply was upgraded this scheme could follow the same second and third stages as Scheme I.

III. The Cistern Flush Scheme. This scheme is essentially for those few users in an urban poor area who already have water connections in their houses. It begins at the second stage of Schemes I and II but with a flush toilet rather than a hand flushed bowl ro squat plate. The eventual installation of small bore sewers would depend on water usage and population density.

All of those schemes require offsite facilities in stage three such as ponds for the treatment of effluent and digestors for sludge treatment. Figure I shows a diagramatic presentation of the various components and their possible combinations into schemes.

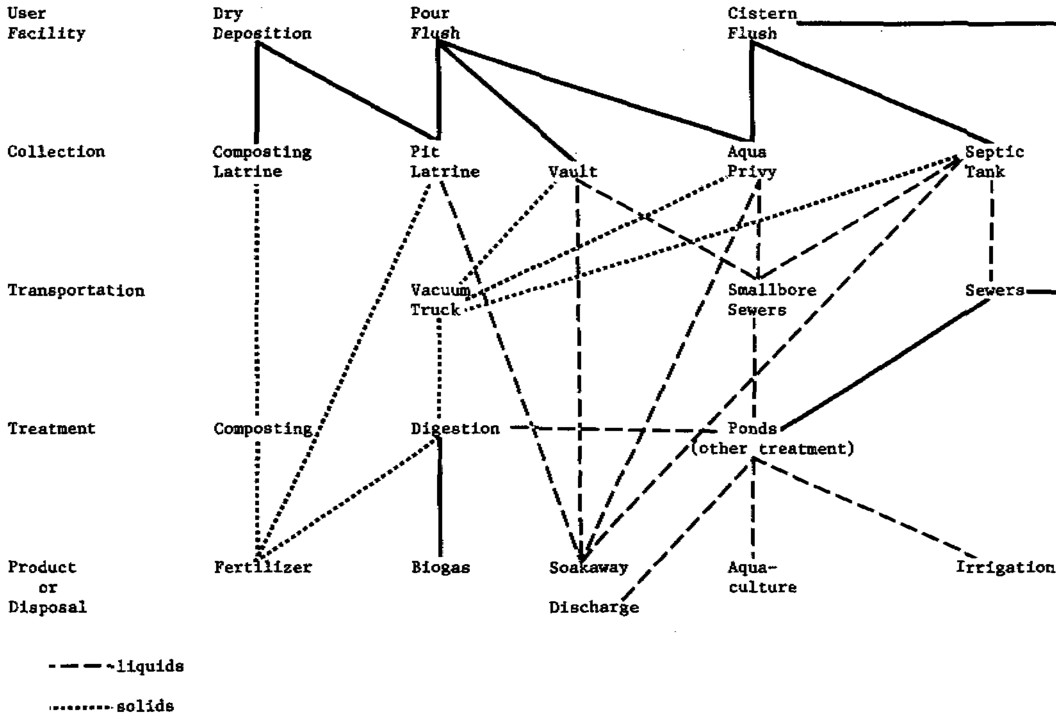


Fig. 1 Alternative technologies for excreta management.

Comparative economic costs, on a household basis, have been prepared for Scheme I and three variations including the alternative of proceeding immediately with the construction of a sewerage system. The costs are derived from those of existing African offset pit latrines, aquaprivies, sewered aquaprivies, and sewerage systems. They include all construction and maintenance costs of on-site, collection, and treatment facilities. They are economic rather then financial costs in that they include costs borne by all parties (not just the utility), and the value of inputs such as water and capital has been set at reasonable opportunity costs rather than at typical market prices[1/]. In addition, the per household cost of sewerage is calculated on the (discounted) population served over time rather than on the design population to reflect its gradual utilization.

Scheme I is costed on the basis of an offset pit latrine installed in year 1, upgraded to an aquaprivy with drain field in year 11, and then connected to small bore sewers in year 21. Sludge removal and composting occurs annually after year 11. Sewage treatment after year 21 is accomplished through two trickling filter plants. The annual cost per household of this three-stage system over 30 years is &72.4[2/].

The second alternative is a two-stage scheme which moves directly from the offset pit latrine (installed in year 1) to small bore sewers in year 11. The annual cost per household over 30 years is $133.5, or about 85% more than the three-stage alternative.

The third alternative is simply to install a small bore sewerage sytem from year 1. This would cost $160.9 per household per year over 30 years.

1/ Water is valued at $0.35/m^3$ and the opportunity cost of capital is taken to be 10%.

2/ This figure includes the "salvage value" of the sewerage system which is assumed to have a 40-year life.

The final alternative, calculated in the same way and with data from the same city as the sewered aquaprivy for purposes of comparison, is the immediate construction of a full sewerage system. The system was designed to serve about 190,000 people in an area of 3,500 hectares. A five-year construction period is assumed. The facility is assumed to be two-thirds utilized upon completion and fully utilized 10 years after completion. Based on these assumptions the annual cost per household over 30 years is $318.0. This includes the cost of flushing water and all regular operating and maintenance costs. It is four times as high as the cost of the three-stage scheme and nearly double that of the one stage sewered aquaprivy alternative.

All calculations utilized conservative assumptions in the sense of choosing a relatively inexpensive sewerage system as the basis for sewerage costs and relatively expensive pit latrines and aquaprivies as the basis for on-site costs. However, they were prepared for illustrative purposes only and should not be taken to represent costs which would be duplicated in an engineering simulation of the various alternatives on a particular site. They do indicate that considerable savings can be achieved through a staged upgrading scheme.

RECOMMENDATIONS

The single most important activity required for a more rational decision making process and subsequent achievement of appropriate solutions for the human excreta disposal problem is the dissemination of information on alternative waste disposal methods and the education of decision makers and designers on how to prepare and implement such projects. Only if the decision makers responsible for providing waste disposal services are alert to the possibility of using methods other than waterborne sewerage, understand the advantages and disadvantages of the various solutions and know the financial and economic costs of the various alternatives, can they make a rational decision on how to allocate a country's scarce fiscal resources.

Governments and development agencies must insist that designing engineers prepare master plans for sanitation rather than sewerage. Master plans should provide intermediate solutions for those areas which are not to be sewered so that all inhabitants of the area obtain excreta disposal services. A master plan should foresee the gradual improvements of services to whatever level the community desires as ability to pay for a higher level of service increases.

The preparation of such sanitation master plans and projects requires both a greater sensitivity by the designer to the needs of the community and a much more direct participation by the community in the design process. It is essential that service level options, their associated cost, and operational requirements be explained to the prospective users so they can select the system which most adequately serves their needs.

Designers have to be paid for undertaking these tasks, rather than by a percentage of construction cost which will be less for sanitation system than for sewerage. Further, the fee will also have to provide for the participation in the design process of sociologists, health specialists, etc., without whose input sanitation projects are unlikely to be successful.

Finally, research work must continue on those aspects of waste disposal not yet fully understood and evaluated, for example, the impact of sullage water disposal on the environment. Another area requiring attention is the development of appliances which provide the amenity of water use without high water consumption. Success in this area would permit the use of such appliances in areas of low-density development without the need to construct waterborne sewerage. Another area requiring additional work is the reuse of excreta, solid wastes, and agricultural wastes. This research topic has the potential of creating a whole new industry which could substantially lower a community's waste disposal costs and produce valuable products ranging from energy to food to pharmaceuticals.

REFERENCES

1. Feachem, R.G., D.J. Bradley, H. Garelick and D.D. Mara, Health Aspects of Excreta and Sullage Management, Draft for review, World Bank, May 1978.

2. Stanbridge, H.H., History of Sewage Treatment in Britain, The Institute of Water Pollution Control, Maidstone, U.K., 1976.

3. Rybczynski, W., C. Polprasert and M.G. McGarry, Sanitation Technology Options, International Development Research Centre, Ottawa, Canada, 1978.

ENVIRONMENTAL EPIDEMIOLOGY AND SANITATION

by

DAVID J. BRADLEY[1] AND RICHARD G. FEACHEM[2]

INTRODUCTION

In considering improved excreta disposal technologies the engineer, administrator and community development worker cannot consider each disease separately. Rather, they require a conceptual framework which links various types of excreta-related infections to the design and implementation of particular disposal or reuse technologies. A biological classification, which groups the viruses, bacteria, protozoa and worms together, may be less helpful in understanding the health aspects of alternative approaches to excreta disposal, than a classification of infections which is based upon their transmission routes and life cycles. Such a classification we call an environmental classification. In fact, the resemblance between a biological and an environmental classification is much closer in the case of the excreta-related infections than in the case of the diseases related to water.

The purpose of an environmental classification is to group infections in such a way that the role of different preventive measures, and the efficacy of different environmental and behavioural modifications, are made clear. The object here is to propose an environmental classification of the infections related to excreta. In devising such a classification we have encountered two major limitations. The first is, remarkably, how little is precisely known about the transmission of several infections

1,2 Professor and Senior Lecturer, Ross Institute of Tropical Hygiene, London School of Hygiene and Tropical Medicine.

and the numbers of microbes needed to pass the infections on to susceptible people. The second is that the bulk of the excreted viruses, bacteria and protozoa, differ quantitatively rather than qualitatively in their transmission characteristics and it is easy to finish up with a big category containing the majority of infections. Understanding of these infections depends on some basic parameters of transmission, especially latency and persistence in the environment, and the infective dose for man. We therefore discuss these other key concepts before setting out the classification.

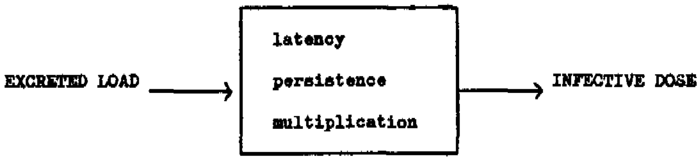
KEY CONCEPTS IN UNDERSTANDING EXCRETA-RELATED INFECTIONS

Excreta may be related to human disease in two ways (Figure 1). The agents of many important infections, escape from the body in the excreta and thence eventually reach others. These are the excreted infections. In some cases the reservoir of infection is almost entirely in animals other than man. These are not dealt with here because such infections cannot be controlled through changes in human excreta disposal practices. However we do include a number of infections for which both man and other animals serve as a reservoir.

The second way in which excreta relate to human disease is where their disposal encourages the breeding of insects. These insects may be a nuisance in themselves (flies, cockroaches, mosquitoes); they may mechanically transmit excreted pathogens either on their bodies or in their intestinal tracts (cockroaches and flies), or they may be vectors for pathogens which circulate in the blood (mosquitoes). Where flies or cockroaches are acting as vehicles for the transmission of excreted pathogens, this represents a particular case of the many ways in which excreted pathogens may pass from anus to mouth. However, where mosquitoes are transmitting non-excreted pathogens the concepts discussed in this paper have little relevance and the important factors are those which determine the breeding habits of the particular mosquitoes.

In considering the transmission of excreted infections, the distinction between the state of being infected, and the state of being diseased, must be kept in mind. Very often, the most impotant section of the population involved in transmitting an infection shows little or no sign of disease; conversely, individuals with advanced states of disease may be of little or no importance in transmission. A good example occurs in schistosomiasis, where as much as 80% of the total egg output in faeces and urine reaching water from a human population may be produced by children in the 5-15 years age group; many of these children will show minimal signs of disease. Conversely, middle-aged people with terminal disease conditions may produce few or no hatchable eggs.

If an excreted infection is to spread, an infective dose of the relevant agent has to pass from the excreta of a case, carrier, or reservoir of infection to the mouth of a susceptible person or some other portal of entry. Spread will depend upon the number of pathogens excreted, upon how these numbers change during the particular transmission route or life cycle and upon the dose required to infect a new individual. Infectious dose is in turn related to the susceptibility of the new host. Three key factors govern the probability that, for a given transmission route, the excreted pathogens from one host will form an infectious dose for another. These are latency, persistence and multiplication. Diagrammatically we can represent the concepts thus:



We will discuss these concepts in turn.

<u>Excreted load</u>. There is wide variation in the concentration of pathogens passed by an infected person. For instance, a person infected by a small number of nematode worms may be passing a few eggs per gram of faeces whereas a cholera carrier may be excreting more than 10^6 *Vibrio* per gram, and a case may pass 10^{13} vibrios in a day.

Where large numbers of organisms are being passed in the faeces they can give rise to high concentrations in sewage (Table 1). Thus, even in England, where water use is relatively high and salmonellosis relatively rare, raw sewage may contain 10^4 *Salmonella* per litre. At these concentrations, removal efficiencies of 99% in treatment works will still leave 10^2 pathogenic organisms per litre in the effluent, and their implications for health will depend upon the disposal method, their ability to survive or multiply and the infectious dose required.

<u>Latency</u>. By latency we mean the interval between the excretion of a pathogen and its becoming infective to a new host. Some organisms, including all excreted viruses, bacteria and protozoa have no latent period and are immediately infectious when the excreta are passed. The requirements for the safe disposal of excreta containing these agents

TABLE I POSSIBLE OUTPUT OF SOME PATHOGENS IN THE FAECES AND SEWAGE OF A TROPICAL COMMUNITY (a)

PATHOGEN	Typical Prevalence of infection in developing country (b)	Typical Average number of organisms per gram of faeces (c)	Total Number Excreted per infected person per day (d)	Total Number Excreted per day in town of 50,000 pop.	Concentration per litre (c) in sewage from town of 50,000 (assuming 100 litres per capita per day of sewage produced and that 90% of excreted pathogens do not enter the sewers or are inactivated in the first few minutes).
Enteroviruses	5%	10^6	10^8	$2.5 x 10^{11}$	5000
Salmonellae	7%	10^6	10^8	$3.5 x 10^{11}$	7000
Shigellae	7%	10^6	10^8	3.5x10	7000
Vibrio cholerae	1%	10^6	10^8	$5 x 10^{10}$	1000
Path. E. coli	?	10^8	10^{10}	?	?
Entamoeba histolytica	30%	$15 x 10^4$	$15 x 10^6$	$2.25 x 10^{11}$	4500
Ascaris	60%	10000 (e)	10^6	$3 x 10^{10}$	600
Trichuris	60%	2000 (e)	$2 x 10^5$	$6 x 10^9$	120
Hookworms	40%	800 (e)	$8 x 10^4$	$1.6 x 10^9$	32
Schistosoma mansoni	25%	40 (e)	$4 x 10^3$	$5 x 10^7$	1
Taenia saginata	1%	10^4	10^6	$5 x 10^8$	10

NOTES:

(a) This table represents an entirely hypothetical situation and the figures are not taken from any real town. However, for each pathogen the figures are reasonable and in line with those found in the literature. The concentrations of each pathogen in sewage, derived in the table, are in line with the higher figures in the literature. However, it is unlikely that all these infections at these relatively high prevalences would occur in any one single community.

(b) The prevalence figures quoted in this column refer to infection and not to morbidity.

(c) It must be remembered that the pathogens listed have different abilities to survive outside the host and the concentration of some of them will rapidly decline after the faeces have been passed.

(d) To calculate this figure it is necessary to estimate a mean faecal weight for those people infected. This must necessarily be the roughest estimate because it depends on the age-specific faecal weights in the community and the age distribution of infected people. It was assumed that over-15 year olds excrete 150 g per day and that under-15's excrete, on average, 75 g per day. It was also assumed that two-thirds of all infected people are under-15's. This gives a mean faecal weight for infected individuals of 100 g.

(e) The distribution of egg output among people infected by these helminths is extremely skewed and some people are putting out very high egg concentrations.

are far more stringent than for those helminthic infections where there is a prolonged latent period. In particular, infections which have a considerable latent period are largely risk-free where nightsoil is being carted whereas the others constitute a major health hazard in fresh nightsoil. Therefore in our classification the first two categories where no latency is observed are separated from the remaining categories where a definite latent period occurs.

Among the helminthic infections only three have eggs or larvae which may be immediately infectious to man when passed in the faeces. These are Enterobius vermicularis, Hymenolepis nana, and sometimes Strongyloides stercoralis. The remaining excreted helminths all have a distinct latent period, either because the eggs must develop into an infectious stage in the physical environment outside the body, or because the parasite has one or more intermediate hosts through which it must pass to complete its life cycle.

Persistence or survival of the pathogen in the environment is a measure of how quickly it dies after it has been passed in the faeces. It is the single property most indicative of the faecal hazard in that a very persistent pathogen will create a risk throughout most treatment processes and during the reuse of excreta.

A pathogen which persists outside the body only for a very short time needs to find a new susceptible host rapidly. Hence transmission cannot follow a long route through sewage works and the final effluent disposal site back to man but will rather occur in the family by transfer from one member to another as a consequence of low personal cleanliness.

More persistent organisms can readily give rise to new cases of disease further afield, and as survival increases so also must concern for the ultimate disposal of the excreta. In addition, pathogens which tend to persist in the general environment will require more elaborate processes if they are to be inactivated in a sewage works. Methods of sequestering them, as by sedimentation into a sludge which receives special treatment, are often needed.

While it is easy to measure persistence or viability of pathogenic organisms by laboratory methods, to interpret such results it is necessary to know how many are being shed in the excreta (relatively easy to determine) and the infective doses for man (extremely difficult).

Multiplication. Under some conditions certain pathogens will multiply in the environment. Thus, originally low numbers can be multiplied to produce a potentially infective dose (see below). Multiplication can take the form of reproduction by bacteria in a favoured substrate (e.g. *Salmonella* on food) or of the multiplication by trematode worms in their molluscan intermediate hosts.

The former case is a mechanism whereby light faecal contamination may build up bacterial numbers to reach the rather high minimal infective doses needed by many excreted bacterial pathogens. The need for this may determine the usual mode of infection, since multiplication in water is limited compared with the massive increases possible in food. Viruses and excreted protozoa do not multiply outside their animal hosts.

Among the helminths transmitted by excreta, all the trematodes infecting man undergo multiplication in aquatic snails. This introduces a prolonged latent period of a month or more while development is taking place in the snail, followed by an output of up to several thousand larvae into the environment for each egg that reached a snail. Category V of the classification is used for infections of this sort where excreta have to gain access to the appropriate snail habitat, but once this happens great amplification is possible.

Infective dose. In a tidy world, from knowing the output of pathogens in the excreta of those infected, the mean infective dose, and the extractive efficiency of the excreta treatment process, it would be a matter of simple calculation to assess risk. The real world is much less predictable than this because of the variable infective dose of most pathogens and the uneven distribution of infection in the environment. While the minimal infective dose for some diseases may be a single organism, or very few, the doses required in most bacterial infections are much higher. Data bearing on this are very hard to acquire, since they involve administering a known dose of a pathogen to a volunteer. Information is scanty, concerned with doses required to infect say half those exposed, rather than a minute proportion, at a single exposure. The volunteers have been well-nourished adults and usually from non-endemic areas. Such results have therefore to be applied with great caution to malnourished peasant children continually exposed to infection. It has been found that changes in the manner of administration, such as preceding a dose of cholera vibrios with an alkaline substance to temporarily reduce free gastric acid, may lower the mean infective dose of such organisms by a factor of 10^3.

Also, in human experimental studies, the infective dose for half the people exposed is the most reliable result but in natural transmission the dose infective for 5% or less of the population may be more relevant.

The consequent uncertainties over the size of the minimal infective dose in nature makes allocation of diseases between the first two categories of our classification uncertain. These difficulties are greatest for the major excreted bacterial infections, and for protozoa. For viruses there is evidence of low infective doses in experiments, and in human populations for some but not all virus infections. Among the helminths a single egg or larva can infect if ingested, though a high proportion of worms can fail to develop to maturity, especially where immunity is present.

Host response. Host response is important in determining the result of an individual receiving a given dose of an infectious agent. In particular, acquired immunity, and the relation of age to pathology, are important for predicting the effects of sanitation. At one extreme would be a short-lived parasite to which little immunity developed and in which the relation between infection and disease was not age-dependent. Then a close, tending to linear, relationship between exposure and disease might be expected with improvements in the appropriate aspects of sanitation giving health benefits proportional to effort. *Ascaris* closely approximates to this model.

At the other extreme would be a viral or bacterial infection which gives rise to long-lasting immunity and where the chance of overt disease in those infected rose with increasing age. An example is infection with poliomyelitis virus. Under very bad sanitary conditions all are infected at a young age, older children and adults are immune and disease is limited to a few of the youngest children who may suffer chronic paralysis. If sanitation improves, infection is deferred and its pathological consequences later in life are more serious. Thus, although poliovirus transmission may be reduced by improving sanitation, reduction in disease is in practice achieved by immunization. Does this apply to any other excreted infection? Possibly to infectious hepatitis and it has been argued in the case of typhoid. However, there are probably several infections where human immunity is of importance in regulating the amount of disease. This will tend to reduce the health significance of moderate sanitary improvements, and may in part explain the disappointing impact of some sanitary programmes.

The balance between exposure to infection and host response to it will determine the pattern of excreta-related disease. If transmission, creating exposure to a particular infection, is low then most people will not have encountered the infection. They will be susceptible. If a sudden increase in transmission of the disease occurs, it will affect all age groups in epidemic form. Improvements in sanitation will have a big effect under these circumstances by reducing the likelihood and/or the magnitude of an epidemic.

By contrast, if transmission is very high all the people will be repeatedly exposed to infection and first acquire it in childhood. Subsequent exposures may be without effect if immunity is acquired from the first attack. Or immunity may be cumulative from a series of attacks. The infection will always be present and is described as

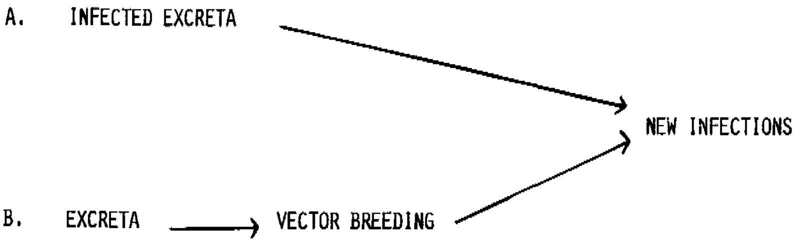


Figure 1: The two main ways in which excreta is related to ill-health. In A, the excreta itself contains the pathogens which may be transmitted by various routes to a new host. In B, the excreta or sewage permits the breeding of certain flies and mosquitoes which may act as vectors of both excreted, and other pathogens.

ZOONOSES

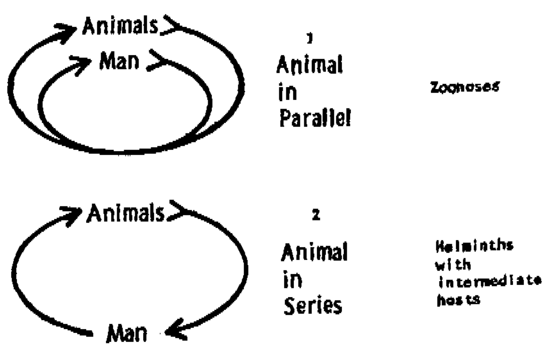


Figure 2: Two ways in which animals are involved in the tranmission of excreted infections.

endemic. Under these conditions much transmission is ineffective because of human acquired immunity, and reduced transmission, as through improved sanitation, will only delay the date of infection somewhat so that older children are seen infected. Very large sanitary improvements will either render the infection very rare or, if the disease was originally very highly transmitted, make it an adult disease. Examples are tyhoid, which by management of excreta and of water supplies can be completely prevented in the community, and poliomyelitis virus infection which requires extreme hygienic precautions to prevent, and in practice improved sanitation increases the disease problem by deferring infection to an age where the clinical course is more severe.

Consequences of a juvenile age-prevalence are that, not only do children suffer chiefly from the diseases, but also they are the main sources of infection, so that the acute need for better community excreta disposal is among young children, the group perhaps least inclined to use any facilities that may be available.

Other hosts besides man. Some excreted diseases are infections exclusively or almost exclusively of man. Others involve animals either as alternatives to man as host or as hosts of other stages in the life cycle (Figure 2). In the first case, where wild or domestic vertebrate animals act as alternative hosts (such infections are called zoonoses), control of human excreta is likely not to suffice for complete prevention of the infection, while if the infection under consideration is strictly anthroponotic (e.g. shigellosis) then it is the control of human excreta which is of importance. In the second case, some excreted helminthic infections have intermediate hosts. They will therefore be controlled if:

(a) excreta are prevented from reaching the intermediate host;

(b) the intermediate hosts are controlled;

(c) people do not eat the intermediate host uncooked or do not have contact with the water in which the intermediate host lives (depending on the particular life cycle).

These infections (animals in series, Figure 2) fall into Categories IV and V of the classification below. In Category IV the intermediate hosts are domestic vertebrate animals and control of either human excreta or the animal infection will suffice to prevent disease. By contrast, with the vertebrates 'in parallel' (Figure 2) it is necessary to control both human and animal excreta, or tackle the problem in some other way.

Some details on the factors discussed above are provided in Table 2, for the excreted infections being considered.

ENVIRONMENTAL CLASSIFICATION OF EXCRETA-RELATED INFECTIONS

There are many ways in which the excreted infections could be grouped on the basis of the information presented in Table 2. We have searched for a classification which is most relevant to the effect of excreta disposal *per se* and which is most helpful in considering the impact of changing excreta disposal facilities and technology. Table 3 presents this classification. We have distinguished six categories of infection.

There is a clear difference between the first five categories of excreted pathogens and the last which contains the excreta-breeding insect vectors of disease. A variety of sanitation methods will control the insects and there are additional specific measures that can be directed

TABLE II SOME BASIC FEATURES OF EXCRETED INFECTIONS (a)

CATEGORY (Table 3)	PATHOGEN	LATENCY Typical min. time from excretion to infectivity	PERSISTENCE Anticipated max. life of infective stage at 20-30°C	CONCENTRATION typical average number of organisms per gram of faeces	MULTIPLICATION Outside human host	MEDIAN INFECTIVE DOSE High > 10^6 Medium 10^4 Low < 10^2	SIGNIFICANT IMMUNITY	MAJOR RESERVOIR OTHER THAN MAN	INTERMEDIATE HOST
I	Enteroviruses	0	6 months	10^6	No	Low	Yes	No	none
	Hepatitis A virus	0	?	10^6 (?)	No	Low	Yes	No	none
	Rotaviruses	0	1 year (?)	10^6 (?)	No	Low	Yes (?)	No	none
	Entamoeba histolytica	0	20 days	$15x10^4$	No	Low	No	No	none
	Giardia lamblia	0	3 months	10^5	No	Low	No (?)	No	none
	Balantidium coli	0	1 month (?)	?	No	Low ?	No	Yes	none
	Enterobius	0	7 days	not usually found in faeces	No	Low	No	No	none
	Hymenolepis	0	a few weeks	?	No	Low	Yes (?)	No	none
II	Salmonella typhi	0	60 days	10^6	Yes (food)	High	Yes	No	none
	Other Salmonellae	0	1 year	10^6	Yes (food)	High	Irrelevant (e)	Yes	none
	Shigella	0	40 days	10^6	Yes (food)	Medium	No	No	none
	Vibrio Cholerae	0	30 days	10^6	unlikely	High	Limited	No	none
	Path. E. coli	0	1 year	10^8	Yes	High	Yes (?)	No	none
	Yersinia	0	6 months	10^5	?	High	No	Yes	none
	Campylobacter	0	?	?	?	?	?	?	none
III	Ascaris	9 days	several years	10^4	No	Low	No	No	none
	Trichuris	3 weeks	1½ years	10^3	No	Low	No	No	none
	Hookworms	7 days	20 weeks	$8x10^2$	No	Low	No	No	none
	Strongyloides	3 days	5 weeks (free living stage very much longer)	10	Yes	Low	Yes	No	none
IV	Taenia	8 weeks (b)	2 years	10^4	No	Low	No	No	cow/pig

TABLE II

CATEGORY (Table 3)	PATHOGEN	LATENCY Typical min. time from excretion to infectivity	PERSISTENCE Anticipated max. life of infective stage at 20-30°C	CONCENTRATION typical average number of organisms per gram of faeces	MULTIPLICATION Outside human host	MEDIAN INFECTIVE DOSE High > 10^6 Medium 10^4 Low < 10^2	SIGNIFICANT IMMUNITY	MAJOR RESERVOIR OTHER THAN MAN	INTERMEDAITE HOST
V (f)	Clonorchis	3 months (c)	life of fish	10^2	Yes (d)	Low	No	Yes	snail and fish
	Diphyllobothrium	4 weeks (c)	life of fish	10^4	No	Low	No	Yes	copepod and fish
	Fasciolopsis	10 weeks (b)	?	10^2	Yes (d)	Low	No	Yes	snail and aquatic plant
	Paragonimus	4 months (c)	life of crab	?	Yes (d)	Low	No	Yes	snail and crab or crayfish
	Schistosoma mansoni	4 weeks (b)	2 days	40	Yes (d)	Low	?	No	snail
	Schistosoma haematobium	5 weeks (b)	2 days	40/10 ml urine	Yes (d)	Low	Yes	No	snail
	Schistosoma japonicum	7 weeks (b)	2 days	40	Yes (d)	Low	Yes	Yes	snail

Notes: (a) Leptospirosis does not fit into any of the categories defined in Table 3

	Leptospira	0	7 days	? (urine)	No	Low	Yes (?)	Yes	none

(b) Life cycle involves one intermediate host. Latency is minimum time from excretion by man to potential reinfection of man. Persistence refers to maximum survival time of final infective stage.

(c) Life cycle involves two intermediate hosts. Latency is minimum time from excretion by man to potential reinfection of man. Persistence refers to maximum survival time of final infective stage.

(d) Multiplication takes place in intermediate snail host.

(e) The large number of serotypes (> 1000) makes immunity epidemiologically irrelevant.

(f) Fasciola, Gastrodiscoides, Heterophyes and Metagonimus are also located in Category V.

TABLE III

A CLASSIFICATION OF EXCRETED INFECTIONS

CATEGORY	FEATURES	INFECTIONS	DOMINANT TRANSMISSION FOCI	MAJOR CONTROL MEASURES
I	Non-latent, low infectious dose	Enterobiasis Enteric viruses Hymenolepiasis Amoebiasis Giardiasis Balantidiasis	Personal contamination Domestic contamination	Domestic water supply Health education Improved housing Provision of toilets
II	Non-latent medium or high infectious dose, moderately persistent and able to multiply	Typhoid Salmonellosis Shigellosis Cholera Path. *E. coli* Yersiniosis *Campylobacter*	Personal contamination Domestic contamination Water contamination Crop contamination	Domestic water supply Health education Improved housing Provision of toilets Treatment prior to discharge or reuse
III	Latent and persistent with no intermediate host	Ascariasis Trichuriasis Hookworm Strongyloidiasis	Yard contamination Field contamination Crop contamination	Provision of toilets Treatment prior to land application
IV	Latent and persistent with cow or pig intermediate host	Taeniasis	Yard contamination Field contamination Fodder contamination	Provision of toilets Treatment prior to land application Cooking, meat inspection
V	Latent and persistent with aquatic intermediate host(s)	Clonorchiasis Diphyllobothriasis Fascioliasis Fasciolopsiasis Gastrodiscoidiasis Heterophyiasis Metagonimiasis Paragonimiasis Schistosomiasis	Water contamination	Provision of toilets Treatment prior to discharge Control of animal reservoirs
VI	Excreta-related insect vectors	Bancroftian filariasis (transmitted by *Culex pipiens*), and all the infections listed in Categories I-V which may be transmitted by flies and cockroaches	Insects breed in various faecally contaminated sites	Identification and elimination of suitable breeding sites

against them.

The excreted infections are divided on the presence (Categories III-V) or absence (I and II) of a latent period so that health problems with fresh faeces or nightsoil are particularly in the first two categories. The distinction between Categories I and II on the one hand, and Categories III-V on the other, is fundamental and clear cut. It also corresponds closely to the biology of the pathogens, in that all infections in Categories III-V are helminthic.

The sub-divisions of the infections with latency (Categories III-V) are also clear cut, with Category III for the soil-transmitted worms, IV for the tapeworms which depend on access of faeces to stock, and V for the trematodes and other worms requiring aquatic intermediate hosts. However, the subdivision of Categories I and II is difficult, and somewhat arbitrary, because the various concepts discussed above split the infections in these categories in different ways. For instance, if we divide Categories I and II on the basis of median infectious dose, stressing as we do so the grave limitations of the available data on infectious dose, we arrive at the following approximate ranking:

increasing median infectious dose	Enteric viruses *Enterobius* *Hymenolepis* *Entamoeba histolytica* *Giardia lamblia* *Balantidium coli*	$< 10^2$

Shigella	10^4
Salmonella typhi Salmonellae *Yersinia* Path *E. coli* *Vibrio cholerae*	$> 10^6$

If, on the other hand, we list the infections according to their persistence outside their animal host, we arrive at approximately the following ranking:

increasing persistence ↓	*Enterobius* *Entamoeba histolytica* *Hymenolepis* *Balantidium coli* *Vibrio cholerae*	$<$ 1 month
	Shigella *Giardia lamblia* *Salmonella typhi* *Yersini a*	$<$ 6 months
	Enteric viruses Salmonellae Path *E. coli*	$<$ 1 year

Another important factor in predicting the impact of improved excreta disposal facilities may be whether or not there is a significant non-human reservoir of infection (Figure 2). Considering the Category I and II infections, there are only two (the salmonellae and *Balantidium coli*) which have a significant animal reservoir.

A quite different approach to the division of Categories I and II is to consider affluent communities in Europe (for instance), which enjoy high standards of sanitary facilities and hygiene, and examine which of the Category I and II infections are commonly transmitted in these privileged communities. We might expect that infections which continue to be transmitted amongst people living in good housing, with indoor plumbing and flush toilets, will not be readily reduced by the introduction of limited sanitary improvements amongst poor people in the less developed countries. A division on this basis is approximately as follows:

Enteric viruses	Pathogens commonly transmitted within affluent communities
Enterobius	
Giardia	
Path. *E. coli*	
Salmonellae	
Balantidium coli	Pathogens rarely transmitted within affluent communities in Europe
Entamoeba histolytica	
Hymenolepis	
Salmonella typhi	
Shigella (other than *sonnei*)	
Vibrio cholerae	
Yersinia	

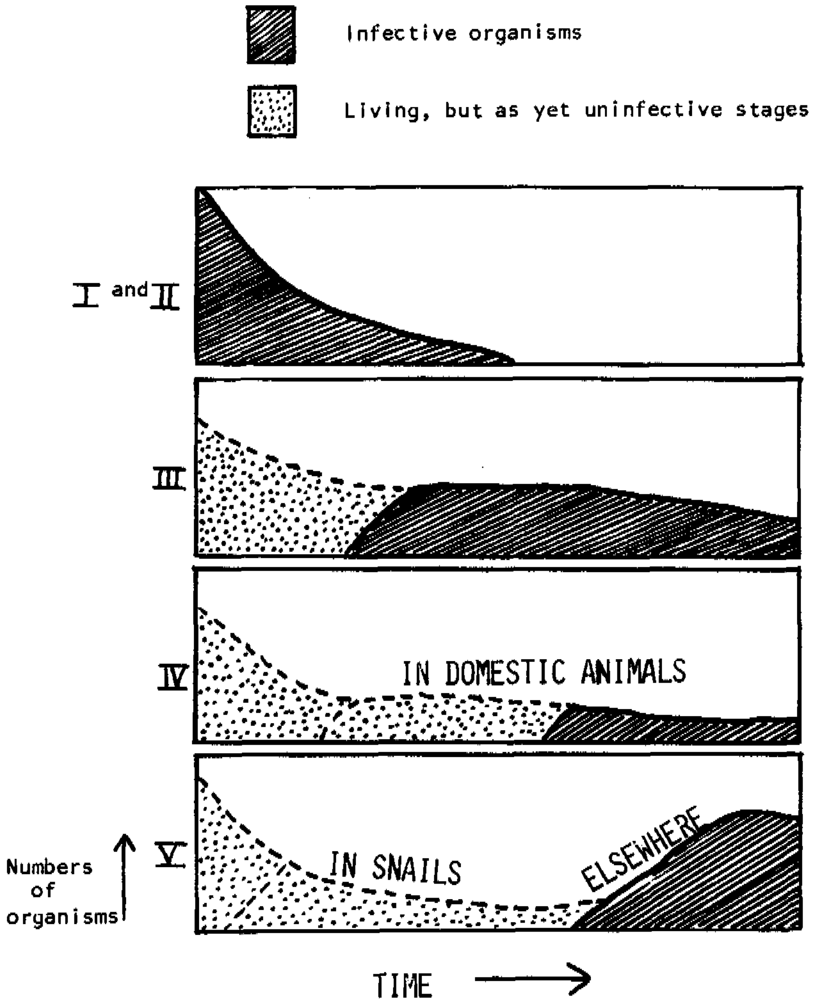


Figure 3: The survival of pathogens over time, outside their definitive hosts, for each category of infection. (see Table 3).

In some cases the reasons for this division are clear (for instance, the salmonellae continue to be transmitted from animals to man in affluent communities through contaminated foodstuffs) whereas in other cases (such as the continued success of Shigella sonnei in Europe) they are obscure.

We believe that, for the time being the most useful division of Categories I and II is on the basis of probable infectious dose, recognising again that our knowledge of infectious dose amongst malnourished peasant children in the tropics is non-existent. Infectious dose divides Categories I and II in a way that makes sense theoretically and also corresponds to some degree with the likely impact of improved excreta disposal facilities.

Each category in Table 3 implies some minimum sanitary requirements for control of the diseases, and often ancillary inputs in addition to excreta disposal facilities if success is to be achieved. The transmission characteristics of the first five categories are set out in Figure 3 which illustrates their typical survival, latency and multiplication features. These in turn affect the 'length' of transmission cycle involved. Length has implications beyond those of time, in that a long cycle is associated with opportunity to spread over a wider area and the pattern of risk changes. These issues are represented in Figure 4, which also summarises some of the conclusions we reach on the relative efficacy of sanitation improvements in controlling infections.

Category I. These are the infections which have a low infective dose ($<10^2$) and which are infective immediately on excretion. We argue that these infections may be spread very easily from person to person whenever personal and domestic hygiene are not ideal (Figure 4). Therefore, it is

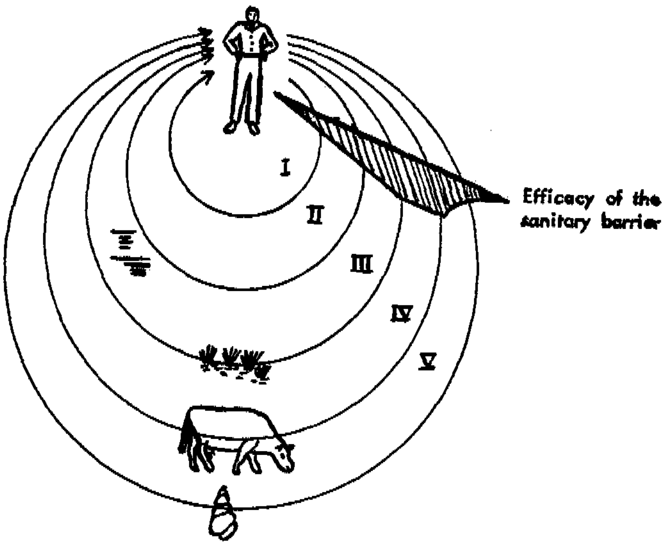


Figure 4: The 'length' and dispersion of the transmission cycles associated with the five categories of excreted infection (Table 3). The possible efficacy of improvement in excreta disposal is also indicated.

likely that changes in excreta disposal technology will have little effect on the incidence of these infections if they are unaccompanied by sweeping changes in hygiene which may well require major improvements in water supply and housing, as well as major efforts in health education. The important facet of excreta disposal is the provision of a hygienic toilet of any kind so that the people in a house have somewhere to deposit their excreta.

What subsequently happens to the excreta (i.e. how it is transported, treated and reused) is of less importance because most transmission will occur in the home. Although transmission can and does occur by more complex routes, we argue most transmission is direct person-to-person and therefore the provision of hygienic toilets alone will have a negligible impact. Having said this, we must at once qualify this category, for Categories I and II grade into each other and really form a continuum (see below). In particular, the parasitic protozoa have some features of each group. The extreme example of a Category I parasite is the pin-worm, _Enterobius_, whose sticky eggs are laid by eme rging females on the anal skin so that transmission is by way of scratching fingers without depending much on eggs in the faeces. At the other extreme, _Giardia_ has been associated with well-documented water-borne diarrhoea outbreaks, and therefore is presumably in part subject to control by excreta management.

Category II. The infections in this category are all bacterial. They have medium or high infective doses ($>10^4$) and so are less likely than Category I infections to be transmitted by direct person-to-person contact. They are persistent and can multiply, so that even the small numbers remaining a few weeks after excretion can, if they find a suitable substrate (such as food), multiply to form an infective dose. Person-to-person routes are important but so too are other routes with longer environmental cycles, such as the contamination of water sources or crops with faecal material (Figure 4). The control measures listed under Category I are important, namely water supply, housing, health education and the provision of hygienic latrines, but so too are waste treatment and reuse practice. Changes in excreta disposal and treatment practices alone may have some but little impact. This impact may be on those infections which, as we have noted above, are not normally transmitted among affluent groups in Europe or elsewhere. These are cholera, typhoid and shigellosis and any monitoring or evaluation programme would do well to examine these, rather than infections with other salmonellae or pathogenic _E. coli_.

Characteristics of Categories I and II

The criteria used to separate these categories have been infective dose and 'length' of the environmental cycle and our aim has been to predict efficacy of sanitation as a control measure. The reason they do not form tidy groups is the variable persistence of the pathogens involved. The extreme type I situation with a low infective dose and environmentally fragile organism will clearly tend to be spread in a familial or other tight pattern and depend for its control more on personal cleanliness and less on sanitation. (An extreme example, though not excreta-transmitted, is given by venereal diseases which do not survive in the environment and depend on intimate contact for their spread).

However, a low infective dose in an environmentally persistent organism will lead to an infection very difficult to shift either by sanitation or by personal and domestic cleanliness. Many viruses fall into this category and pose very major problems of control so that induced immunity may be the best approach, as discussed above for poliomyelitis. In Category II the role of sanitary improvements is to reduce the efficacy of the longer cycles and thus have a greater overall benefit than for Category I pathogens where these longer cycles are of little significance.

Category III. This category contains the soil-transmitted helminths. They are latent and persistent (Figure 3). Their transmission has little or nothing to do with personal cleanliness since the helminth eggs are not immediately infective to man. Domestic cleanliness is relevant only as it affects incoming infective stages by food preparation methods or the maintenance of latrines in a tolerable state so that eggs do not remain on the surrounds for the days or weeks of their latent period. If ova are not deposited in soil, or other suitable development sites, transmission will not occur. Therefore, any kind of latrine which contains or removes excreta, and does not permit the contamination of the floor, yard or fields, will limit transmission. Because persistence is so long (see Table 2) it is not sufficient to stop fresh faeces from reaching the yard or fields. Any faecal product which has not been adequately treated must not reach the soil. Therefore, in societies which reuse their excreta on the land, treatment is vital prior to application. Effective treatment for the removal of these ova requires waste stabilization ponds or thermophilic digestion, though prolonged storage will remove many species.

Category IV. Category IV is for the beef and pork tapeworms. Any system which prevents untreated excreta being eaten by pigs and cattle will control transmission of these infections (Figure 4). Cattle are likely to be infected in fields treated with sewage sludge or effluent. They may also eat faeces deposited in the shippen. Pigs are likely to become infected eating human faeces deposited around the home or in the pig pen. Therefore, the provision of toilets of any kind to which pigs and cattle do not have access, and the treatment of all wastes prior to land application, are the necessary control methods. It is also necessary to prevent birds, especially gulls, from feeding on trickling filters and sludge drying beds and subsequently depositing tapeworm ova in their droppings on the pastures. Personal and domestic cleanliness are irrelevant, except insofar as the use of toilets is concerned.

Category V. These are the water-based helminths which have an obligatory aquatic host or hosts to complete their life cycles. Control is achieved by preventing untreated nightsoil or sewage from reaching water in which the aquatic hosts live (Figure 4). Thus any land application system or any dry composting system will reduce transmission. There are two complications. Firstly, in all cases, except *Schistosoma mansoni* and *S. haematobium*, animals are an important reservoir of infection

Therefore any measures restricted to human excreta can only have a partial effect. Secondly, in the case of *Schistosoma haematobium* it is the disposal of urine which is of importance and this is far more difficult to control than the disposal of faeces. Because multiplication takes place in the intermediate hosts (except in the case of the fish tapeworm - Diphyllobothrium) one egg can give rise to many infective larvae. A thousandfold multiplication is not uncommon. Therefore effective transmission may be maintained at low contamination levels and the requirements of adequate excreta disposal in terms of the percentage of all faeces reaching the toilet may be demanding.

Category VI. The excreta-related insect vectors of disease comprise three main groups. Among the mosquitoes there is one cosmopolitan species Culex pipiens fatigans which preferentially breeds in highly contaminated water, and is medically important as a vector of the worms which cause filariasis. The other two groups, flies and cockroaches, proliferate where faeces are exposed. Both have been shown to carry numerous pathogenic organisms on their feet and in their intestinal tract, but their importance in actually spreading disease from person to person is controversial though their nuisance value is great. Flies have also been implicated in the spread of eye infections and infected skin lesions.

The implied control measures are to prevent access of the insects to excreta and may be achieved by many sanitary improvements of differing sophistication. In general, the simpler the facility the more care is needed to maintain it insect-free. Cockroaches, flies and Culex mosquitoes have numerous breeding places other than those connected to excreta disposal and will never be controlled by excreta disposal improvements alone.

The way in which the categories correspond to the length of transmission routes is shown in Figure 4. The discussion has emphasised the importance of complementary inputs for control of most diseases. If excreta disposal is improved in isolation, likely control of each category is as follows:

Category I	negligible
Category II	slight - moderate
Category III	great
Category IV	great
Category V	moderate
Category VI	slight - moderate

The outstanding difference is between Categories I and II together, which depend so strongly on personal and domestic cleanliness, and the other categories which do not. If one considers the changes necessary to control Categories III and IV they are relatively straightforward - namely the provision of toilets which people of all ages will use and keep clean and the treatment of faecal products prior to land application. The reason why the literature on the impact of latrine programmes often does not show a marked decrease in the incidence of Category III and IV infections is because, although latrines were built, they were typically not kept clean, not used by children, nor by adults when working in the fields.

ENVIRONMENTAL EPIDEMIOLOGY AND SANITATION

by

David J. Bradley and Richard G. Feachem

Abstract

This paper reviews the key variables determining the transmission of excreta-related diseases, sets out an environmental rather than biological classification of these infections, and relates this to the efficacy of different technologies for excreta disposal in improving health. Excreta-related diseases comprise the excreted pathogens and also those infections whose vectors breed in relation to excreta. Spread of excreted infections depends on the excreted load of organisms and the infective dose for man, and upon three characteristics of the pathogen: latency of infection, persistence of the organism in the environment, and any multiplication that occurs there. Human host responses and the existence of animal reservoirs of infection are also relevant. We recognise six categories: immediately infective pathogens with large and small minimal infective doses, and four others. Three have latent periods with development in the soil, domestic stock, or aquatic invertebrates respectively, and the last category is for excreta-related insect vectors. Each has a different response to excreta disposal methods.

COST-EFFECTIVE USE OF MUNICIPAL
WASTEWATER TREATMENT PONDS

by
Sherwood C. Reed,[1] M. ASCE and Alan B. Hais [2]

ABSTRACT

Treatment ponds are a cost-effective alternative for municipal wastewater treatment. When compared to other secondary treatment alternatives, ponds are generally the least costly, require less energy and less skilled operational attention. They can be designed to consistently meet BOD removal requirements and can achieve significant reductions in nutrients, bacteria, and viruses.

INTRODUCTION

The passage of the Federal Water Pollution Control Act in 1972 required the utilization of cost-effective waste treatment technology to "restore and maintain the chemical, physical, and biological integrity of the Nation's waters." There was an apparent rush to adopt mechanical systems and "sophisticated" technologies that could be described mathematically and precisely managed to yield a specific effluent quality. Such approaches in effect trade energy and labor skills for space and time in the treatment process. Many concepts such as land treatment and treatment ponds were either ignored or considered "old fashioned" and ineffective. It is the purpose of this paper to demonstrate that treatment ponds will continue to be a cost-effective alternative for many small and moderate sized communities.

HISTORY

Natural ponds and water bodies have been used as receptacles for wastes for centuries. The first use of a constructed pond for this purpose in the United States was in San Antonio, Texas, in 1901. By the 1920's a systematic design approach, based largely on trial-and-error experience was emerging. By 1950 ponds were an accepted wastewater

[1] Environmental Engineer, USA Cold Regions Research Laboratory, Hanover, NH
[2] Chief, Municipal Technology Branch, Office of Water Program Operations, US EPA, Washington, DC

treatment technology, particularly for small municipalities in rural areas. During the 1960 decade and continuing to the present, engineers and scientists were hard at work developing theories and procedures for a more rational basis for pond design. At present, there are over 5000 communities in the United States using waste treatment ponds, with greater than 99% designed for flows of 2 MGD or less.

However, after 75 years of development, construction and operation, there is still a very limited data base on performance of these ponds. Such data are essential, not only to validate design theories but to ensure realization of discharge standards and water quality goals. In 1975 the US EPA began a series of evaluations and characterizations of long-term ponds performance in various climatic zones of the U.S. Sites included: Corinne, Utah; Eudora, Kansas; Kilmichael, Mississippi; and Peterborough, New Hampshire. Comprehensive reports on these systems were published in 1977 ([3,5,7,15]). These were all non-aerated "facultative" ponds. A similar study of partially mixed aerated ponds was also undertaken([9]); locations and design factors are listed in Table 7.

As in other aspects of environmental engineering the jargon is somewhat confusing. Terms such as: ponds, lagoons, basins, in various combinations with: oxidation, stabilization, aerobic, facultative and aerated-facultative are often used interchangeably. For the purposes of this paper the following definitions are offered below. While not identical to the presentation of terminology in the Environmental Protection Agency Technical Bulletin, "Wastewater Treatment Ponds,"([19]) the definitions are consistent with those contained in that Bulletin.

Wastewater Treatment or Stabilization Ponds - Any type of earthen basin, lined or unlined, that has been designed to stabilize wastewater with significant dependence on natural processes.

Oxidation Pond - A shallow (3 ft) totally aerobic pond dependent on photosynthetic production of oxygen by algae and other aquatic plants. Shallow depth essential for complete sunlight penetration.

Facultative Pond - A deeper pond (3-6 ft) with two zones of treatment. The near-surface layers are aerobic because of the photosynthesis and natural surface reaeration. The lower depths are anaerobic and accumulated benthic sludge undergoes anaerobic decomposition.

Partial Mix Aerated Pond - Similar or deeper than facultative pond. Designed for mechanical aeration as a source of oxygen but not for complete mixing. Lack of complete mixing allows accumulation of some anaerobic benthic deposits on bottom and development of algae and other aquatic plants near the surface.

Controlled Discharge Pond - Could be any one of the above. Designed for long-term retention and controlled discharge once or twice a year when receiving waters and effluent quality are compatible.

Complete Retention Pond - Usually the shallow oxidation type. Dependent on evaporation and/or percolation (where permitted) so that there is no discharge.

Treatment-Storage Pond - Used as a preliminary step in land treatment systems. Treatment portion either in separate cell or proportional part of single pond. Functional treatment is usually facultative or partial mix aeration.

It should be noted that complete mix aerated ponds are not included in the above listing. A complete-mix system with sludge return can be designed as a variation of the activated sludge process and therefore falls outside the scope of this paper.

Anaerobic ponds are also not listed since by themselves they can only provide partial treatment. They are usually incorporated as the first component in the treatment of strong organic wastes either from industry or agriculture. Where site conditions permit, there are advantages to including a small anaerobic cell in a multi-cell treatment pond system.

POND DESIGN

A complete presentation of process design criteria is beyond the scope of this paper. Table 1 summarizes the ranges of criteria used for the various types of ponds. A range of factors based on average winter time air temperatures is given for facultative ponds since the natural oxygen sources are inhibited by low temperatures and ice cover. Data in the table for oxidation ponds reflect the assumption that they are significantly shorter detention times and higher organic loadings are possible in warm sunny climates. A range of termperature-dependent values is not necessary for partial mix aerated ponds since the oxygen is supplied mechanically but adjustments for detention time and for volume are necessary in colder climates due to lower reaction rates and ice cover. Additional descriptive details on design of facultative and partial mix aerated ponds are given below since these are the most common types in use.

Facultative Ponds

There are several approaches to the design of facultative lagoons: Oswald's method[11] is based on anaerobic fermentation of benthic deposits and algal growth potential in the liquid. It requires specific knowledge of incident radiant energy and its rate of useful uptake by the algae.

The Marais model[6] describes the dynamic behavior of ponds using differential equations based on first order reaction kinetics. It is an exacting approach requiring careful evaluation of the kinetic rate constants used in the design equations.

Thirumurthi's[18] approach is similar to the Marais model and is based on a first order kinetics removal rate for BOD, and assumes that basin conditions are intermediate between plug flow and completely mixed.

Gloyna's method[4] is a set of empirical equations based on surface area as the critical parameter and temperature as the major variable. It also requires some experience generated estimates for toxicity factors and sulfide oxygen demand. His basic equation is:

$$A = 3.5 \times 10^{-5}\, QS_u \; [\theta^{(35-T)}]ff' \qquad (1)$$

Where A = surface area of pond $(M)^2$; assumes treatment *volume equal to 1 meter depth. Additional* volume for ice cover and sludge storage must be provided

Q = sewage flow rate (l/day)

S_u = ultimate BOD of influent (mg/l)

θ = temperature coefficient = 1.085

T = mean water temperature in treatment volume during coldest period (°C)

f = algal toxicity factor (assume = 1 for domestic wastes)

f' = sulfide or other immediate COD, f' = 1 for SO_4 equivalent ion concentration of less than 500 mg/l.

This equation is widely used because of its simplicity and the relative ease in determining input values.

All of these methods were evaluated against the performance of a facultative pond in Corinne, Utah([15]). None of them accurately described the system. The Gloyna method overestimated surface area by approximately 17% with the input data used. Both the Marais and Thirumurthi methods underestimated the BOD removal capacity of the system.

As shown in Table 1 the detention time must be increased for facultative ponds in cold climates. Winter time performance under a thick ice cover is little better than primary treatment so detention over the winter is necessary. As indicated previously, controlled discharge systems offer further advantages when such long detention times are necessary in that flexibility is insured to allow discharge when effluênt and receiving water conditions are best.

Partial Mix Aerated Ponds

Although by definition such ponds are not completely mixed, a common approach to their design is based on first order reaction equations for completely mixed flow([16;20],):

$$\frac{Se}{So} = \frac{1}{1 + Kt} \qquad (2)$$

where:

Se = effluent BOD_5 (mg/l)

So = influent BOD_5 (mg/l)

K = overall reaction rate coefficient

Base e: winter (0.5°C) 0.14

summer (16-10°C) 0.28

t = time (days)

For multiple cell ponds this can be rearranged to:

$$Kt = \left[N \left(\frac{So}{Se}\right)^{\frac{1}{N}} - 1\right] \qquad (3)$$

where:

N = number of cells

other factors as defined previously

Equation (3) can be solved on a trial and error basis to optimize the number of cells and total detention time. The rate coefficient for winter conditions should be used in this determination since biological activity is the slowest at that time. Summer conditions should be used to determine oxygen requirements. The detention time resulting from equation (3) determines actual treatment volume required in the winter. Additional space must be allowed for sludge accumulation and ice formation. In Alaska, which might represent the worst case, an allowance of 5% is made for sludge storage and 15% for ice formation in calculating total volume[14,16]. The depth of the cell or pond is based on requirements for the type of aeration equipment chosen.

Table 2 compares approximate area and cell requirements for facultative, partial-mix aerated and controlled discharge ponds for a typical 1 MGD system.

CONSTRUCTION REQUIREMENTS

Standard earthmoving procedures usually suffice for pond construction. A design can usually be based on a balanced cut and fill so that most of the excavated material can be used in dike construction. Outside slopes of dikes are usually 4:1 or flatter to permit grass mowing. Inside slopes are steeper, ranging from 2:1 to 3:1.

Lining of the pond bottom and inner dike surfaces may be necessary if compaction of the in-situ soils does not produce an acceptable level of impermeability. In general, all of the states in the U.S. require protection of the beneficial use of groundwater beneath a pond. Only 17 states define a specific seepage limitation; these are listed in Table 3. Most states do not have a specific value but decide on a case-by-case basis for protection of groundwater([10]). Lining materials range from locally available clays, bentonite, asphalt, concrete, soil cement, and various membranes. Some of the low seepage rates listed in Table 3 would be difficult to achieve with soil stabilization techniques so constructed liners or membranes might be necessary. It is anticipated that standards will become more stringent in the future, particularly for industrial applications, so that the use of essentially impervious lining will likely be more prevalent.

Partial lining of dikes and pond bottoms may also be necessary to control soil erosion from wave action or aeration turbulence. The membrane liners are easily punctured and some are sensitive to solar radiation so it is common practice to overlay the above water portion with soil and rip-rap.

COSTS AND ENERGY

Waste treatment ponds are usually a cost-effective alternative when sufficient land is available. Table 4 compares estimated construction costs for 100,000 gpd ponds to other forms of biological treatment and indicates the land area required for each type of system.

Operational energy requirements for ponds and other biological treatment processes are compared in Table 5. The values do not include energy for raw sewage pumping, preliminary treatment or disinfection since these generally are common to all alternatives. It is clear that ponds require the least energy. The differences shown would be even greater if sludge digestion and disposal requirements were added to the non-pond alternatives.

PERFORMANCE

Facultative and Oxidation Ponds

In an analysis of the data from the EPA sponsored performance study, Middlebrooks[9] concluded that facultative lagoons:

- Are subject to seasonal performance variation
- Can produce an effluent BOD_5 of 30 mg/l or less
- Can produce an effluent with relatively low suspended solids
- Effluent suspended solids tend to be higher during the summer due to algal growth
- Fecal coliform reductions are primarily a function of hydraulic residence time

A statistical analysis was conducted of the seven cell facultative lagoon in Corinne, Utah[15] to determine the optimum number of cells for pollutant removal. A summary of these results is shown in Table 6. For both BOD_5 and suspended solids removal a maximum of 5 cells was recommended. In the Corinne system this would provide a total detention time of approximately 67 days. The removal of nitrogen and phosphorus appeared to be dependent on hydraulic residence time with significant removals continuing through all of the cells. In the case of BOD_5 and suspended solids, the analysis indicated that further significant removals were not achieved beyond the fifth cell. In fact, the effluent from the second cell (total detention 44 days) had an annual average BOD_5 less than 30 mg/l.

Partial-Mix Aerated Ponds

The US EPA has also sponsored a performance analysis of five aerated lagoon systems. A partial analysis of data from these systems, by Middlebrooks(9) indicated the following:

- o Aerated ponds can produce an effluent BOD_5 concentration of less than 30 mg/l.
- o Aerated pond suspended solids concentrations are affected by seasonal variations.

Design criteria for the five systems studied are given in Table 7. The number of cells ranged from two to three and detention times from 25 to 68 days. A comparison of the BOD loading values in this table to the recommended design factors in Table 1 indicate that the actual values are significantly higher than the criteria commonly found in the engineering literature.

Controlled Discharge Ponds

Pierce(12) has described performance of controlled discharge ponds in Michigan and in Minnesota. The number of cells in the Michigan ponds ranged from 2 to 5. A statistical analysis of the data indicated that two cells were sufficient to yield a most probable effluent BOD_5 and suspended solids of 30 mg/l or less.

Thirty six of the 39 systems in Minnesota had an effluent BOD of less than 25 mg/l and suspended solids less than 30. Discharge from these ponds is in late spring and early fall. The treatment responses are functionally the same as facultative ponds with the discharge period chosen to avoid algal blooms and high BOD from minimal biological activity in the winter. The discharge period ranges from five to greater than 31 days. Piping is arranged so that flow into the cell to be discharged can be diverted to another cell for a period prior to discharge.

A five cell facultative pond originally designed as a twice-annual controlled discharge has, because of high flows, been forced to discharge more frequently. Operational data from 1975([13]) is summarized in Table 8. The first cell in the system is a small anaerobic pond. The other four cells total 50 acres. It has been noted([13]) that there is little additional treatment in the fifth cell. The characteristics cited in Table 8 can therefore be achieved by a three cell (42.5 acres, 155 day detention capacity) facultative pond preceded by a small anaerobic cell.

Treatment-Storage Ponds

These are similar in some respects to controlled discharge concepts but the discharge in this case will be to a land treatment system. The volume required for storage is due to climatic constraints on operation of the land treatment system or a seasonal imbalance between wastewater supply and application schedules. The period and total volume for storage can be determined in accordance with procedures in the Process Design Manual for Land Treatment of Municipal Wastewater (EPA 625/1-77-008)([22]). The storage volume should be designed with the maximum depth appropriate for site conditions.

The treatment portion of such systems is usually designed as an aerated or facultative unit. These could either be in separate cells or combined with storage in the same cell.

An unaerated treatment volume will function as a facultative system and will provide a comparable degree of stabilization. This can be estimated using one of the design approaches for facultative ponds. For calculation purposes it would be conservative to assume that approximately 1 meter of depth in the storage is providing treatment. For example:

Assume: 1 MGD system, mean temperature in storage 5°C

120 day storage in 10 ft. deep pond

60 mg/l BOD_5 acceptable for land application

So, surface area = 37 acres

1 meter depth equivalent to 39 days retention

Using the Marais equation:

$$P_i = P_o \quad K_T R + 1 \qquad (3)$$

Where: P_i = influent BOD_5 (mg/l)

P_o = effluent BOD_5 (mg/l)

R = detention time (days)

$K_T = 1.2 \left[\theta^{-(35-T)}\right]$

$\theta = 1.985$

T = mean temperature during period of concern

The allowable influent BOD_5 entering the storage pond would be 250 mg/l to produce an effluent of 60 mg/l under the conditions assumed. If the raw sewage BOD were greater than 250 mg/l then additional treatment cells should precede the storage unit. They would be designed in the standard manner.

Data from the storage ponds at the land treatment system in Muskegon, Michigan[23] are summarized in Table 9. The BOD_5 entering the ponds was 81 mg/l and after storage 13 mg/l. Equation (3) above, with the assumptions listed would predict a BOD of 16 mg/l with an input of 81 mg/l.

As indicated in Table 9 the fecal coliform concentration was reduced by at least three orders of magnitude during storage at Muskegon. Bowles[2] has reported on fecal coliform reduction in waste stabilization ponds in Utah and has developed equations to predict detention time required for a desired reduction. For a reduction from 10^6 to 10^3 per 100 ml the predicted retention time would be 15 days under summer conditions.

Sagik, et.al[17] has reported on the survival of human enteric viruses in holding ponds. Their tests with primary effluent in model ponds showed significant removals of both poliovirus (CHAT) and coxsackievirus (B3) as summarized in Table 10. Based on this research they recommend, for temperate climates, a detention time of 30 days to maximize virus reduction in ponds.

CURRENT REGULATION AND POLICY

In September 1976 the US EPA proposed an amendment to the Secondary Treatment Regulations regarding waste treatment ponds. It was recognized that if properly designed and operated, ponds were a form of secondary treatment but that at times they might not be capable of meeting suspended solids limitations. The responsible factor in the general case was algae. Algae are naturally formed in wastewater treatment ponds. The non-aerated type are specifically designed to rely on photosynthetic oxygenation. These algal cells which are an integral part of the treatment system do no settle readily and may be carried out of ponds as suspended solids in the effluent.

Methods for removing algae from pond effluents have been developed and are described in the literature([9]). However, any type of supplemental treatment method unavoidably adds to the complexity of pond design and may strain the operational capability of small communities. There was also a reluctance on the part of many communities and state agencies to adopt these emerging polishing technologies, with the only alternative remaining being replacement of the pond with a more complex mechanical treatment system. As a result the proposed amendment (41 FR 37222, Sept. 2, 1976) would allow an upward adjustment of suspended solids limits under certain circumstances. Following review and discussion, the final rule was published in October 1977 (42 FR 54664, Oct. 7, 1977). An adjustment was allowed if: (1) waste treatment ponds are the sole process used for secondary treatment, and (2) the maximum design capacity is 2 MGD or less.

The adjusted value was to be equal to the effluent concentration achieved 90% of the time within a state or appropriate contiguous geographical area by ponds that are achieving the levels of BOD removal required by regulations (i.e., 30 mg/l on a 30 day average, 45 mg/l on a 7 day average and 85% removal). A statistical analysis of available data by the states and EPA Regional Offices was conducted to determine the amended values which are shown in Table 11. In some cases the data base for the analysis was quite limited and in all cases additional data are being collected. A periodic re-evaluation of this expanding data base could result in further changes so the values are considered "interim final" by the US EPA.

Unless specified in the table, the values given apply to all types of ponds in the location specified with the following exceptions: ponds used as a final polishing step for other secondary treatment systems are not eligible, nor are ponds which include complete-mix aeration and sludge recycle or return since these systems are in essence a variation of the activated sludge process. The amended values only apply to ponds with a design flow of 2 MGD or less.

CONCLUSIONS

The data and discussion presented earlier in this paper tend to indicate:

- Ponds offer significant economic advantages for small and moderate sized communities.
- Ponds require less energy than potentially alternative technologies.
- Successful operation is not dependent on highly skilled operating personnel.
- Ponds can consistently meet the 30 mg/l BOD_5 effluent requirement.
- Ponds can achieve very significant reductions in bacteria and viruses.

Ponds also offer advantages where future population expansion and/or more stringent treatment requirements are expected. In this context a pond offers a simple cost-effective treatment method for present conditions but lends itself to higher rate operation as the preliminary step to some final form of treatment. As an example a pond could be designed to meet current BOD requirements but then in the future be used as a preliminary step to land treatment if the loading increases and/or treatment requirements become more stringent or including nutrient removal.

The combination of treatment-storage ponds are particularly advantageous for land treatment systems as a result of the Clean Water Act of 1977 (PL 95-217) since the land required for the storage portion is eligible for Federal grant funding. In accordance with Program Requirements Memorandum (PRM #78-4) issued by the EPA in February 1978, the following criteria are used to determine eligibility:

- o If the entire cell or pond is used for storage, land is eligible.
- o If a portion is used for storage and that portion is greater than the volume provided for treatment then the total land area for that cell is eligible.
- o If the storage volume is equal to or less than the treatment volume then the eligible area will be determined as the ratio of storage volume to total.

Under these terms, the pond described earlier in the sample calculation using the Marais equation would be totally eligible for funding since less than 33% (1 meter depth) is identified as providing a treatment function.

Wastewater treatment ponds also seem to have a promising future as part of the emerging technology in aquaculture and wetland treatment systems.

As a result it is the authors' opinion that ponds will continue to be an important alternative for cost-effective wastewater treatment.

APPENDIX - REFERENCES

1. Benjes, H. H. Small Communities Wastewater Treatment Facilities - Biological Treatment Systems. Section II, Design Seminar Handout, Small Wastewater Treatment Facilities, ERIC, USEPA. Jan. 1978.

2. Bowles, D. S., E. J. Middlebrooks, J. H. Reynolds. Coliform Decay Rates in Waste Stabilization Ponds. Presented at WPCF annual conference, October 1977.

3. Bowen, S. P. Performance Evaluation of Existing Lagoons, Peterborough, N.H. EPA 600/2-77-085, MERL, USEPA Aug. 1977.

4. Gloyna, E.F. Facultative Waste Stabilization Pond Design. Ponds as a Wastewater Treatment Alternative. Univ. Texas, Austin. June 1975.

5. Hill, D. O., A. Shindala. Performance Evaluation of Kilmichael Lagoon. EPA 600/2-77-109. MERL, USEPA Aug. 1977.

6. Marais, G. V. R. Dynamic Behavior of Oxidation Ponds, 2nd International Symposium for Waste Treatment Lagoons. June 1970.

7. McKinney, R. E. Performance Evaluation of An Existing Lagoon System at Eudora, Kansas. EPA 600/2-77-167, MERL, USEPA. Sept. 1977.

8. Metcalf & Eddy, Inc. Wastewater Engineering, McGraw Hill, N.Y., N.Y. 1977.

9. Middlebrooks, E. J, J. H. Reynolds, C. H. Middlebrooks. Performance and Upgrading of Wastewater Stabilization Ponds. Section I, Design Seminar Handout, Small Wastewater Treatment Facilities. ERIC, USEPA, Jan. 1978.

10. Middlebrooks, E. J. et al. Wastewater Stabilization Lagoon Linings. USA CRREL Special Report (In press). Hanover, N.H. Aug. 1978.

11. Oswald, W. J. et al. Designing Ponds to Meet Water Quality Criteria, Second International Symposium for Waste Treatment Lagoons. June 1970.

12. Pierce, D. M. Performance of Raw Waste Stabilization Lagoons in Michigan With Long Period Storage Before Discharge. Proceedings Symposium Upgrading Wastewater Stabilization Ponds Meet New Discharge Standards Utah State University PRM G159-1, Nov. 1974.

13. Pierce, D. M. Wastewater Treatment Ponds at Belding, MI Meet Secondary Treatment Requirements, unpublished Feb. 1978.

14. Reed, S. C. Alternatives for Upgrading USAF Wastewater Lagoons in Alaska, USACRREL, June 1976.

15. Reynolds, J. H., et al. Performance Evaluation of an Existing Seven Cell Lagoon System, EPA 600/2-77-086, MERL, USEPA, Aug. 1977.

16. Reid, L. C. Design & Operation of Aerated Lagoons for the Arctic and Subarctic. *EPA Tech. Transfer Design Seminar, Anchorage, AK.* April 1975.

17. Sagik, B. P. et al. The Survival of Human Enteric Viruses in Holding Ponds Draft Final Report, US Army Medical Research and Development Command. Contract DHMD - 17-75-C-5062, Jan. 1978.

18. Thirumurthi, D. Design Criteria for Waste Stabilization Ponds. J.W.P.C.F. 46 (9) 2094-2106 (1974)

19. USEPA Wastewater Treatment Ponds. EPA 430/9-74-011, Office of Water Program Operations. March 1974.

20. USEPA Process Design Manual - Wastewater Treatment Facilities for Sewered Small Communities. EPA-625/1-77-009. ERIC, USEPA. October 1977.

21. USEPA Construction Costs for Municipal Wastewater Treatment Plants: 1973-1977. EPA 430/9-77-013, OWPO, Jan. 1978.

22. USEPA, Process Design Manual for Land Treatment of Municipal Wastewater. EPA 625/1-77-008, ERIC, Oct. 1978.

23. Walker, J. M. Wastewater: Is Muskegon County's Solution Your Solution? Region V, USEPA. 1976.

TABLE 1 - Design Factors for Treatment Ponds

Type (1)	Detention Time days (2)	Depth ft (3)	BOD Loading lb/acre/day (4)
Oxidation	10-40	1.5-3	60-120
Facultative			
Winter Average Air Temperature			
Above 60°F	25-40	3-5	40-80
32-60°F	40-60	4-6	20-40
Below 32oF	80-180	5-7	10-20
Partial Mix Aerated	7-20	8-10	30-100

TABLE 2 - Size Requirements for 1MGD Treatment Pond

Type	Surface Area (acres)	Minimum Number of Cells	Cell Size (acres)
Oxidation	30	3	10
Facultative			
Flow-through	25	3	2-10
Controlled discharge	95	3	2-10
Partial Mix Aerated	7	3	2-10

TABLE 3 - Allowable Seepage Rates From Ponds (10)

State	Seepage Limitation
Arizona	Zero with toxic substances; no liner for domestic wastewater if perc. rate greater than 60min/in.
Florida	Holding ponds for treated wastewater 0.25"/day; water supply well within 1000ft 0.1"/day
Idaho	0.25"/day
Indiana	Initial rate 0.125"/day
Iowa	0.125"/day
Michigan	No written policy 0.062 - 0.125"/day generally accepted.
Missouri	0.25"/day
Montana	Initial rate 0.25"/day
Nebraska	0.125"/day
North Dakota	0.125"/day
Oregon	0.125"/day
Pennsylvania	Coefficient of Permeability 2.54×10^{-7} in/sec
South Dakota	0.062"/day
Texas	2.54×10^{-7} in/sec
Utah	0.25"/day
Vermont	Permeability 2.54×10^{-7} in/sec
Washington	0.25"/day

TABLE 4 - Estimated Construction Costs 100,000 gpd Wastewater Treatment*
1977 $

System Type	Capital Costs $1000	Land Area (acres)
Facultative Pond (20)		
Northern Climate	319	5.0
Southern Climate	143	3.0
Partial Mix Aerated Pond (20)	336	1.0
Oxidation Ditch (1)	384	1.0
R.B.C. (1)	549	1.5

* Does not include raw sewage pumping, disinfection or land costs

TABLE 5 - Estimated Operational Energy Requirements 100,000 gpd Wastewater Treatment*

System Type	Annual Energy Requirement (1000 kwh/yr)
Facultative Pond	0
Partial Mix Aerated (19)	15
Oxidation Ditch (1)	43
R.B.C. (1)	18

*Does not include raw sewage pumping, preliminary treatment, disinfection, or sludge digestion and disposal.

TABLE 6 - Recommended Number of Cells at Corinne, Utah Facultative Lagoon (15)

Parameter	Recommended Number of Cells	Detention Time (days)	Annual Average Influent (mg/l)	Annual Average Effluent (mg/l)
BOD_5	5	67	75	16
S.S.	5	67	69	36
Fecal Coliform	4	58	928,673/100ml	17.4/100ml
Phosphorus	7	88	4.0	2.1
Nitrogen	6	76	15.3	3.0

TABLE 7 - EPA Aerated Ponds Study (9)

Location	BOD Loading (#/acre/day)	Number of Cells	Detention Time (days)	Surface Area (acres)	Design Flow (MGD)
Bixby, OK	128	2	67.5	5.8	0.4
Pawnee, IL	77	3	60	11.0	0.5
Gulfport, MS	179	2	26.2	6.3	0.5
Lake Koshkonong, WI	149	3	57	6.9	0.6
Windber, PA	145	3	55	20.7	2.0

TABLE 8 - 1975 Performance Belding MI Facultative Pond (13)

Discharge Period	No. of days	Total Discharge MG	Final Effluent (mg/l)			
			BOD_5	SS	NH_3-N	NO_3-N
Feb. 2-20	8	22	20.2	18.2	13.7	0.1
Mar. 18-30	12	8	10.2	13.0	13.6	0.1
Apr. 7-25	15	77	10.8	20.3	11.3	0.2
May 5-9	5		7.7	18.4	2.1	1.0
Oct. 16-31	12	50	1.2	3.5	0.6	0.3
Nov. 3-12	8	24	0.6	1.6	3.8	0.6
Dec. 1-5	5	14	1.2	3.2	4.9	0.9

TABLE 9 - 1975 Renovation in Muskegon, MI Storage Pond (22)

Parameter	In (mg/l)	Out (mg/l)
BOD_5	81	13
Suspended Solids	144	20
Phosphorus	2.4	1.4
Nitrogen ($TKN + NO_3$)	8.3	5.6
Fecal Coliform	$>10^6$/100ml	10^3/100ml

TABLE 10 - Virus Survival in Holding Ponds (17)

Parameter	% Removal at:					
	5 days	20 days	30 days	60 days	100 days	120 days
Polio virus at 20^{o}c	98	99.9	-	-	-	-
Coxsackie virus at 20^{o}	80	99.9	-	-	-	-
Polio virus at 4^{o}c	1	60	75	95	99	99.5
Coxsackie virus at 4^{o}c	1	35	50	75	92	95

TABLE 11 - Suspended Solids Limitations for Wastewater Treatment Ponds

Location	Suspended Solids Limit* (mg/l)
Alabama	90
Alaska	70
Arizona	90
Arkansas	90
California	95
Colorado	
Aerated Ponds	75
All others	105
Connecticut	N.C.
Delaware	N.C.
District of Columbia	N.C.
Florida	N.C.
Georgia	90
Guam	N.C.
Hawaii	N.C.
Idaho	N.C.
Illinois	37
Indiana	70
Iowa	
Controlled Discharge, 3 cell	Case-by-case but not greater than 80
All others	80
Kansas	80
Kentucky	N.C.
Louisiana	90
Maine	45
Maryland	90
Massachusetts	N.C.
Michigan	
Controlled Seasonal Discharge	
Summer	70
Winter	40
Minnesota	N.C.
Mississippi	90
Missouri	80
Montana	100
Nebraska	80
North Carolina	90
North Dakota	
North & East of Missouri River	60
South & West of Missouri River	100
Nevada	90
New Hampshire	45
New Jersey	N.C.
New Mexico	90
New York	70
Ohio	65
Oklahoma	90
Oregon	
East of Cascade Mts.	85
West of Cascade Mts.	50
Pennsylvania	N.C.
Puerto Rico	N.C.
Rhode Island	45
South Carolina	90
South Dakota	110
Tennessee	100
Texas	90
Utah	N.C.
Vermont	45
Virginia	
East of Blue Ridge Mts.	60
West of Blue Ridge Mts.	78
Eastern slope counties: Loudoun, Fauquier, Rappahannock Madison, Green, Albemarle, Nelson Amherst, Bedford, Franklin, Patrick	case-by-case application 60/78 limits
Virgin Islands	N.C.
Washington	75
West Virginia	80
Wisconsin	60
Wyoming	100*

NOTES: N.C. - no change from existing criteria
* - thirty consecutive day average or average over the period of discharge when the duration of the discharge is less than 30 days.

LAND TREATMENT SYSTEMS AND THE ENVIRONMENT

by

Harlan L. McKim[1], John R. Bouzoun[2], C. James Martel[2],
Antonio J. Palazzo[3] and Noel W. Urban[4]

INTRODUCTION

Many countries have been applying wastewater to the land for centuries. In almost all instances the nutrients in the wastewater from agricultural and domestic sources have been used to grow crops. Forage crops produced at land treatment sites are of high quality and can be used as an animal food source. When wastewater is applied to various forest species, yields may increase by a factor of two (1). This increase in yield may provide an alternative energy source to replace diminishing oil and gas supplies. This alone can make the concept of land application of wastewater a cost-effective alternative to other forms of wastewater treatment.

In the United States several land treatment systems have been in operation since the turn of the century (2). In the past ten years, however, Federal legislation (such as Public Law 92-500, the Water Pollution Control Act Amendments of 1972, and Public Law 95-217, the Clean Water Act of 1977) has given municipalities added

---

1. Program Manager, Land Treatment of Wastewater Research Program, U.S. Army Corps of Engineers Cold Regions Research and Engineering Laboratory, Hanover, New Hampshire.

2. Environmental Engineer, U.S. Army Corps of Engineers Cold Regions Research and Engineering Laboratory, Hanover, New Hampshire.

3. Agronomist, U.S. Army Corps of Engineers Cold Regions Research and Engineering Laboratory, Hanover, New Hampshire.

4. Chief, Engineering Management Branch, Research Division, Directorate of Civil Works, Office of the Chief of Engineers, U.S. Army Corps of Engineers, Washington, D.C.

incentive to utilize land treatment, by increasing the percentage of funding under the federal construction grants program. With this incentive the cost of land treatment systems will be more competitive when compared to conventional treatment processes. This should encourage the construction of more land treatment systems.

Corps of Engineers research conducted over the last five years has emphasized land application of wastewater as a treatment process rather than a disposal method. The primary objective of the program is to develop design criteria and management procedures which allow the maximum amount of wastewater to be applied to the minimum amount of land without adversely influencing the quality of receiving waters. The following paper summarizes the results from several experiments using the rapid infiltration, overland flow and slow infiltration modes of land treatment.

PRETREATMENT

Experiments conducted by the Cold Regions Research and Engineering Laboratory (CRREL) have demonstrated that land treatment can be used to produce excellent product water quality with pretreatment of the sewage to only primary levels. Pretreatment beyond the primary level is not necessary from a process performance point of view. Secondary treatment can be justified only if disinfection is required prior to land application (3,4,5).

For small communities and recreational areas, storage ponds are **usually the most cost-effective** method of pretreating sewage before application to the land. In most instances the sewage is treated to secondary levels in the holding or storage ponds and the nutrient levels are low (6,7). When this secondary wastewater is applied to land for its nutrient value, the crop yields may be low and fertilizer may need to be added to obtain a desirable plant yield. Therefore, it is important to establish treatment and utilization goals before final design of a treatment facility.

METHODS OF LAND TREATMENT

Basically there are three methods of land treatment: rapid infiltration, slow infiltration and overland flow. All of these systems can be designed and operated to remove BOD, suspended solids, phosphorus, nitrogen and trace elements. In rapid infiltration systems the wastewater is applied to a very deep permeable soil (Fig. 1). Soils that have an infiltration rate of 75.0 cm/hr can be considered (8). The water moves through the soil primarily under unsaturated flow. Mounding of the water beneath the system should not be a problem but will occur at high application rates. Usually the land area required for a rapid infiltration system is small because large quantities of wastewater can be treated on a minimum of land.

The slow infiltration (slow rate) mode of land treatment is the method used primarily when the infiltration rate of the soils is between 0.15 to 50 cm/hr (8). In slow infiltration systems the wastewater percolates slowly through the soil with vegetative cover being an integral part of the treatment process (Fig. 2).

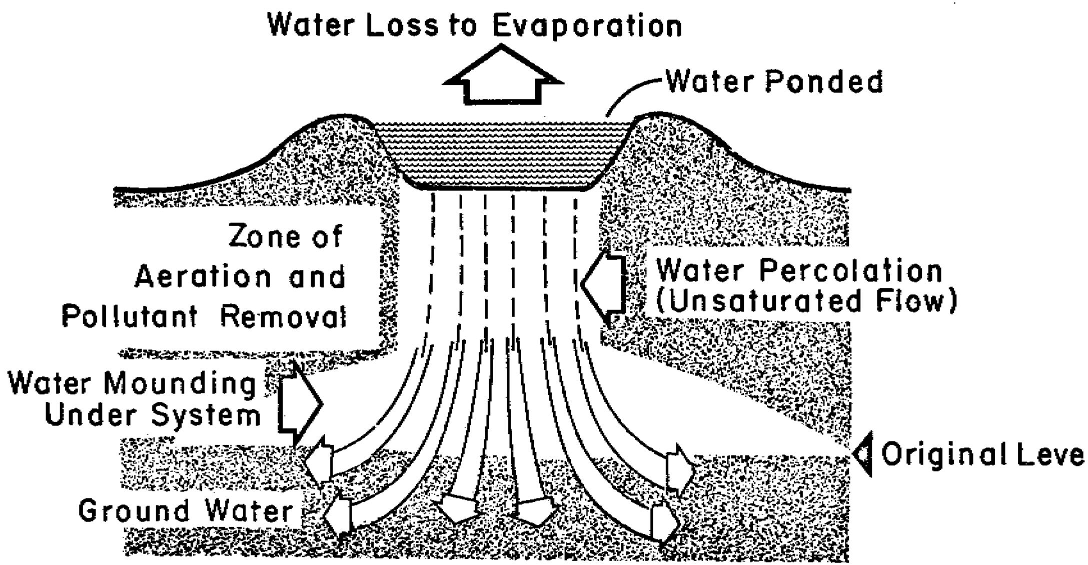


Figure 1. Rapid Infiltration.

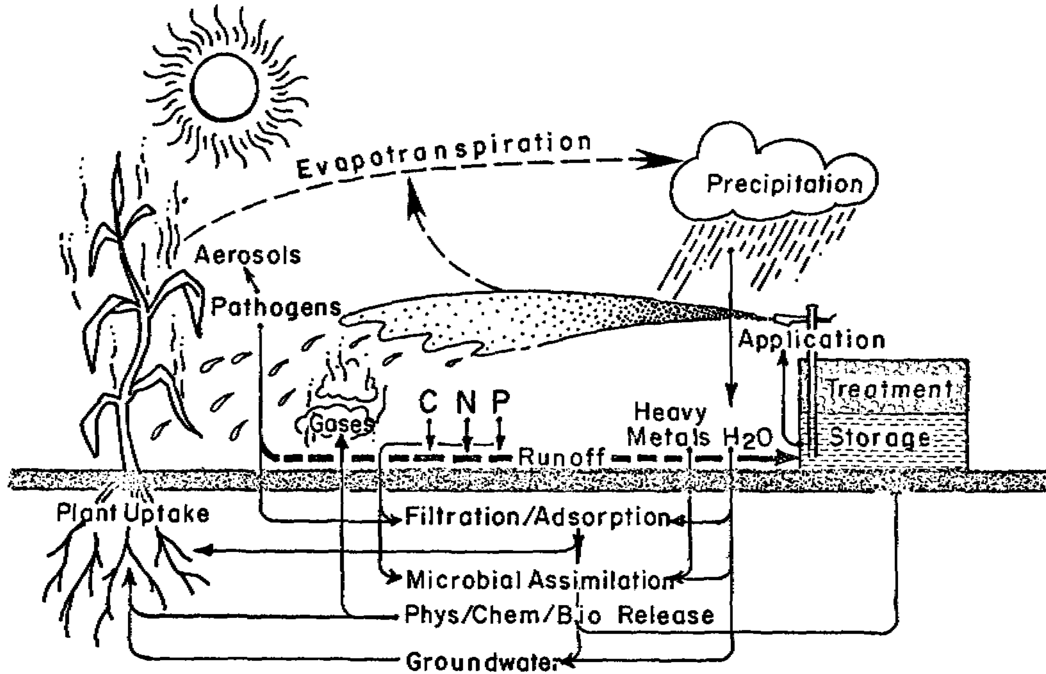


Figure 2. Slow Infiltration.

In overland flow systems the wastewater moves slowly over the surface of a very impermeable soil (< 0.5 cm/hr) (Fig. 3). Approximately 60% of the applied wastewater is collected downslope and is usually discharged to surface waters. Grass is usually the crop grown on overland flow systems.

RAPID INFILTRATION

When rapid infiltration systems are operated and managed properly they can provide long-term renovation of wastewater (9,10,11,12,13,14, 15,16). The primary design criteria for rapid infiltration systems are application rate and schedule. Application rate depends on the soil infiltration rate, wastewater solids content and desired percolate water quality. The length of the wastewater inundation period varies depending on such parameters as geographical area, soil characteristics and climatic conditions. Also, adequate drying periods are required to ensure that the infiltration capacity of the soil is not permanently impaired.

At Fort Devens, Massachusetts, unchlorinated effluent from Imhoff tanks has been applied to 22 treatment beds for 30 years. Currently infiltration beds are flooded for a period of two days, followed by a 14-day drying period. Using this cycle, each treatment bed receives primary sewage effluent about 52 days each year. Given a mean annual flow of 5,061 m^3/day (1.34 mgd) and equal effluent distribution per unit

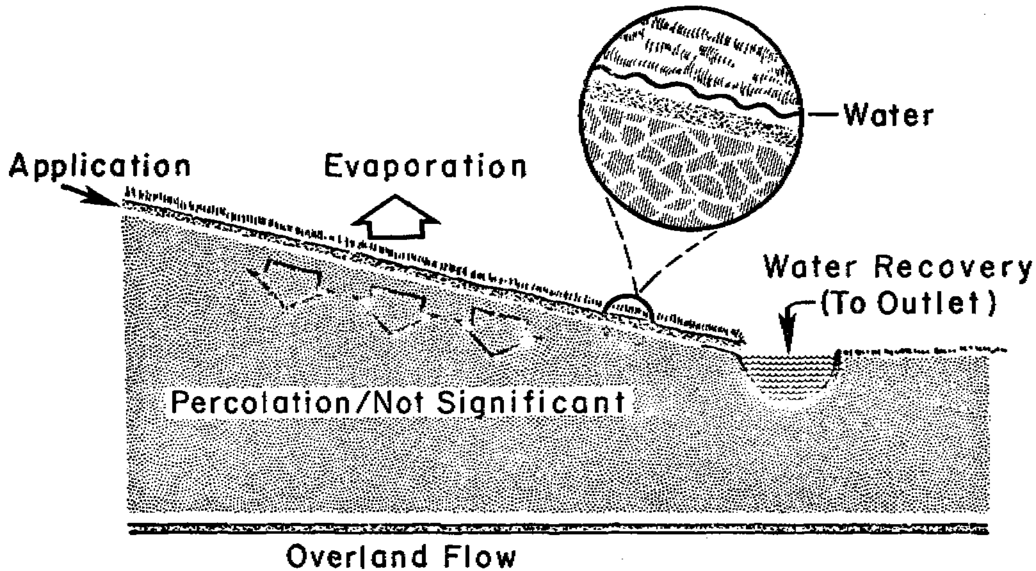


Figure 3. Overland Flow.

area, the wastewater applied to the beds has averaged about 28.3 m/yr (93.8 ft/yr) over the last 15 years (17,18).

To ensure proper operation of the Fort Devens system the infiltration capacity of the soil is maintained by annually removing the top 0.3 m of the soil surface for each treatment bed. The excavated materials are replaced with locally available sand and gravel.

The average water quality characteristics of the primary effluent and groundwater obtained from wells adjacent to the treatment site are shown in Table 1. Wells were installed at depth intervals of 1.5 m so that water quality changes with depth could be observed (17,18). The data indicate that removals of phosphorus, BOD and nitrogen were >85, 98 and 60 percent respectively.

Results obtained from an experimental rapid infiltration site at Flushing Meadows, Arizona have shown that a 9-day flooding and 12-day drying period with hydraulic loading rates of 60-75 m/yr resulted in nitrogen removal rates of 60 percent (13). The nitrogen in the applied effluent ranged from 20 to 40 mg/l, and the product water from the treatment site contained average concentrations of nitrate, ammonium and organic N of 8.6 mg/l, 1.7 mg/l, and 0.5 mg/l, respectively. The primary mechanism of nitrogen removal was identified as denitrification. Phosphorus applied in the effluent at Flushing Meadows averaged about 8 mg/l in 1977 and a reduction of 90% was found in the percolate water after 60 m of lateral water movement. Viruses were not detected in the water sampled at 6- to 9-m depths.

Table 1. Effluent and groundwater chemical and bacteriological characteristics at Fort Devens, Massachusetts, land treatment site (after Satterwhite 1976).

A. Chemical and bacteriological characteristics of Imhoff tank effluent and annual wastewater additions to treatment beds.

Parameter	Effluent* Range	Mean
pH (standard units)	6.2-8.0	
Conductivity (μmhos)	402-700	511
Alkalinity (ppm $CaCO_3$)	116-245	155
Hardness (ppm $CaCO_3$)	22-60	41
BOD_5	30-185	112
COD	110-450	192
Total Nitrogen	19-78	47
Organic Nitrogen	11.5-32.8	23.4
NH_4-N	6.2-42	21.4
NO_3-N	0.4-2.8	1.3
NO_2-N	0.002-0.06	0.02
Total PO_4-P	6-16	11
Ortho PO_4-P	3-15	9
Chloride	75-210	150
Sulfate	27-72	42
Total Coliform bacteria x 10^6/100 ml	18-53	32

* mg/l unless otherwise indicated.

B. Chemical and bacteriological characteristics of groundwater in selected observation wells.

Parameter*	Well 3	8	9	10	11	12	13	14
pH (standard units)	6.3	6.6	6.5	6.1	6.3	6.6	6.2	6.5
Conductivity (μmhos)	360	443	310	333	305	71	36	327
Alkalinity (ppm $CaCO_3$)	28	58	67	14	53	29	17	49
Hardness (ppm $CaCO_3$)	44	44	32	30	31	17	6	30
BOD_5	2.5	2.0	1.4	0.9	0.8	1.2	1.0	0.9
COD	19	15	8	10	9	10	13	11
Total Nitrogen	19.5	19.8	10.4	20.3	12.1	3.7	1.9	9.7
Organic Nitrogen	2.3	4.2	3.7	1.2	1.0	0.8	1.2	1.5
NH_4-N	1.3	4.5	3.2	0.5	1.0	0.3	0.3	0.4
NO_3-N	15.6	10.7	3.5	18.6	10.1	2.6	0.4	7.8
NO_2-N	0.3	0.4	0.02	0.02	0.02	0.01	0.01	0.02
Total PO_4-P	0.9	0.8	1.4	1.3	1.9	0.6	0.7	1.1
Ortho PO_4-P	0.2	0.1	0.4	0.1	0.2	0.1	0.1	0.1
Chloride	230	220	144	257	221	40	15	162
Sulfate	39	36	33	35	44	9	7	46
Total Coliform bacteria (no./100 ml)	210	158	230	620	130	120	370	120

* mg/l unless otherwise indicated.

Both the Fort Devens and Flushing Meadows results have shown that rapid infiltration systems can be operated effectively with no detrimental effect to the groundwater. Again, it is important to emphasize that rapid infiltration systems should be operated and managed to maintain the initial infiltration rate of the soil.

SLOW INFILTRATION

Many slow infiltration land application systems have been successfully designed and operated to dispose of domestic and municipal wastewater (2,19,20,21,22,23,24,25,26,27). However, it is only recently that research on these systems has been conducted to maximize treatment and agricultural benefits.

Wastewaters contain on the average about 20 mg/l total nitrogen (N), 10 mg/l phosphorus (P) and 15 mg/l of potassium (K) (28). Normal plant requirements could be met with an application of 250 cm/ha per year of wastewater which would supply the plants with 500 kg N/ha, 250 kg P/ha and 375 kg K/ha (29).

Many types of crops can be grown on slow infiltration land treatment systems (30,31,32). Two extensive studies at Pennsylvania State University and the University of Washington have shown that a forest ecosystem can effectively be used to renovate wastewater (33,34,1). Grasses, legumes and row crops have also been planted at various land treatment sites. These crops remove large quantities of nitrogen from wastewater (29,35,36).

The success of slow infiltration systems is dependent primarily on the management of the system. Design parameters such as the infiltration rate of the soil, physical and chemical soil properties, and depth to groundwater will govern, in part, the cost of a system, but proper management of the system will ensure that the groundwater quality will be maintained.

Over the past four years CRREL has been conducting extensive studies on the slow infiltration method of land treatment. Similar experiments were also accomplished at the USDA Apple Valley Test Facility in Minnesota (30) and University of Washington Pack Forest Research Facility (1).

Results from these experiments documenting plant uptake of nutrients using various forage grasses are summarized in Table 2. These data indicate that 50-80% of the nitrogen applied can be utilized by the plant during the growing season. If wastewater application is continued beyond this time, removal rates will be lower. Reed canarygrass removed the largest percentage of the nitrogen applied in the wastewater, whereas corn, because of its smaller rooting system, removed the least amount of nitrogen.

If the nitrogen loading rate is increased by increasing the amount of water applied to a site, the plant will use more of the nitrogen applied but at a decreasing rate (Table 3). This means that if nitrogen is applied at rates exceeding plant uptake, the amount of nitrogen that may enter the groundwater will increase and the potential for an adverse environmental impact will increase (38).

Table 2. Typical nitrogen uptake values for forage grasses receiving wastewater.*

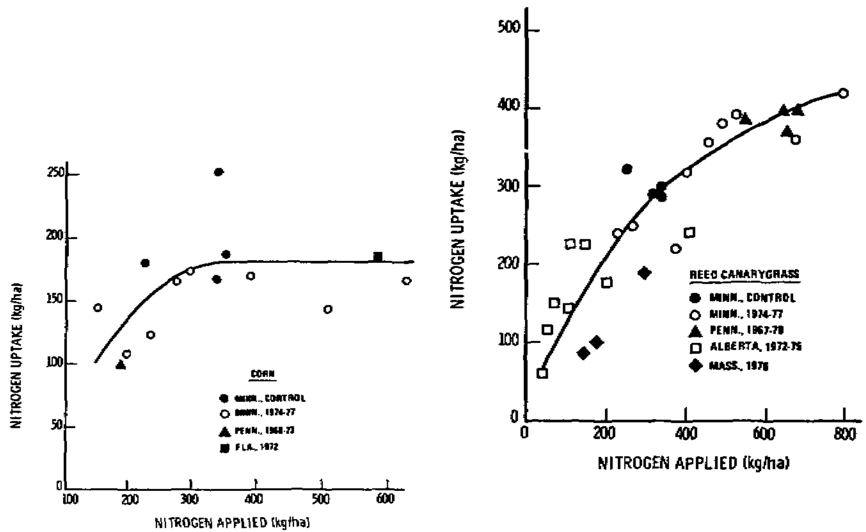
Crop	Nitrogen applied	Nitrogen removed by crop
	kg/ha	
quackgrass	410-488	238-341
bromegrass	410-458	210-267
orchardgrass	410-458	210-267
bluegrass	410-488	236-260
reed canarygrass	410-488	252-357
timothy	410-458	221-254
tall fescue	410-458	329-348
corn	337-397	169-170

*From Engineering Technical Letter "Agronomic design guidance for slow infiltration land treatment of municipal wastewater - 1978."

Table 3. Changes in crop removal for nitrogen at various application rates*.

Nitrogen applied (kg/ha)	Nitrogen in crop (kg/ha)	Percent removed (%)	Nitrogen not removed (kg/ha)
427	331	78	96
612	361	59	251
1,119	450	40	669

*From Engineering Technical Letter "Agronomic design guidance for slow infiltration land treatment of municipal wastewater - 1978."



a. Total seasonal nitrogen uptake by corn irrigated with secondary municipal wastewater effluent.

b. Total seasonal nitrogen uptake by reed canarygrass irrigated with secondary municipal wastewater effluent.

Figure 4. Nitrogen uptake vs nitrogen loading relationship for corn and reed canarygrass. *After Clapp 1978.

Results from research throughout the U.S. have been used to establish the relationships between nitrogen uptake and nitrogen applied for corn and reed canarygrass that can be used in the design of a slow infiltration land treatment system (Fig. 4). Under proper management, loading rates of greater than 600 kg/ha of nitrogen can be applied to a slow infiltration land treatment system planted with reed canarygrass and 400 kg/ha of the nitrogen can be removed by the crop. The efficiency of crop removal increases with decreasing nitrogen loading to the system.

Phosphorus at concentrations normally found in wastewater (<10 ppm) does not pose a significant problem in slow infiltration systems, if the system is properly managed. A study of many existing systems has shown that the plant and soil can virtually remove all the phosphorus contained in the wastewater for the life of the system (40). A summary of phosphorus removal by various crops at several locations is shown in Table 4 (29). The data show that a narrower range exists for P removal than N removal by crops. P removal ranged from 24 to 44 kg/ha and was not appreciably affected by increases in the application rate of P. Only about 25% of the P in normal wastewater can be taken up by the plant (38). This means that the remaining 75% of the phosphorus in the wastewater must be removed by soil adsorption and precipitation. Results from studies of existing systems that have been in operation for up to 80 years (40) have shown only trace amounts of phosphorus in the percolate water from properly managed slow infiltration systems.

Table 4. Crop dry matter yields and uptake of phosphorus for several crops at various locations as a function of application rate of secondary municipal wastewater effluent. After Clapp, et al. (29)

Crop	Location	Years	Rate	Yield	Applied	Uptake	Uptake
			cm/wk	t/ha/yr	--- kg/ha/yr ---		%
Corn*	Penn	1970	0.0	10.7	--	24	--
	Penn	1969-70	2.5	13.4	22	34	154
	Penn	1968-71	5.0	9.3	46	29	63
	Penn	1972-73	7.5	7.7	64	34	53
	Minn	1974-77	0.0	16.0	76	39	51
	Minn	1974-77	5.1	12.7	104	35	34
	Minn	1974-77	9.4	13.4	190	37	19
Reed Canarygrass†	Penn	1967-70	5.0	12.4	162	56	35
	Minn	1974-77	0.0	11.1	76	38	50
	Minn	1974-77	7.0	9.7	138	42	31
	Minn	1974-77	12.6	12.2	248	54	22
	Alberta	1972-75	3.0	5.8	27	26	96
	Alberta	1972-75	6.0	9.4	54	36	67
	Mass	1976	5.0	6.0	150	24	16
Smooth Bromegrass‡	Alberta	1972-75	3.0	6.0	27	23	85
	Alberta	1972-75	6.0	10.0	54	35	65
	Minn	1974-75	0.0	9.6	152	44	29
	Minn	1974-75	6.5	7.4	158	34	22
	Minn	1974-75	11.4	7.9	277	43	16
Timothy§	Minn	1974-75	0.0	8.8	152	34	22
	Minn	1974-75	6.5	7.6	158	32	20
	Minn	1974-75	11.4	8.6	277	40	14
Tall Fescue¶	Minn	1974-76	0.0	11.6	101	40	40
	Minn	1974-76	6.3	10.9	151	43	28
	Minn	1974-76	11.3	12.0	268	49	18
Mixed Forages**	NH	1974-75	5.0	10.2	94	32	34
	NH	1974-75	15.0	13.3	262	44	17

*Zea mays L. (Varieties: Penn - 'Pa.890-S' and 'Pa.602-A'; Minn - 'Minhybrid 4201,' 'Pioneer 3780,' 'Northrup King PX-488' and 'PX-476').

†Phalaris arundinacea L. (Varieties: Penn - not given; Minn - 'Rise,' and 'NCR-Cl'; Mass - not given).

‡Bromus inermis Leyss. (Varieties: Alberta - not given; Minn - 'Fox').

§Phleum pratense L. (Variety: Minn - 'Climax').

¶Festuca arundinacea Schreb. (Variety: Minn - 'Kentucky 31').

**Phalaris arundinacea L., Phleum pratense L. (Variety: 'Climax', Bromus inermis Leyss. var 'Lincoln'.

This indicates that the soil can effectively remove phosphorus for many years.

Although not a pollutant, potassium is a required element for crop growth. Most grasses require more potassium than any other element except nitrogen, carbon, hydrogen, and oxygen. The latter three are readily available to plants from the air. Concern over potassium limitations at land treatment sites and recommendations to fertilize these sites with potassium have been reported (32, 41, 42). Since excessive applications of potassium can alter the uptake of other elements by crops (43), discretion should be used when fertilizing. Results from experiments at CRREL have indicated that potassium imbalances were related to the N/K ratio of the wastewater applied (39). They noted that adequate potassium could be determined for the crop if the following equation were used:

$$K_f = 0.9\ U - K_{ww}$$

where:

K_f = the annual amount of potassium fertilizer applied in the spring in kg/ha

U = the estimated annual crop uptake of nitrogen in kg/ha

K_{ww} = the amount of potassium to be applied in the wastewater in kg/ha.

OVERLAND FLOW

For many years the overland flow mode of land treatment has been used for treating industrial wastes, especially from canneries. However, the use of this type of system to treat municipal wastewater is just developing (44,45,11). Results from a seven-year study conducted at Ada, Oklahoma, have shown that runoff water obtained from a site receiving raw domestic wastewater is similar to that discharged from advanced wastewater treatment plants (45). In addition, there was little difference in the runoff water quality from sites receiving either primary or secondary preapplication treatment (46).

Experiments at the U.S. Army Corps of Engineers Waterways Experiment Station have shown that, for lagoon effluent, effective treatment for reducing nitrogen at least 80 percent can be achieved at overland flow application rates of 2.54 cm applied over 18 hours during the spring, summer and fall (47). A lower application of 1.27 cm in 18 hours can be used during the winter season. Their data also show phosphorus reductions of 40 to 60%. This would mean in many instances that a pre- or post-treatment step for phosphorus removal is required.

One of the major costs of any land treatment system is a storage facility required to hold wastewater flow during inclement weather. Storage costs are especially significant in the design of land treatment systems in cold climates. Experiments conducted at the CRREL overland flow test facility in Hanover, N.H. last year demonstrated that less storage may be needed than originally estimated. Tests using primary effluent showed that BOD removal could be maintained within secondary effluent standards until mid-December. Treatment efficiency was restored by mid-April, which indicates that approximately 4-5 months of storage was needed. Many northern states require 5 to 8 months storage. The soil temperature at which BOD removal effectively ceased was 4°C (46).

The performance of the overland flow test facility is shown in Table 5. Both primary and secondary effluents were treated to <5.0 ppm nitrogen through October 1977. The data indicate that ammonium is more effectively removed than nitrate. This information suggests that using a preapplication process which nitrifies, such as extended aeration, could reduce the total nitrogen removal efficiency of the overland flow process.

Phosphorus removal ranged between 80 and 90 percent during the period from May to October 1977. However, phosphorus concentrations in the runoff averaged 2.2 ppm. Fecal coliform counts were high, indicating that disinfection prior to discharge may be required.

Table 5. Average quality of applied wastewater and runoff - May 1977-October 1977, Jenkins et al. (46).

Parameter	Applicant Concentrations Primary	Applicant Concentrations Secondary	Runoff Concentrations Primary	Runoff Concentrations Secondary
N(T) N(mg/l)	36.6	33.5	5.4 (94%)*	8.0 (87%)
NH_4 N(mg/l)	33.1	27.3	3.2	2.6
NO_3 N(mg/l)	0.5	5.1	1.6	5.2
P(T) P(mg/l)	6.3	5.9	1.9 (89%)	2.2 (80%)
BOD_5 (mg/l)	85.3	53.2	11.2 (91%)	4.6 (95%)
SS(T) (mg/l)	74.6	30.2	6.7 (97%)	3.8 (96%)
SS(V) (mg/l)	60.7	21.7	5.2	3.2
C(O) C(mg/l)	89.2	57.0	29.0	26.0
Cond (μmhos/cm)	524	519	329	324
pH (pH units)	7.4	7.5	7.7	7.6
Fecal Coliform (#/100 ml)	$7.9x10^4$	$1.8x10^4$	$6.3x10^2$	18
K (mg/l)	12.4	11.9		

* numbers in parentheses refer to mass percent removal.

CONCLUSION

Land treatment is an effective method of treating municipal wastewater without adversely affecting the environment. However, the complexity of the soil continuum and plant ecosystem makes the design and construction of these systems very site-specific. Once the system has been constructed, proper management and operation will ensure that the product water quality will meet design specifications.

Research conducted by CRREL has focused on the planning, design and management problems associated with land treatment of wastewater. It has been determined that pretreatment is necessary beyond the primary level only if the effluent requires disinfection before discharge.

Research on rapid infiltration has shown that it can be used to treat wastewater throughout the year. Management practices which maintain the soil infiltration capacity are required.

In slow rate systems nitrogen removal by the vegetation is dependent on nitrogen loading rate and plant species. Reed canarygrass can remove 400 kg N/ha per year when the nitrogen loading rate is 600 kg/ha per year, whereas corn alone can remove a maximum of 175 kg N/ha per year.

Data obtained from both slow and rapid infiltration systems have shown that the plant and soil system can effectively remove 90-95% of the applied phosphorus. Of this amount 25% can be removed by plants.

In some cases, the limiting factor for nutrient removal at vegetated land treatment sites may be potassium. The use of a simple equation can provide an estimate of the amount of supplemental potassium that is needed.

In overland flow systems, ammonium is more effectively removed than nitrate, and plant uptake accounts for more than 50 percent of the nitrogen removed. Treatment efficiency in overland flow systems is dependent on soil temperature. Wastewater should not be applied at temperatures below 4°C. Preliminary test results indicate that the storage volumes required by many states for overland flow systems may be overly stringent.

1. Cole, D.W. and P. Schiess (1978). Renovation of wastewater and response of forest ecosystems: The Pack Forest Study. International Symposium on Land Treatment of Wastewater Proceedings, Hanover, NH, August 1978, Vol. I, p. 323-331.

2. Jewell, W.J. and B.L. Seabrook (1977). Historical review of land application as an alternative treatment process for wastewaters. EPA Report 430/9-77-008.

3. Lance, J.C. and C.P. Gerba (1978). Pretreatment requirements before land application of municipal wastewater. International Symposium on Land Treatment of Wastewater Proceedings, Hanover, NH, August 1978, Vol I, p. 293-304.

4. Thomas, R.E. (1978). Preapplication treatment for overland flow. International Symposium on Land Treatment of Wastewater Proceedings, Hanover, NH, August 1978, Vol I, p. 305-311.

5. Loehr, R.C. (1978). Preapplication strategies for wastewater irrigation systems. International Symposium on Land Treatment of Wastewater Proceedings, August 1978, Vol I, p. 283-291.

6. McKim, H.L. and D.J. Lambert (1977). Deer Creek Lake - land wastewater treatment system. In Land as a Waste Management Alternative--Proceedings of the 1977 Cornell Agricultural Waste Management Conference, April, Ann Arbor Science Publishers, Inc., Syracuse, New York, pp. 79-93.

7. King, D.L. (1978). The role of ponds in land treatment of wastewater. International Symposium on Land Treatment of Wastewater Proceedings, Hanover, NH, August 1978, Vol II, p. 191-198.

8. Environmental Protection Agency (1977). Process design manual for land treatment of municipal wastewater. U.S. Environmental Protection Agency Technology Transfer, EPA 625/1-77-008 (COE EM 1110-1-501), October.

9. Levine, P.E., J.V. Olson and R.W. Crites (1978). Nitrogen and phosphorus removal after 30 years of rapid infiltration. International Symposium on Land Treatment of Wastewater Proceedings, Hanover, NH, August 1978, Vol II, p. 17-25.

10. Baillod, C.R., R.G. Waters, I.K. Iskandar and A. Uiga (1977). Preliminary evaluation of 88 years rapid infiltration of raw municipal sewage at Calumet, Michigan. In Land as a Waste Management Alternative--Proceedings of the 1976 Cornell Agricultural Waste Management Conference, 28-30 April 1976, Ann Arbor Science Publishers, Inc., Rochester, New York, p. 489-510.

11. Luley, H.B. (1963). Spray irrigation of vegetable and fruit processing wastes. Journal Water Pollution Control Federation, 35:1252-1261, October.

12. Bendixen, T.W., R.D. Hill, F.T. Dubyne and G.G. Robeck (1969). Cannery waste treatment by spray irrigation-runoff. Journal Water Pollution Control Federation, 41:385-391, March.

13. Bouwer, H. and R.C. Rice (1978). The Flushing Meadows Project. International Symposium on Land Treatment of Wastewater Proceedings, Hanover, NH, August 1978, Vol. I, p. 213-220.

14. Aulenbach, D.G. (1978). Purification of secondary effluent in a natural sand filter. Journal Water Pollution Control Federation, 50:86-94.

15. Gilbert, R.G., J.B. Robinson and J.B. Miller (1974). The microbiology and nitrogen transformations of a soil recharge basin used for wastewater renovation. Proc., Intl. Conf. on Land for Waste Management, 1-3 October 1973, Ottawa, Ont., Canada, p. 87-96.

16. Tamburini, J.U., K.D. Linstedt, E.R. Bennett and D.G. Smith (1978). Demonstration plant evaluation of an infiltration-percolation system for Boulder, Colorado. International Symposium on Land Treatment of Wastewater Proceedings, Hanover, NH, August 1978, Vol II, p. 9-16.

17. Satterwhite, M.B., B.J. Condike and G.L. Stewart (1976). Treatment of primary sewage effluent by rapid infiltration. U.S. Army Cold Regions Research and Engineering Laboratory, CRREL Report 76-49.

18. Satterwhite, M.B., G.L. Stewart, B.J. Condike and E. Vlach (1976). Rapid infiltration of primary sewage effluent at Fort Devens, Massachusetts. U.S. Army Cold Regions Research and Engineering Laboratory, CRREL Report 76-48.

19. Iskandar, I.K., R.P. Murrmann and D.C. Leggett (1977). Evaluation of existing systems for land treatment of wastewater at Manteca, California, and Quincy, Washington. U.S. Army Cold Regions Research and Engineering Laboratory, CRREL Report 77-24.

20. McPherson, J.B. (1978). Renovation of waste water by land treatment at Melbourne Board of Works Farm Werribee, Victoria, Australia. International Symposium on Land Treatment of Wastewater Proceedings, Hanover, NH, August 1978, Vol I, p. 201-212.

21. Tietjen, C., A. Bramm, N. El-Bassam and H.O. Fleer (1978). Land treatment of wastewater in Braunschweig and in Wolfsburg, Germany. International Symposium on Land Treatment of Wastewater Proceedings, Hanover, NH, August 1978, Vol I, p. 221-229.

22. Sanai, M. and J. Shayegan (1978). Water pollution control through land disposal of secondary-treated wastewater effluents. International Symposium on Land Treatment of Wastewater Proceedings, Hanover, NH, August 1978, Vol I, p. 231-239.

23. Dean, R.B. (1978). The sewage farms of Paris. International Symposium on Land Treatment of Wastewater Proceedings, Hanover, NH, August 1978, Vol I, p. 253-256.

24. Cebula, J. and J. Kutera (1978). Land treatment system in Poland. International Symposium on Land Treatment of Wastewater Proceedings, Hanover, NH, August 1978, Vol I, p. 257-264.

25. Nutter, W.L., R.C. Schultz and G.H. Brister (1978). Land treatment of municipal wastewater on steep forest slopes in the humid southeastern United States. International Symposium on Land Treatment of Wastewater Proceedings, Hanover, NH, August 1978, Vol I. p. 265-274.

26. Burton, T.M. and J.E. Hook (1978). Use of natural terrestrial vegetation for renovation of wastewater in Michigan. International Symposium on Land Treatment of Wastewater Proceedings, Hanover, NH, August 1978, Vol II, p. 199-206.

27. Walker, J.M. and Y.A. Demirjian (1978). Muskegon County, Michigan's own land wastewater treatment system. International Symposium on Land Treatment of Wastewater Proceedings, Hanover, NH, August 1978, Vol I, p. 417-427.

28. Pound, C.E. and R.W. Crites (1973). Characteristics of municipal effluents. In Proc. Joint Conference Recycling Municipal Sludges and Effluents on Land, July 9-13, Champaign, IL, p. 49-61.

29. Clapp, C.E., A.J. Palazzo, W.E. Larson, G.C. Marten and D.R. Linden (1978). Uptake of nutrients by plants irrigated with municipal wastewater effluent. International Symposium on Land Treatment of Wastewater Proceedings, Hanover, NH, August 1978, Vol. I, p. 395-404.

30. Linden, D.R., W.E. Larson and R.E. Larson (1978). Agricultural practices associated with land treatment of domestic wastewater. International Symposium on Land Treatment of Wastewater Proceedings, Hanover, NH, August 1978, Vol I, p. 313-322.

31. Urie, D.H., J.H. Cooley and A.R. Harris (1978). Irrigation of forest plantations with sewage lagoon effluents. International Symposium on Land Treatment of Wastewater Proceedings, Hanover, NH, August 1978, Vol II, p. 207-213.

32. Karlen, D.L., M.L. Vitosh and R.J. Junze (1976). Irrigation of corn with simulated municipal sewage effluent. Journal of Environmental Quality, 5:269-273.

33. Sopper, W.E. and L.T. Kardos (eds.) (1973). Recycling treated municipal wastewater and sludge through forest and cropland. The Pennsylvania State University Press, University Park.

34. Sopper, W.E. and S.N. Kerr (1978). Utilization of domestic wastewater in forest ecosystems: The Pennsylvania State University Living Filter Project. International Symposium on Land Treatment of Wastewater Proceedings, Hanover, NH, August 1978, Vol I, p. 333-343.

35. Palazzo, A.J. (1977). Land application of wastewater: Forage growth and utilization of applied nitrogen, phosphorus and potassium. In Land as a Waste Management Alternative--Proceedings of the 1976 Cornell Agricultural Waste Management Conference, 28-30 April, Ann Arbor Science Publishers, Inc., Rochester, New York, p. 171-180.

36. Clapp, C.E., D.R. Linden, W.E. Larson, G.C. Marten and J.R. Nylund (1977). Nitrogen removal from municipal wastewater effluent by a crop irrigation system. In Land as a Waste Management Alternative--Proceedings of the 1976 Cornell Agricultural Waste Management Conference, 28-30 April, Ann Arbor Science Publishers, Inc., Rochester, New York, p. 139-150.

37. Iskandar, I.K., R.S. Sletten, D.C. Leggett and T.F. Jenkins (1976). Wastewater renovation by a prototype slow infiltration land treatment system. U.S. Army Cold Regions Research and Engineering Laboratory, CRREL Report 76-19.

38. Palazzo, A.J. and H.L. McKim (1978). The growth and nutrient uptake of forage grasses when receiving various application rates of wastewater. International Symposium on Land Treatment of Wastewater Proceedings, Hanover, NH, August 1978, Vol. II, p. 157-163.

39. Palazzo, A.J., T.F. Jenkins and H.L. McKim (1978). Nitrogen/potassium relationships of forage grasses receiving wastewater. Northeastern American Society of Agronomy meeting, Storrs, Conn.

40. Iskandar, I.K. (1978). Overview of existing land treatment systems. International Symposium on Land Treatment of Wastewater Proceedings, Hanover, NH, August 1978, Vol I, p. 193-200.

41. Palazzo, A.J. (1976). The effects of wastewater application on the growth and chemical composition of forages. U.S. Army Cold Regions Research and Engineering Laboratory, CRREL Report 76-39.

42. Overman, A.R. and H.C. Ku (1976). Effluent irrigation of rye and ryegrass. Journal of Environmental Engineering Division, ASCE, 102:475-483.

43. Reid, R.L. and G.A. Jung (1974). Effects of elements other than nitrogen on the nutritive value of forages. In Forage Fertilization, Am. Soc. Agron., Madison, Wisc., p. 395-435.

44. Law, J.P., Jr., R.E. Thomas and L.H. Myers (1969). Nitrogen removal from cannery wastes by spray irrigation of grassland. 16080--11/69, R.S. Kerr Environmental Research Laboratory, Ada, OK.

45. Thomas, R.E., B. Bledsoe and K. Jackson (1976). Overland flow treatment of raw wastewater with enhanced phosphorus removal. EPA-600/2-76-131, R.S. Kerr Environmental Research Laboratory, Ada, OK.

46. Jenkins, T.F., C.J. Martel, D.A. Gaskin, D.J. Fisk and H.L. McKim (1978). Performance of overland flow land treatment in cold climates. International Symposium on Land Treatment of Wastewater Proceedings, Hanover, NH, August 1978, Vol II, p. 61-70.

47. Peters, R.E. and C.R. Lee (1978). Field investigations of advanced treatment of municipal wastewater by overland flow. International Symposium on Land Treatment of Wastewater Proceedings, Hanover, NH, August 1978, Vol. II, p. 45-50.

SYSTEMS OF WASTE WATER MANAGEMENT IN EUROPE

by

Willi Gujer[1] and Hans R. Wasmer[2]

INTRODUCTION

This paper will review the development of waste water management in Europe and discuss some typical alternative systems. All of the major European streams flow through several countries. National pollution control policy has to be in compliance with common goals of neighboring countries. Such mutual interests are usually implemented through international agreements. Some of the legal, technical, and administrative aspects, both national and international, will be presented in form of case studies.

HISTORICAL REVIEW

The history of water sanitation dates back to antiquity. Among the great documents of European water and waste technology are the aqueducts and sewers of Rome and her colonies. The Roman aqueducts were from 20 to over 80 km long and had cross sections from 0.6 to 4.5 m^2. Their total capacity was estimated at 318 000 $m^3 d^{-1}$ (84 mgd). Even the Collosseum with its capacity of 100 000 people had sufficient sanitary facilities. Rome's main sewer line, the Cloaca Maxima, is still in use.

[1]Head, Department of Engineering, Federal Institute for Water Resources and Water Pollution Control (EAWAG)
CH-8600 Dübendorf, Switzerland

[2]Deputy Director (same address)

The following centuries of mass migration and the Middle Ages brought a continuous decline of public hygiene. The situation improved again in the 16th century. There are numerous documents on public ordinances relating to refuse collection and excreta disposal for towns in Germany, the Netherlands, France, etc. Many towns had public sewers already at that time. However, wars always caused setbacks in the construction and maintenance of suitable facilities and epidemic diseases were widespread.

The reform in public sanitation started in the 19th century and had its origin in England. The re-invention of the water closet in 1810 was so successful that this facility was declared obligatory in London in the year 1850. The WC discharges were first collected in pits but their capacity was usually not sufficient; the drainoff into roadside ditches and the problems associated therewith led to the construction of sanitary sewers.

The objective of these sewers was simply to transport wastes into the next receiving water body. But the combined effects of industrial and population growth led to a continuous deterioration of rivers and other drinking water reservoirs. From 1830 to 1840 cholera epidemics caused up to 160 000 annual deaths in England.

England initiated rapid programs for public sanitation in the years between 1840 and 1880. These actions and programs were very successful and had a great influence on the rest of Europe.

Some of the highlights of that period are mentioned below.

- 1836: Establishment of a national agency for health statistics.

- 1958: "Local Government Act". Direct discharge of waste waters are forbidden if other interests are affected.

- 1861: Legislation asking for waste water treatment prior to discharge into rivers.

- 1865: Establishment of the first Royal Commission on Pollution Control. The "Sewage Utilization Act" authorizes the communities to acquire land for treatment facilities outside of city limits.

- 1872: "The Public Health Act" enables communities to form inter-community interest groups for regional pollution control.

- 1876: Introduction of the "Rivers Pollution Prevention Act", the first comprehensive legislation on water pollution control.

The sanitation programs and the technologies developed in England set the standards for European practice in sanitary engineering. This trend continued well into the first half of the 20th century. Although the transfer of technology was very successful, the accomplishments of pollution control were not everywhere as outstanding as in England. Some of the scientific and technical reasons for this development are listed below: England is an island with relatively short water courses. (Some people track the BOD_5 philosophy back to the fact that the water in Great Britain's streams enters the Ocean in less than 5 days.)

The waste characteristics changed from wastes of human metabolism to wastes with a significant amount of chemical and industrial substances. Pollution control was (and sometimes still is today) looked at as a technical problem only and thus was put in the hands of engineers. Consequently, the orientation of pollution control was rather on practicable technology than on water quality objectives of the receiving water body. The problems of water pollution control in continental Europe with its multinational streams can best be shown by some representative data published by Stumm (3).

TABLE 1 - Parameters of Pollution Load for Some Streams

	Population Density cap/km^2	Population relative to water flow cap/m^3 sec	GNP relative to water flow $\$/m^3$
Rhine	140	15 000	3.4
Donau	83	10 400	1.1
Rhône	63	3 700	0.55
Ohio	76	5 800	1.3
Mississippi	19	3 300	0.75

LEGAL ASPECTS OF WASTE WATER TREATMENT

Examples of a National Approach

Germany - Waste water charges law. - In the Federal Republic of Germany the discharge of any storm- and waste water is subject to waste water charges. The charges are determined by weighing different fractions of the pollution load (settlable solids, COD, Hg, Cd, fish toxicity). Every damage unit- the approximate equivalent pollution load of one inhabitant - disposed of in a receiving water is levied with a discharge fee of DM 12.- in 1981 and DM 40.- in 1986 per year. Dischargers who apply at least waste water treatment according to "accepted rules of technology" pay only 50 % of the amount due for the residual load. Accepted rules of technology at present seem to be secondary treatment.

The Laender (States) collect the waste water charges and will use them to finance additional treatment (private or public) according to cost/benefit analysis. It is expected that this will help to provide treatment up to a limiting cost of approximately DM 70.- per year for every damage unit removed.

Waste water charges which are uniform throughout the nation will hardly be the adequate tool to solve every local problem. These charges will however speed up the construction of treatment plants for major polluters, and they will help the federal government to fulfill its international obligations in the large rivers.

In addition to the waste water charges discharge standards have to be formulated by the Laender (States) authorities in the discharge permissions. These standards may be set based on local conditions and therefore are to some extent based on receiving water quality.

Switzerland - Federal ordinance for waste water discharges. - In Switzerland the federal law for water pollution control does not allow a classification of receiving waters according to local conditions. The federal ordinance for waste water discharges specifies the desired quality for all rivers and lakes. These specifications describe a river which is hardly affected by waste water discharges.

From all industries discharging to public sewers - the dominant practice - pretreatment is required to the same quality levels, independent of industrial branch. Discharge to a receiving water is subject to a variety of water quality parameters (52 physical and chemical parameters are specified - the most important ones are summarized in Table 2). If the goal of receiving water quality cannot be met, local discharge standards have to be adjusted based on regional considerations.

Public waste water treatment plants may partially be subsidized by the federal environmental protection agency. The underlying ordinance requires biological treatment in order to make funds available. The standard discharge requirements (Table 2), which are applicable throughout most of the country, are such that they can usually be met with conventional mechanical biological treatment processes. These standards together with subsidies usually result in the choice of the activated sludge process. A recent survey of the waste water and sludge treatment processes used in Switzerland (2) gave the results indicated in Table 3.

Approximately 30 treatment plants that discharge directly to receiving waters are owned and operated by industrial complexes. The general requirement, to treat industrial and trade effluents combined with domestic sewage, resulted in the fact that approximately 10 000 firms discharge their pretreated waste water to public sewers.

TABLE 2 - Some discharge standards of the Swiss federal ordinance for waste water discharges

PARAMETERS	UNITS	EFFLUENT STANDARD		RECEIVING WATER STANDARD
		CONC. 80% of time	EFFICIENCY % 3)	95% of time
Total Suspended Solids	mg/l	20	-	-
BOD_5	mg O_2/l	20	> 85	< 4
Dissolved organic Carbon (0.45 μm)	mg C/l	10	85 2)	2
	mg C/l	15 1)	75 1) 2)	
Total organic Carbon (TOC)	mg C/l	17		-
	mg C/l	22 1)		
Total Phosphorous	mg P/l	1	> 85	lakes only
Ammonium	mg N/l	-	-	< 0.5
Ammonia (NH_3)	mg N/l	-	-	< 0.1

1) can only be applied if the particular waste water does not cause significant damage.

2) applicable if primary effluent TOC exceeds 65 mg C/l.

3) with respect to primary effluent.

TABLE 3 - Waste water treatment and sludge handling processes utilized in Switzerland (Jan 1st 1977). Very small plants are excluded.

Total number of plants 626 (100%) Total hydraulic capacity 10^7· population equivalents (100%)		Number of Plants	% of Plants	% of hydraulic capacity
Waste Water Treatment	Primary treatment	58	9	1
	Primary plus secondary treatment			
	activated sludge	431	64	91
	trickling filters	122	18	6
	rotating discs	65	10	2
	Primary plus secondary treatment including P removal			
	operating	117	17	27
	operating soon	140	21	19
	total	257	38	46
Sludge Handling	Digestion anaerobic	406		
	aerobic	108		
	Pasteurisation	91		
	Dewatering mechanical	72		
	Composting	39		
	Incineration	26		
	agricultural use	726		
	Landfill	66		

The federal agency has the competence to selectively subsidize technical installations in which important point sources are reduced. Nevertheless the Swiss system lacks the necessary tools for the implementation of efficient pollution control systems on a regional basis.

Examples of an International Approach

Lake Constance - Technical guidelines. - Lake Constance is a prealpine lake in the system of the river Rhine. It has a total surface of 540 km^2 and its border countries are the Federal Republic of Germany, Austria, and Switzerland. The International Commission for Lake Constance issued the first technical guidelines in 1967 which later were amended in 1972.

These guidelines are an example of the concept of uniform technology. By means of standard design parameters for treatment facilities and standard effluent concentrations all final discharges to the lake should have the same characteristics.

Such an approach is ideal if all waste sources are of the same quality and are distributed uniformly around the lake. However, if dominant waste sources are present, the aspects of scaling-up are rather important and the overall efficiency of the concept of standard technology should be evaluated carefully.

International agreements for the protection of the Rhine. - The "International Commission for the Protection of the Rhine against Pollution" was put into action in 1950. Member countries of the Commission are the Federal Republic of Germany, France, Luxembourg, the Netherlands, and Switzerland. Because the "European Community" (EC) carries out regulatory duties concerning chemical pollutants, the EC is also an official member of the Commission.

The reduction of the chloride loads had top priority after the Commission was put into action. High chloride concentrations cause great difficulties for the Netherlands as the Rhine is the major reservoir for drinking water supply and irrigation. The common goal is to reduce the Cl^- loads to such an extent that concentrations at the German/Netherlands border should not exceed 200 mg Cl^-/l. Today's concentrations are as high as 300 mg/l. Obviously there are numerous sources of chlorides in the watershed area of the Rhine, and most of these sources are diffuse. However, there is a dominant point source: the potassium mines in the Alsace. In 1972 it was agreed upon to reduce the chloride load from these potassium mines by 60 kg/s and to deposit the waste chlorides on land. The costs for the necessary installations were first estimated at 100 million French Francs, but a detailed study showed that the effective costs would be several times as high.

A new project was thus developed by which the chloride loads would be reduced gradually. In a first step a load reduction of 20 kg/s will be carried out during a 10 year experimental period. The total costs are estimated at 132 million French Francs of which France will pay 30 %, Germany 30 %, the Netherlands 34 %, and Switzerland 6 %. The treatment facilities should be in operation within 18 months after ratification of the agreement.

The remaining loads will be reduced by two 20 kg/s increments. Engineering design of these additional installations should be finished by 1980.

The agreement includes also a so called "standstill clause": For each member country the maximum increment of discharge within a certain distance of the river is fixed. The results of continuous monitoring are published every six months.

ALTERNATIVE SYSTEMS - CASE STUDIES

Expansion of the Waste Water Treatment Plant for the City of Zürich (Switzerland)

The City of Zürich is the most important point source for organic, phosphorous, and nitrogen pollution in Switzerland. The existing treatment plant does not meet the prescribed effluent standards by far and therefore has to be expanded. Because of severe site limitations a process was sought which would make use of unconventional space saving unit operations.

An international competition for the design of the future process was opened. Six working teams, each composed of international and local engineering firms, were invited to compete. The proposed processes had to remove biodegradable as well as refractory organic material, phosphorous, ammonia, and total nitrogen. A short summary of the proposed solutions is given in Table 4 (4).

From the results of the competition the City of Zürich concluded that none of the proposed process combinations could be

TABLE 4 - Process combinations proposed for expansion of the City of Zürich treatment plant

PROJECT NR.	PROCESS COMBINATION
1	Sedimentation - Nitrifying activated sludge - Flocculation/Precipitation - Microflotation.
2	Sedimentation - Nitrifying activated sludge (Simultaneous precipitation, pulverized activated carbon)- Denitrification filter - Ozonation.
3	Sedimentation - Flocculation/Sedimentation - Filtration - Breakpoint-chlorination - Activated Carbon Filtration.
4	Activated Sludge (BOD) - Activated Sludge (Flotation, Nitrification) - Denitrification (Plastic media) - Flocculation/Precipitation - Lamella Separation.
5	Sedimentation - Trickling Filter (roughing filter) - Activated sludge (BOD, Simultaneous precipitation) - Nitrifying activated sludge - Denitrification filters - Multi media filtration.
6	Sedimentation - Activated sludge (air, O_2, simultaneous precipitation) - Nitrifying activated sludge (O_2) - Denitrification filter - Chlorination - Activated carbon filters.

accepted without major changes. Close inspection of the most interesting solutions (Nr. 4,5,6 in Table 4) revealed however that biological nitrification in context with the low load activated sludge process and integrated phosphorus removal (simultaneous precipitation) in combination with final effluent deep bed contact filtration could solve the present day problems. In view of the results of the competition these processes would be necessary in future treatment installations.

A large scale pilot study was initiated to yield design information for biological nitrification under winter conditions and phosphorus removal in the combination of simultaneous precipitation and tertiary filtration. Based on this pilot study the process indicated in Fig. 1 was adopted and recently SFr.$223 \cdot 10^6$ (equ. approx. US \$ $100 \cdot 10^6$) were granted in a public vote by an overwhelming majority of 19:1.

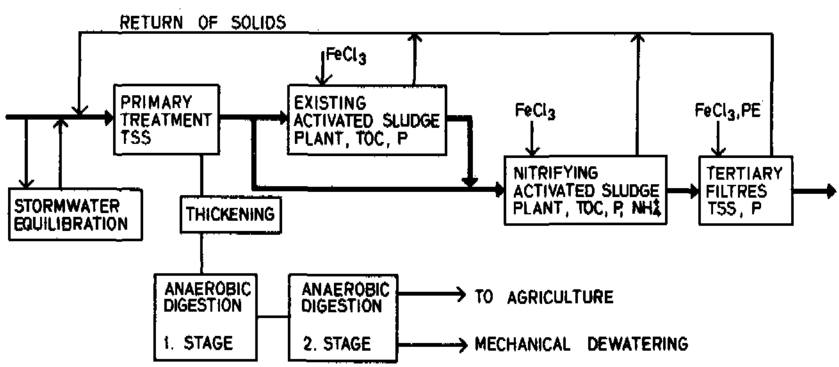


FIG. 1 - Proposed waste water treatment process for City of Zürich

It appears that the City of Zürich provided the ideal basis for the implementation of unconventional process alternatives: one would expect that an international competition, severe effluent requirements, site limitations and a heavily overloaded existing plant do not necessarily call for a conventional process combination. Surprisingly, the resulting solution however does not indicate any important deviations from generally accepted standard processes in Switzerland.

Deep Well Disposal of Brine Solutions in France

Reference is made to the potassium mines in the Alsace (France). As already mentioned in connection with the International Rhine Commission waste chlorides in the amount of 20 kg/s (ultimately 60 kg/s) will no longer be discharged into the Rhine.

These chlorides will be deposited in the underground at a depth of 1500 to 2000 meters in oölitic rocks. The concept is rather simple. The connate water contained in this geological layer is displaced by concentrated brine solutions. The system can briefly be described as follows: 1) Installations for the production of concentrated brine solutions, 2) storage basins, 3) pipelines from storage basins to discharge wells, 4) discharge wells, combined gravity and pressure systems, 5) wells from which the connate water is drained off, 6) pipelines from these wells to the storage basins, 7) remote control for pumps and pipelines.

Phosphorous Removal from River Water in Germany

The Wahnbach reservoir in Germany (FRG) is the source of drinking water for the Bonn-Siegburg area. In this reservoir phosphorous is the limiting nutrient. Ever increasing eutrophication in the reservoir caused more and more problems in the production of drinking water from this raw water source. The catchment area of the reservoir is dominantly agriculturally used (63% of area), the population (8000) is spread out in small villages.

A detailed investigation of the nutrient sources revealed that approximately 60% of the total phosphorous load was due to diffuse sources (mainly agriculture) and could not be reached with a point source program. In addition low density housing would make point source programs inefficient and expensive.

The main influent to the reservoir contains approximately 90% of the total phosphorous load. In connection with a flow equilibration basin for the influent a generously designed phosphorous removal process for the entire river can therefore possibly remove a substantial amount of the total phosphorous load.

Pilot tests indicated that precipitation/flocculation with Ferric in combination with deep bed filtration would result in very low total phosphorous and turbidity concentrations in the river (from 80 µg/l total P in the influent to less than 10 µg/l total P in the effluent) (2).

In 1978 the full scale plant with a maximum capacity of 4 m^3/sec (90 MGD) will be taken into operation to treat more than

95% of the yearly flow of the main influent. The plant which operates according to Fig. 2 is hoped to reduce total phosphorous load on the lake from a present 2 g/m^2-yr down to less than 0.3 g/m^2-yr.

Even though the operating costs to remove 1 kg of P are in the order of DM 460.- (US $ 200.-) it appears that for this particular problem a reasonable solution has been found. Source programs alone could never be as efficient as direct treatment of the river.

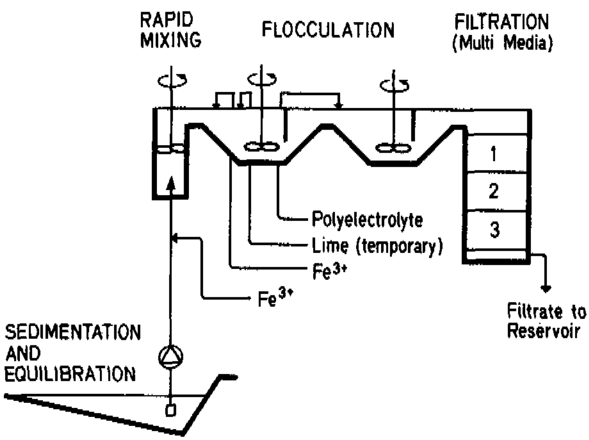


FIG. 2 - Treatment Process for River Water (Wahnbach reservoir)

TRENDS IN EUROPEAN WASTE WATER TREATMENT

Financing and Risk Coverage for New Technologies

In Europe a large fraction of industrial and trade effluents is treated together with domestic waste water. Therefore, waste water treatment is financed mainly by public money. This slows down the implementation of new technologies: High risk projects are seldom paid with public money.

Several countries provide the possibility of additional financing, fast depreciation, low interest loans or risk coverage for the construction of innovative processes. In Switzerland a federal risk guarantee is established. The "general water pollution control ordonnance" of the federation says:

> If the application of innovations promises success and is in the general interest of water pollution control, federal subsidies can be supplemented with a risk coverage if no guarantees by delivering firms can be obtained.

Not more than 55 percent of the total costs may be covered. Additional support by the Cantons (States) should be provided.

In the years 1973-1978 this risk coverage was not used more than once per year. Examples of processes funded are the first pure oxygen activated sludge plant in Europe and the first tertiary filters for phosphorous removal in Switzerland. In view of the several hundred treatment plants built in Switzerland during the same period, this is a rather disappointing result.

The problem to the slow application of new technologies is partially due to the engineering community itself. Traditionally civil engineers were in charge of the design of waste water treatment processes. Engineers who could approach this type of process design from a scientific point of view have only been trained in the last decade. The decision making generation has only empirical experience to a large extent.

Water Basin Management

Throughout Europe a trend can be observed towards integrated water basin management.

In 1964 France was divided into six independent river basins. Each basin has its own form of management suitable to its particular characteristics and economic requirement.

In England and Wales the water act of 1973 provided for ten regional water authorities. This reorganisation made one authority responsible for many aspects of water management in a river basin: Water supply, prevention of pollution, water resources conservation and development, sewerage and sewage disposal, fisheries and land drainage.

For an important part of Germany (FRG) water associations covering entire river basins existed for a long time mainly in highly industrialized areas. Well known examples are the Ruhrverband, the Emschergenossenschaft etc. Federal legislation requires the Laender (States) to prepare regional plans with regard to water management. Detailed plans for the improvement of the water quality in entire river systems exist for example for the river Neckar (1977) and the river Donau (1978)., Simulation of river water quality with regard to organic compounds are the basis for these two plans.

In Switzerland discharge standards which require advanced technologies may be set for entire river systems if the receiving water quality does not meet the legal requirements. Advanced phosphorous removal (tertiary filtration) and nitrification have been required based on regional considerations.

It appears that in many countries the historical emissions control (specification of technology to be applied) slowly evolves towards emission control. A large number of mathematical modelling and simulation projects for river basins throughout Europe will help to define immission oriented discharge standards.

CONCLUDING REMARKS

European waste water management is not characterized by extraordinary innovation in general or rapid changes in legislation and administration, nor by a lot of technological breakthroughs. It can be shown that the accepted rules of technology lay approximately 20 to 30 years behind the findings of present research. A typical example is eutrophication control by phosphorous removal. The various steps in the transfer of progress from research, pilot studies, large-scale demonstrations, legislation, funding, acceptance and know-how of the engineering community did take close to 30 years as mentioned above.

However, waste water management which is not only oriented towards treatment but carried out in a broader perspective of ecological considerations gained considerable momentum during the past 10 years. The classic approach to pollution control by applying standard technology was certainly adequate in the initial phase. There are many examples, however, where so-called "complete" sanitation by accepted standards of technology did not bring the desired results. The need for priority programs became evident. It can be expected that the above mentioned lag time between research and practical application will be shortened in the future. The past few years of recession and the continuous shortage in public money may even accelerate this trend. It was a shock for many executive agencies that with less money available, they did not have a set of criteria to define priorities nor did they have the necessary instruments for carrying out priority programs.

The major streams in Europe flow through several countries. Thus water pollution control and environmental protection in general cannot be dealt with on a national basis alone. The ever decreasing quality of the major European streams brought enough pressure for international cooperation and action. The experience gained might be useful for further cooperation if we want to protect the entire biosphere and the oceans in particular.

REFERENCES

(1) Bernhardt, H., Clasen, J., and Schell, H., Phosphate and Turbidity Control by Flocculation and Filtration, Jour. AWWA, 63, 355-368 (1971).

(2) Boller, M. and Conrad T., in "Gewässerschutz in der Schweiz", Gas-Wasser-Abwasser, 57, 766-779 (1977).

(3) Stumm, W., "Die Beeinträchtigung aquatischer Oekosysteme durch die Zivilisation", Naturwissenschaften, 64, 157-165 (1977).

(4) Wiesmann, J. and Roberts, P.V., Ergebnisse eines internationalen Wettbewerbes für die weitergehende Abwasserreinigung auf der Kläranlage Zürich-Werdhölzli, Berichte der ATV, Heft 28, 259-284 (1976).

ABSTRACT

SYSTEMS OF WATER MANAGEMENT IN EUROPE

KEY WORDS: Environmental engineering; foreign engineering; government agencies; legislation; management; waste disposal; water pollution; water resources.

ABSTRACT: The history of European waste water management is briefly reviewed. Legal aspects of waste water management are presented by examples from Germany and Switzerland. International agreements on the river Rhine and lake Constance are discussed. Case studies of alternative systems are presented for the waste water treatment plant in Zürich, deep well disposal of brine solutions in France, and treatment of an entire river in Germany. Trends in pollution control policy, technology, and management are outlined.

REFERENCE: Gujer, W., and Wasmer, H.R., "Systems of Water Management in Europe", ASCE Annual Convention 1978, Preprint paper.

MEASURING THE EFFECTS OF MAN'S WASTES ON THE OCEAN

Willard Bascom*

INTRODUCTION

Understanding the ecological effects of municipal wastewater discharged into the ocean depends on data developed by reliable measurements. Municipal wastes are largely residential sewage flavored with industrial effluent; the amount and toxicity of the latter depend, of course, on the kind and amount of industry and the local regulations. Usually this mixture is given some level of treatment in a sewage plant at which time some of the solids (grit and sludge) are removed. The remainder may be treated by aeration, bacteria, addition of various chemicals, etc. until the final effluent is a thin but complex soup containing many fine particles. Upon discharge out the deep diffuser outfalls used in California, this liquid is immediately diluted by at least 100 to 1 forming a plume which drifts off to sea with the prevailing currents.

These discharges have very little, if any, effect on man but they do have some effect on life in the sea. Now the question arises which makes this a controversial subject. Are these effects

---

*Director, Southern California Coastal Water Research Project

serious enough to require a major effort or are they of no more importance than the excavation of a building, the plowing of a field, or the concreting over of land to make an airport? Some believe that the waste disposal should be treated like these other projects with benefits algebraicly added to alleged damages and value received compared with costs. The ultimate question is: How can our society best make rational, unemotional decisions on such matters?

This paper summarizes how measurements can be made that not only define the present situation but which can be used as a basis for forecasting what future effects would be if certain changes were made in the character of the wastes discharged. These techniques have been very useful in the open coastal waters of California and probably they will be effective in estimating the effects on similar waters elsewhere.

The reader is also reminded that pollution means that a "damaging excess" of one or more materials is present. Change may or may not be evidence of damage. The amount of change in an area where pollution is suspected should be compared with the natural variations in pristine areas; often the investigator is surprised by how large the normal variability is.

MEASUREMENTS

There are several categories of things to be measured. In order of usage these are: (1) Those aspects of the ocean's shape and motion that are important to disposal, (2) the effects caused by man's construction that may be influential, (3) the nature of the material to be disposed of and the dilution at release, (4) the effect of wastes on the water, the bottom, and the sea animals.

Although this paper is primarily concerned with the last of these, each one contributes something useful to an overall understanding of the effects of municipal wastes on coastal waters. Let us consider each in turn.

(1) The physical situation is the backdrop of conditions against which effects can be measured. The information required, whether from existing data or from new measurements, includes the topography of the nearshore bottom and the exposure of the site to open ocean waves and currents. Generally, headlands (or the seaward side of islands) with steep offshore gradients are preferable for discharge because refraction by undersea contours causes wave action to be greater; morover, ocean currents are likely to be stronger. Wave statistics from wave meters and hindcasts are needed and current velocity/direction at several depths should be known for at least a year. The material of the bottom which ranges from hard rock through sand to silt and mud should be determined as it is likely to be important to construction as well as to subsequent chemical or biological measurements.

Much of the year the ocean is stratified with a relatively warm mixed surface layer of water extending 60 to 200 feet beneath the surface. Then there is the matter of turbidity, which usually is caused either by a great influx of plankton (tiny living creatures) or by storm waves roiling the bottom. Thus, temperature and clarity with depth are needed. All the above can be measured with well known instruments to obtain a reasonable picture of ocean conditions at a disposal site.

(2) The effects caused by the installation of a large structure such as a pipe may have some local importance. If the pipe is 10 feet or so (3 meters) in diameter, partly buried in a trench, and extends several miles offshore (as is common in southern California) it causes certain changes in sea life. The pipe may form a barrier to the movement of certain kinds of animals which will be more numerous on one side. Because it offers a hard rock-like substrate, sea life such as algae and colonial animals grow profusely on it. These in turn help attract fish and crabs that like the protection combined with food from the outfall. The material originally excavated for the pipe trench is now spread on the bottom and the addition of fist-sized ballast rock offer two other new variations in habitats that attract other sea animals. Changes in the sea life as it adapts to this locally changed environmental situation is best studied by direct examination with cameras or television.

(3) The material to be disposed of is best measured just before it is discharged and most treatment plants dutifully combine hourly samples into a daily composite and run it through a chemistry laboratory to satisfy state regulations. This kind

of measurement is useful for some purposes but other measurements may give a better picture of what actually happens after ocean discharge.

For example, one of the most important factors is the settling velocity of the waste particles after they are mixed with and diluted by seawater. Because of a combination of chemical and electrostatic effects, the particles discharged may either agglomerate or break up when mixed with seawater. After this mixing, the density and settling velocity changes and some fraction of the particles fall much faster than the rest. Those that settle more rapidly than about one centimeter in 100 seconds have a reasonable chance of landing on the bottom near the discharge point (within a kilometer or so). This reconcentration of sewage material can cause a problem. However, the more slowly settling material is so widely dispersed it cannot be concentrated anywhere. The animals of the bottom gladly utilize the waste particles if they are not overwhelmed by them.

We find it useful to perform two experiments under carefully controlled laboratory conditions. In the first wastewater is mixed with seawater at the minimum initial dilution ratio (usually about 100 to 1) in a settling tube and the amount of deposition of particles over a period of 2 hours is measured. This gives the volume of material that settles faster than 0.01 cm/sec and remains within 2 kilometers of an outfall.

A similar experiment using somewhat larger volumes of fluids at lower dilutions is used to determine the amounts of contaminants on the material that will settle to the bottom within 2 km of the discharge point. That is, the wastewater and seawater are mixed at 30 to 1, allowed to settle for 2 hours, the water decanted off and the settled solids collected. These are then subjected to chemical analyses that make it possible to determine the amounts of possible pollutants on the material that settles nearby; from this possible toxic effects can be forecast.

Finally, the actual dilution achieved must be measured in the sea to confirm the theoretical calculations and model experiments. This requires a precise navigation device such as Loran C which gives the boat or sample position within about 60 feet (20 meters). We usually lower a Tygon hose with an electrically driven pump on the lower end (total length 300 feet or 100 meters) and pump water from 6 to 10 levels. By thoughtfully selecting sample stations (based on temperature profiles and current drogue movements) it is possible to make measurements that define the waste field and the levels of minimum initial dilution. The values come from measuring the water pumped aboard as it flows through a nephelometer (for turbidity), or a fluorometer (for optical brighteners) or by measuring ammonia levels, or by measuring fluorescence of added Rhodamine WT dye (which does not stick to particles). The results from these various methods usually confirm each other; the maximum concentrations are deemed to be representative of minimum initial dilution.

(4) The most important effects of wastes are on the bottom and the sea animals.

Municipal waste effluents are a very thin soup that usually has between 20 and 200 milligrams per liter of solids. The bulk of the discharge is material in solution or in very tiny particles (under 4 microns). Off California, the original concentration after average initial dilution is reduced by a factor of about 200 and only the most careful sampling in large, super-clean containers, and accurate analysis (to tenths of parts per billion) can detect discharged material in solution beyond 10 kilometers from the outfall. Except for the measurements noted in the previous section we have not found chemical measurements in the water to be helpful in understanding the effects of outfalls.

We were able to collect particles in the water a kilometer or two from a large outfall by hanging bags of mussels (Mytilus californianus) from taut-moored buoys at wastefield depth and letting these animals do the collecting. Mussels are filter feeders and a medium sized one (7 cm) will pump about one liter of seawater per hour. The particulates are removed by the animals and measured when the mussel is taken. This is an unusual technique, useful for special purposes because contaminants, including bacteria and viruses, tend to be attached to the particles that the mussels collect.

The muddy bottom that exists around outfalls that discharge several miles offshore is where effects are most readily measured. The best tool for bottom sampling is a chain-rigged Van Veen grab which reliably snaps up a virtually undisturbed sample of one-tenth of a square meter (Word, 1974) Because sedimentation is a slow process it is important to separate the recently (last year or so) deposited material from the older deposits. Therefore, we sub-sample part of the surface of the material still in the grab to a depth of 2 cm. This sub-sample is returned to the laboratory for analysis for volatile solids or total carbon, BOD, COD, 8 metals, and chlorinated hydrocarbons.

Results from samples like the above are most easily interpreted if they are taken on an elongated grid laid out parallel to depth contours. This is because the currents which distribute the discharged material tend to flow along lines of equal depth. Depending on the size of the discharge and other matters some 15 to 30 grab stations can be arranged to adequately define the outfall effect area. A sample interpretation is given in Table 1.

Some of the sea animals are sampled by the same grabs described above. These are the benthic infauna, small creatures that live in the bottom mud and do not move substantial distances. Ecologists are particularly interested in them because they must live and reproduce in the areas most likely to be affected. They are most likely to react to whatever is discharged.

When the grab is brought to the surface it is discharged onto a one mm screen; the mud is washed through leaving the animals which must then be sorted, counted and identified. Such work is fairly routine up to a point. But deciding how many samples to take, where to draw the line at identifying rare animals and how to process the data are important biological management decisions. After careful work on thousands of samples we have reached some important decisions that should be helpful to everyone facing similar problems because they give more usable information in a shorter time and with lower costs.

We do not take biological replicate samples at any station, preferring instead to sample at more locations. We identify only 26 taxa (Table 2) that indicate the response of animals to the environment. We save the rare one-of-a-kind for some scholar who has more esoteric objectives. We do not subject the data to complex mathematical manipulations but use the simple "Infaunal Index" which is derived as follows: The 25 taxa (species) are sorted into 4 feeding groups that range from animals that live only in clean water (mostly suspension feeders) to animals that prefer organic materials in the bottom (mostly deposit feeders). Then the total number of animals in each group is used in a simple formula. Infaunal Index = $100 - 33.3 \left(\frac{n_2 + 2n_3 + 3n_4}{n_1 + n_2 + n_3 + n_4}\right)$ in which n is the number of animals in each group. The index number falls between 0 and 100 and is consistent for mud bottoms in depths of 20 to 200 meters. We find that indices higher than 80 are characteristic of areas undisturbed by man and which are low in organic solids.

This brings us to the final step which is to assess the biological situation and communicate the condition of the animal communities to other persons in an understandable form. Now each sample point can be assigned 3 simple numbers that represent the main characteristic of that location. We also show the biomass (in grams per square meter) for the same location and the percent of voltile solids. When the infaunal index number drops because of the presence of excess organic materials the biomass usually goes up. These numbers can be used to define the area affected by an outfall.

The last important measurement is that of the larger sea animals that live on and just above the bottom. We use two techniques for determining their species and numbers; the bottom trawl and the baited camera. These do not give the same result but the answers are complementary. The baited camera was devised to study animals in rough or rocky areas where trawls could not be operated.

Baited cameras come in various forms but we routinely use a 35 mm, wide angle lens, 250 exposure Minolta. It is enclosed in an aluminum case that will stand about 100 atmospheres of pressure and mounted on a pipe frame shaped like the edges of a 2 meter cube. The camera's pressure housing also holds batteries, condensers, and timing circuits that operate the camera and two stroboscopic lights in synchronization. The light level is sufficient that we can use Kodachrome (ASA 40) at f 11. The bait is usually very dead squid dangled on a line about a meter in front of the lens. We usually take one frame every 3 minutes for the half hour on station (while grabs are being taken). The result is about 10 high quality slides per station that can be viewed later by biologists of several disciplines.

The otter trawl is a net that is dragged along mud or sand bottoms while being held open by a pair of otter boards. This net scoops up the animals that are on and just above the bottom such as crabs, urchins, starfish, bottom fish and rockfish. Standard trawls of 10 minutes on the bottom at 2 knots (about 1 meter per second) are customary. The net is retrieved and the catch is sorted, identified and counted on deck. Each fish is measured for standard length and examined for signs of disease or parasites. Then all are returned to the sea; only data is brought home.

The analysis and comparison of fish and large invertebrates does not give as lucid results as the infauna because the season, the water temperature, the entrance of great numbers of juveniles into the system and an invasion by predators all contribute to the statistical noise. Nevertheless, it is possible to make useful comparisons of animal communities that will show outfall effects if there are any. Examples are shown in Figures 1 and 2.

SUMMARY AND CONCLUSIONS

The devices and methods described have been successfully used on repeated occasions and are probably the best available for the assessment of the effects of man's wastes on the ocean. It is no longer necessary for the EPA or local pollution control agencies to rely on vague allegations about the biological conditions around outfalls. These can be readily measured and reduced to simple numbers that can be understood by anyone who is interested.

Table 1. Forecasts of chemical and biological changes in the sea based on a proposed change in effluent starting January 1, 1983.

	Natural background range	Orange County Stations B0 to B5				
		Present 1977	New effluent 1983	After 1 year 1984	After 5 yrs. 1988	1993
% Organic carbon in sediment	0.5-2.0	1.7	1.8	1.5	1.3	1.3
Escess metals in sediment (mg/kg)						
Silver	0.3	0.2	0.2	0.16	0.12	0.13
Cadmium	0.2	1.2	1.2	0.72	0.26	0.22
Chromium	23.0	17	17	13	8.5	8.2
Copper	7.0	29	29	18	8.3	7.7
Infaunal Index Numbers	75-80+	45-50	45-50	60	65	75
Total lineal distance to background conditions		8km	8km		0.5	
Solids discharge metric tons/yr		25,000*	8,940	8,940	8,940	
Excess standing crop of benthic organisms (metric tons)		549	848	280	110	89

*Rising to 32,300 just before changeover.

Table 2. Taxa for calculation of Infaunal Index numbers

Group 1 - Suspension Feeders (pristine area)

Phoronis spp.	Phoronid
Amphiodia spp.	Brittle star
Sthenelenella uniformis	Worm
Ampelisca spp.	Amphipod
Paraphoxus spp.	"
Heterophoxus oculatus	"
Metaphoxus frequens	"

Group 2 - Mixture of Feeding Strategies

Mediomastus spp.	Worm
Myriochele spp.	"
Tharyx spp.	"
Axinopsida serricata	Clam
Mysella spp.	"
Photis spp.	Amphipod
Euphilomedes	Ostracod

Group 3 - Deposit Feeders, mainly molluscs

Parvilucina tenuisculpta	Clam
Macoma carlottensis	"
Bittium spp.	Gastropod
Spiochaetopterus costarum	Worm

Group 4 - Deposit Feeders, mainly worms (outfall area)

Armandia bioculata	Worm
Shistomeringos longicornis (=*Stauronereis rudolphi*)	"
Ophryotrocha sp.	"
Oligochaeta, UI	"
Capitella capitata	"
Dorvelleidae, UI	"
Stenothoidae, UI	Amphipod
Solemya panamensis	Clam

$$\text{Infaunal Index} = 100-33.3\left(\frac{n_2 + 2\,n_3 + 3\,n_4}{n_1 + n_2 + n_3 + n_4}\right)$$

where n = the number of animals counted in a sample from the appropriate group.

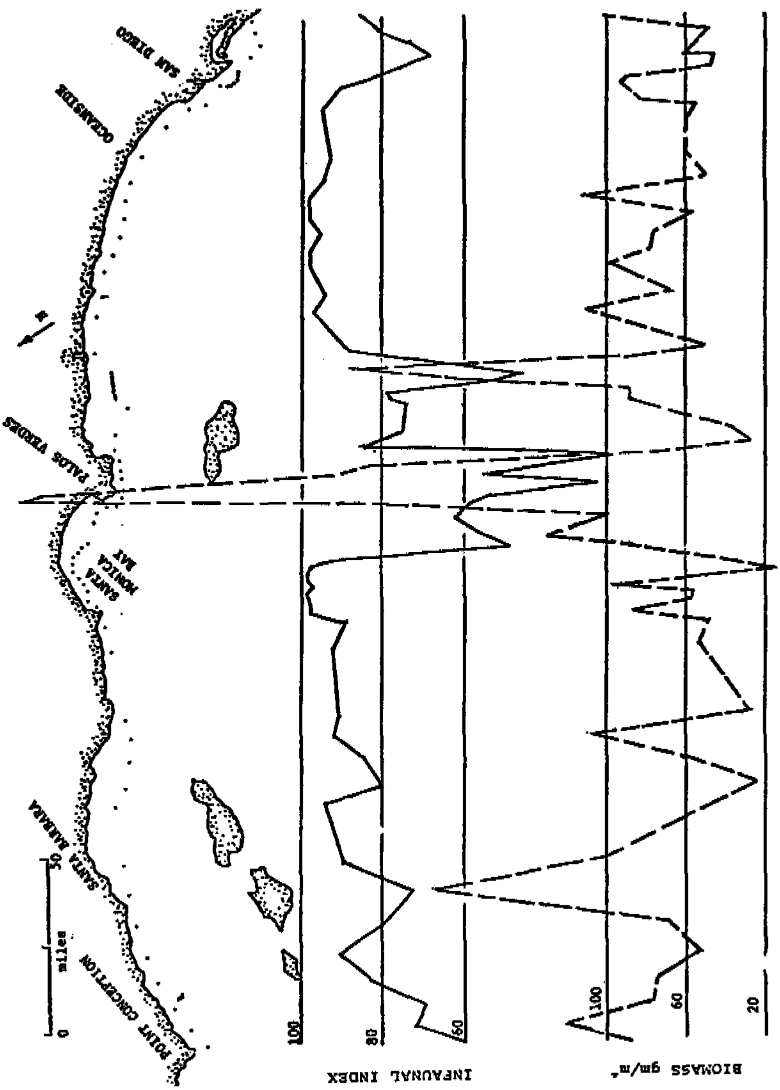


Figure 1. Infaunal index numbers and biomass for 60m depth and 10km intervals in Southern California Bight. Natural conditions exist when index ≥80 and biomass ≥ 70 gm/m^2. Note generally inverse relationship.

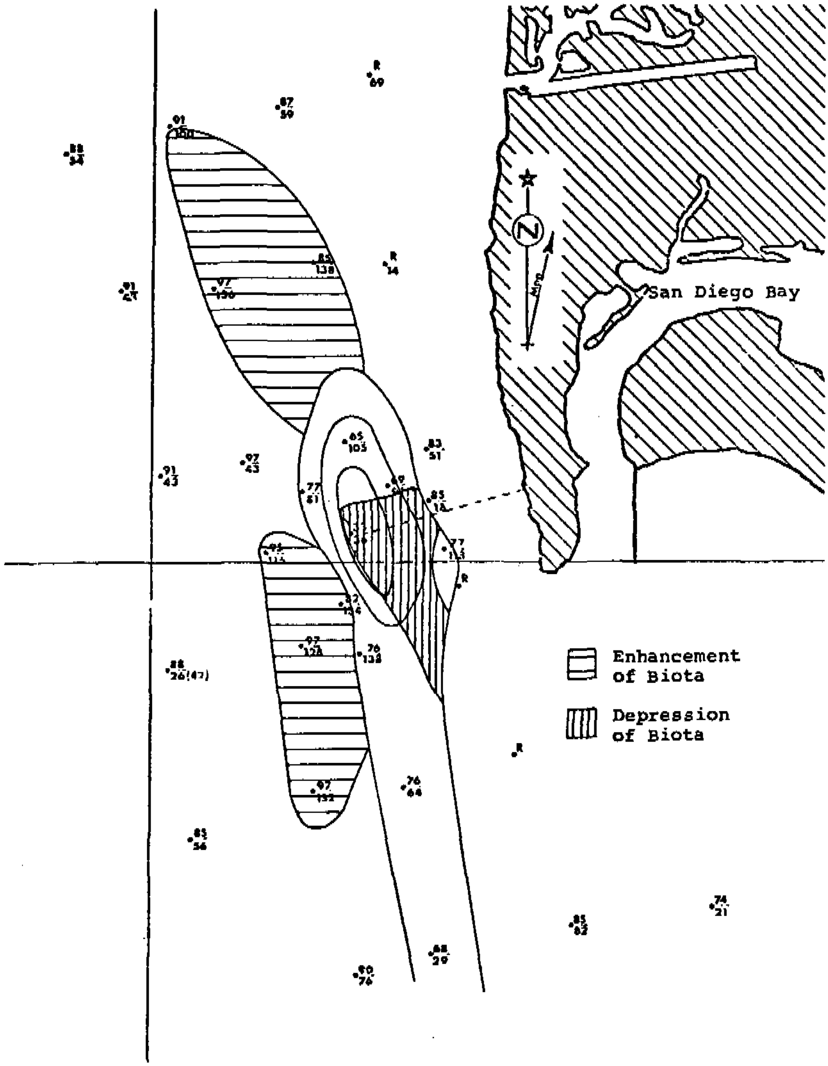


Figure 2. Infaunal index (top number) and biomass (bottom number) near San Diego outfall.

GEOGRAPHICAL INDEX

SUBJECT INDEX

LIST OF CONTRIBUTORS

Arlosoroff, Saul. Director, MAOT Ltd., Consulting Engineers, PO Box 21577, Tel Aviv, Irsael

Bascom, Willard. Director, Southern California Coastal Water Research Project, 1500 E. Imperial Highway, El Segundo, California 90245

Bouzoun, John R. Environmental Engineer, US Army Corps of Engineers, Cold Regions Research and Engineering Laboratory, Hanover, New Hampshire 03755

Bradley, David J. Professor and Director, Ross Institute of Tropical Hygiene, London School of Hygiene and Tropical Medicine, Keppel Street (Gower Street), London WC1E 7HT, U.K.

Feachem, Richard G. Senior Lecturer, Ross Institute of Tropical Hygiene, Keppel Street (Gower Street), London WC1E 7HT, U.K.

Gujer, Willi. Head, Department of Engineering, Federal Institute for Water Resources and Water Pollution Control (EAWAG), CH-8600, Duebendorf, Switzerland

Gunnerson, Charles G. Environmental Engineering Advisor, National Oceanic and Atmospheric Administration, U.S. Department of Commerce, Boulder, Colorado 80303

Hais, Alan B. Chief, Municipal Technology Branch, Office of Water Program Operations, USEPA, Washington, DC 20460

Julius, DeAnne S. Economist, Energy, Water and Telecommunications Department, The World Bank, Washington, DC 20433

Kalbermatten, John M. Water and Wastes Adviser, Energy, Water and Telecommunications Department, The World Bank, Washington, DC 20433

Kolsky, Peter J. Senior Engineer, Camp, Dresser & McKee, Inc., One Center Plaza, Boston, Massachusetts 02108

Lauria, Donald T. Professor, School of Public Health, University of North Carolina, Chapel Hill, North Carolina 27514

Martel, C. James. Environmental Engineer, US Army Corps of Engineers, Cold Regions Research and Engineering Laboratory, Hanover, New Hampshire 03755

McKim, Harlan L. Program Manager, Land Treatment of Wastewater Research Program, US Army Corps of Engineers, Cold Regions Research and Engineering Laboratory, Hanover, New Hampshire 03755

Middleton, Richard N. Senior Sanitary Engineer, Energy, Water and Telecommunications Department, The World Bank, Washington, DC 20433

Palazzo, Antonio J. Agronomist, US Army Corps of Engineers, Cold Regions Research and Engineering Laboratory, Hanover, New Hampshire 03755

Presecan, N.L. Senior Vice-president and Chief Engineer, Engineering-Science, Inc., 150 No. Santa Anita Ave., Arcadia, California 91006

Reed, Sherwood C. Environmental Engineer, US Army Corps of Engineers, Cold Regions Research and Engineering Laboratory, Hanover, New Hampshire 03755

Saunders, Robert J. Chief, Telecommunications Division, Energy, Water and Telecommunications Department, The World Bank, Washington, DC 20433

Urban, Noel W. Chief, Engineering Management Branch, Research Division, Directorate of Civil Works, Office of the Chief of Engineers, Washington, DC 20314

Warford, Jeremy J. Economics Adviser, Energy, Water and Telecommunications Department, The World Bank, Washington, DC 20433

Wasmer, Hans R. Deputy Director, Federal Institute for Water Resources and Water Pollution Control (EAWAG), CH-8600 Duebendorf, Switzerland

White, Anne U. Research Associate, Institute of Behavioral Sciences, University of Colorado, Boulder, Colorado 80309

White, Robert L. President, Engineering-Science, Inc., 150 N. Santa Anita Ave., Arcadia, California 91006

White, Gilbert F. Professional Staff, Institute of Behavioral Sciences, University of Colorado, Boulder, Colorado 80309