Multiple Water Supply Approach for Urban Water Management

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Principal Investigator: Arun K. Deb, Ph.D., P.E.

WESTON
ENVIRONMENTAL CONSULTANTS-DESIGNERS

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ABSTRACT

An efficient and practical systems model has been developed to help cities, planners and engineers analyze and decide whether the multiple supply approach to urban water management will be beneficial in long-term planning of water resources. This systems model is a powerful tool for technical and economical analysis of various alternatives for long-term water supply management.

Three grades of water have been considered in developing the model: potable, subpotable, and nonpotable. The potable water can come from protected, naturally pure sources or from unprotected sources treated beyond present standards to assure the highest quality for ingestion. The subpotable supply, providing the bulk of the water, can be of questionable quality in terms of trace chemicals, but would be bacteriologically safe. The nonpotable supply will not be safe for human ingestion, and will essentially be reuse of wastewater effluent for industrial uses and for urban irrigation.

Basic water supply data were generated by conducting surveys of water supply systems serving populations around 20,000, 100,000 and 500,000 people all over the country. Cost functions of 36 unit processes have been developed and incorporated in the model.

The systems model that has been developed in this study is very flexible and can handle a conventional system, a dual or multiple system including reuse, or a regional system with up to 10 cities. The applicability of the model has been illustrated by applying the model to several hypothetical single, dual and multiple water supply systems.

Two seminars were organized, one at West Chester, Pennsylvania and the other at San Jose, California, to present the study results and invite discussions from the participants. Both seminars were attended by about 90 professionals.

In the second phase of the project, the model will be applied in two test case sites, and the applicability of the model in real world situations will be evaluated.

SECTION 1

BACKGROUND OF MULTIPLE WATER SUPPLY

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INTRODUCTION

The vital importance of good quality water for human consumption, agriculture and industry is well recognized. A fundamental need of any community is an adequate supply of biologically and chemically safe, palatable water of good mineral quality. While demand for this good quality water is high, its availability is limited.

Technological advances and increases in population during the past decades have caused both the demand for fresh water and the discharge of wastewater to increase. If the present rate of growth of population and industry continues, the quality of natural water will deteriorate, and it will be difficult to guarantee a high quality bulk water supply for domestic, industrial and commercial uses.

Every year our industries produce 600 to 700 new chemicals. The development of these new chemical compounds for the increasing demands of the consumer market and the growth of chemical use in agriculture and industry have allowed new micropollutants to enter the natural water courses. In addition to chemicals that are discharged into streams, many other chemicals are formed, primarily through chemical reactions with chlorine. The carcinogenic or otherwise toxic behavior of these chemicals is not clearly known. In a recent statement Harris (1) mentioned that "there is a relationship between drinking water quality and cancer." DDT and other substances, now proven dangers to man, were of little concern two decades ago. The latency period of these substances is in the order of 20 years. If, after 20 years or so it is concluded that these substances in drinking water do cause cancer, adverse effects on society cannot be overcome by short-term measures.

Good quality water for potable uses to meet primary and secondary drinking water standards should be obtained from protected natural sources. It has become increasingly difficult and expensive to bring water as is found in the natural lakes, streams, ponds and subsurface locations to potable quality. At this point it appears desirable to question the need to supply only water of potable quality to meet the divers community needs, only a small fraction of which require potable quality water.

The average total urban water usage for Americans today is approximately 160 gallons-per-capita/day. Vast amounts of potable quality water for industrial, commercial and public sectors is not required. Of the 60 gpcd average interior residential usage, 40 percent is required for toilet flushing, 30 percent for bathing, 15 percent for laundering, 6 percent for dishwashing, 5 percent for drinking and cooking, and 4 percent for other miscellaneous uses. In fact, of the average amount of water consumed per person, only about one-half gallon per day is required to be of high potable quality. The EPA and the National Academy of Science in their studies(2) to determine the maximum contaminant level (MCL) for primary drinking water regulations, assumed water consumption of 2 liters per capita per day. From a health standpoint, therefore, only 5 percent of water for interior residential use (drinking and cooking) actually needs to be of the highest quality. Okun(3) mentioned that a hierarchy of water supply should be established, with the quality of water being adapted for the use to which it is put.

CONCEPT

It is appropriate to consider a multiple supply approach to long-term water management. In dual or multiple supply systems, two or more grades of water would be supplied to consumers through separate distribution systems, according to the quality and quantity requirements for various uses. Since only a small fraction of household water must be of primary drinking water quality, the volume of water to be treated by an expensive and sophisticated process also would be small, allowing economy in treatment. The remaining portion of the domestic water supply would receive only the standard treatment processes and be supplied through a separate distribution system for subpotable uses. The two or more qualities of water (potable, subpotable, and nonpotable) could be supplied economically through separate distribution systems to meet various demands.

For the purpose of this study, various grades of water in a multiple supply system can be defined as follows:

- "Potable water" may be defined as water that is completely safe for long-term continuous human ingestion. This water should, as a minimum, satisfy primary and secondary drinking water regulations of the Safe Drinking Water Act.

- "Subpotable water" may be defined as water that is bacteriologically safe, may contain trace heavy metals and organics, but is not safe for long-term continuous human ingestion.

- "Nonpotable water" is not intended for and may not be safe for human ingestion.

In a multiple supply system, potable water can be used for the limited occasions for which water of high quality is required, and subpotable water can be used for all other household uses. The subpotable water would be disinfected and safe for occasional drinking but might not meet primary drinking water regulations for trace heavy metals and organic materials. In fact, the subpotable supply would be what some cities are now providing. Non-potable water can primarily be used for urban irrigation and limited industrial uses.

The balance between potable, subpotable and nonpotable supply will vary for each community depending on its residential, industrial, commercial and public water demands. In residences, the possibilities for potable water range from supplying all needs except toilet flushing and exterior uses, to providing only potable drinking and cooking water, and using subpotable water for the rest of household uses. The breakdown of potable versus subpotable water for each demand combined with the total levels of water usage will determine the character of the multiple water system.

BENEFITS

The benefits derived from a multiple water supply will be as follows:

- The risk of health hazard resulting from continuous ingestion of low levels of toxic contaminants over a period of years would be eliminated.

- The good quality water, which would have been used for subpotable purposes, not warranting high-quality water, would be conserved for potable use.

- Reuse of wastewater would reduce urban effluent, and conserve raw water.

REVIEW OF THE LITERATURE

Haney and Beatty(4) mentioned in a recent article that the concept of multiple water systems is not new and in fact such systems were considered as early as 1894. The late Gordon M. Fair revived the concept some 25 years ago, predicting that future water quality problems would be microchemical in nature. He also emphasized that a good quality water source, relatively free of contaminants, was a resource of great value, worthy of protection and wise use. Fair suggested that multiple systems offer a means of conserving a limited supply of good water.

The first systematic cost study of multiple water supply systems was made by Haney and Hamann(5). They assumed that the potable supply (27 percent of total water usage) was used for drinking, cooking, bathing and laundering, and limited industrial and commercial uses. The nonpotable supply would be used for toilet flushing, lawn irrigation, and fire protection. They found that a savings of up to 20 percent could be obtained by a multiple supply when demineralization was required to produce potable water.

Okun and McJunkin(6) made a case study in which they proposed to use a second supply to cope with increased demand for Raleigh, N.C. The second supply proposed for nonpotable uses would be taken from the polluted Neuse River. They estimated that the costs of a multiple system would be 21 percent greater than the costs of a conventional system, but that the consumer would be assured of a good quality water supply which had not been exposed to urban and industrial wastewaters.

In a comprehensive study of multiple water supply systems of hypothetical British towns, using systems models, Deb and Ives (7) found that, for a new town where the raw water contained a high total of dissolved solids, a multiple supply would be cheaper. They developed methodology for analysis of water systems for cities of two sizes, incorporating various economical and technical parameters. From this it appears that demineralization of wastewater effluent for reuse would be more expensive than demineralizing only the potable supply. In the case of a conventional treatment system with chlorination to produce potable water, the total cost of a multiple supply is less than a conventional system if the potable supply requirement is less than 30 percent of the total. Use of limited supplies of high-quality groundwater for potable supply and polluted surface water, adequately treated, for subpotable supply was found to be more economical than a conventional system treating the polluted source. In their study, reuse was not considered as one of the options.

Jackson(8) made a study to utilize the heavily polluted Trent River in England as a source for industrial water supply, eventually to replace about one-third of the demand for potable supply.

In 1975, the American Waterworks Association, realizing the potential importance of a multiple water supply, formed a committee on multiple distribution systems. The committee conducted the first seminar on multiple distribution systems in June, 1976. In the introductory remarks at the seminar, Okun(9) mentioned that "requirements for water pollution abatement have resulted in the production of effluents of such quality that in many instances they are too valuable to be discarded, but are useful as resources for nonpotable purposes."

EXISTING MULTIPLE WATER SYSTEMS

Multiple water supply systems have been used in various parts of the world where water supply is scarce, as in the Bahamas, Catalina Island, Hong Kong, and Grand Canyon Village. In Singapore, a part of the wastewater effluent after filtration is used for industry and for toilet flushing. In England and Wales about 42 water authorities are multiple supply systems: domestic (potable), and industrial (nonpotable).

In the United States, St. Petersburg, Florida plans to use reclaimed municipal wastewater for lawn sprinkling because of the limitations on the availability of fresh water. Thus, up to 40 percent of good quality ground water will be conserved(10). In Colorado Springs, Colorado, about one-third of the wastewater is treated and disinfected, stored, and then distributed through a second distribution system for serving nonpotable water to customers using more than 10,000 gallons per day. The water is sold at two-thirds the price of the potable water that had previously been used, thus, both the purveyor and the user profit(11).

Irvine Ranch Water District in California is presently supplying 8 mgd of secondary activated sludge effluent after filtration and chlorination for urban irrigation uses through a multiple distribution system. The supply is in the process of being increased to 15 mgd. This nonpotable system reduces the load on the fresh water demand by an equivalent amount.

SUMMARY AND CONCLUSIONS

The drought in California focuses the attention of the whole country on the need for water conservation and reuse. California Governor Edmund G. Brown, Jr. has recently created a new state office of water recycling. Reuse of water has been emphasized in the Water Pollution Control Act (PL 92-500). In the near future, water reclamation and reuse will be a necessary feature in developing a long term water management plan.

If it is assumed that recycled water will not be employed for potable uses, a separate distribution system will be necessary for supplying recycled water for subpotable or nonpotable uses. In fact, multiple supply systems would be a necessary feature of water management in areas where wastewater reuse will be considered.

A multiple water supply can also be adopted in cases where raw water source quality is poor and has high organic, heavy metal, and total dissolved solids (TDS) content. The cost of removal of these materials is high and multiple supply systems may be economically viable over the conventional system. Again, this depends on the savings of additional treatment costs over the additional distribution cost.

A multiple water supply should be an essential consideration in developing a long term water management plan for a city or a region considering conservation and optimum uses of water. Unfortunately no methodology for analysis of multiple water supply systems is available. Weston, with the support from the National Science Foundation (RANN) program, has developed a systems computer model by which various options of conventional water supplies can be compared and evaluated for cost effectiveness on a present worth basis.

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SECTION 2

COLLECTION AND ANALYSIS OF WATER DEMAND DATA

COLLECTION AND ANALYSIS OF WATER-DEMAND DATA

The purpose of this task in the Dual Water Supply Study has been to gather and analyze water-demand data as input to the system methodology. Information from an extensive literature search, water-utility survey, and direct contacts with people in the water-supply field provided a basis for comparison of water usages and projection of pre-capita demand for the six categories in Table 1.

WATER-DEMAND COMPONENTS

In December 1976, Weston conducted a detailed survey of selected water utilities serving populations of 20,000, 100,000, and 500,000, respectively. Of the 65 water utilities which were sent the questionnaire, 26 or 40 percent responded with applicable data. The average breakdown of urban water demands from this survey, along with a summary of results from the literature search of other studies, is presented in Table 2.

Table 1

Urban Water Demand Categories

- <u>Interior Residential</u>
  - Toilet Flushing
  - Bathing
  - Laundry
  - Dishwashing
  - Drinking and Cooking
  - Miscellaneous
- <u>Exterior Residential</u>
  - Irrigating
  - Car Washing
  - Swimming Pools
  - Cleaning
- <u>Commercial</u>
  - Office Buildings
  - Hotels
  - Restaurants
  - Car Washes
  - Laundries
  - Golf Course
  - Cemeteries
  - Shopping Centers
  - Retail Business
- <u>Industrial</u>
  - Manufacturing
  - Processing
  - Cooling
- <u>Public</u>
  - Schools
  - Prisons
  - Public Hospitals
  - Civic Buildings
  - Public Parks
- <u>Unaccounted-for</u>
  - Leakage and Loss
  - Fire Hydrant Usage
  - Testing
  - Flushing
  - Meter Under-Registration

Table 2

Urban Water Demands as a Percentage of Average Daily Use

Reference	Reference No.	Category of Use: Residential	Commercial	Industrial	Public	Unaccounted-For	Total Flow gpcd
McPherson (1976)	1	33	12	33	7	15	
California DWR (1976)	2	68	10	18	← 4 →		
Haney and Hamann (1964)	13	43	19	25	← 13 →		
US PHS (1955)	18	41	18	24	← 17 →		
Linaweaver, Geyer, and Wolff (1955)	10	50					160
Fair, Geyer, and Okun (1966)	21	33	← 43 →		7	17	150
Bostian: EPA (1974)	22	46	17	25	12	-	
Hirshleifer, DeHaven, Milliman (1960)	23	45	18	32	5	-	
Murray and Reeves, USGS (1970)	26	38	← 32 →		← 30 →		166
U.S. Water Resources Council (1968)	27	46	18	23	← 13 →		157
Frey, Gamble, and Sauerlender: NE US (1975)	28	49	12	21	← 18 →		166
AWWA (1970)	29	42	18	22	← 18 →		179
Weston National Water Utility Survey (1977)		52	17	15	7	9	153
Pennsylvania Water Utility Survey (1975)		39	12	31	5	13	162
AVERAGE		40	15	25	5	15	160

Nationally, the approximate categorization for the 160 gallons of per capita average daily use (gpcd) is as follows:

Use	Percentage
Residential	40
Commercial	15
Industrial	25
Unaccounted-for	20

Table 3 shows the urban water usage of 27 water utilities in Pennsylvania.

The way in which water is consumed in residential households is important in dual-water systems. Various researchers have made determinations of interior residential water demand (included in Table 4) for the following six categories: toilet flushing, bathing, laundry, dishwashing, drinking and cooking, and miscellaneous. A high quality of water is required for drinking, cooking, and dishwashing, which constitute about 11 percent of the total interior residential demand; however, the remaining 89 percent of interior residential water may not require water of potable quality.

Exterior residential water demand varies over a wide range throughout the country, depending on season and area. On an annual daily average basis, exterior usage amounts to about 7 percent of residential demand in Pennsylvania and 44 percent in California. The climate influences water consumption, with the difference between summer and winter representing the exterior residential usage. A high quality of water is not required for exterior residential uses, and the demand can be satisfied using subpotable water. Table 5 shows interior and exterior residential water uses as percentage of average daily use as obtained by various researchers.

Table 3

1975 Pennsylvania Urban Water Demands
as a Percentage of Average Daily Use

Population Class	Water Utility	CATEGORY OF USE WITH %				
		Residential	Commercial	Industrial	Public	Unaccounted-For
20,000	Easton	55	22	23	--	--
	Phoenixville	40	10	30	15	5
	Media	72	7	2	5	14
	Pottstown	29	14	31	1	25
	North Wales	26	4	44	5	21
	Coatesville	38	31	30	--	1
	West Chester	59	--	25	15	1
	Hanover	38	10	46	2	4
	Keystone	12	8	57	7	16
	Lewistown	32	8	40	1	19
	Meadville	39	30	31	--	--
	Highland	42	18	8	1	31
	Western PA	34	11	24	21	10
	Latrobe	24	10	49	1	16
	Uniontown	46	35	9	5	5
	Aliquippa	26	8	41	4	20
100,000	Allentown	34	15	32	9	10
	Bethlehem	21	8	46	8	17
	Chester	20	6	56	1	17
	Lancaster	53	3	26	6	12
	York	48	11	37	4	--
	Erie	40	10	35	3	12
	Wilkinsburg	44	5	24	7	20
	West View	66	22	5	5	2
500,000	Pennsylvania Gas and Water	22	3	34	1	40
	Pittsburgh	41	18	25	5	21
	W. Penn Water	40	15	17	8	20
TOTAL	AVERAGE	39	12	31	5	13

Table 4

Interior Residential Water Usage Comparison as a Percentage of Average Daily Use

Reference	Reference No.	Category of Use: Toilet Flushing	Bathing	Laundry	Dishwashing	Drinking and Cooking	Miscellaneous	Total Flow gpcd
McPherson (1976)	1	42	27	← 17 →		8	6	
California DWR (1976)	2	42	32	14	← 12 →			
ERCO (1975)	3	39	34	14	6	5	2	64
Laak (1975)	4	47	21	18	← 9* →			41*
Murawczyk and Ihrig (1973)	5							62
Ligman (1972)	6	41	26	19*	← 10* →			45*
Wallman (1972)	7	27-45	18-36	18	← 13* →			30-50*
Howe, et al (1971)	8	45	30	← 20 →		5		
Bailey and Wallman (1971)	9	39	34	14	← 11 →		2	64
USGS (1962)	12	41	37	4	← 11 →		7	
Haney and Hamann (1965)	13	39	32	14	← 11 →		4	61.5
Bennett (1975)	14	33	24	27	← 16 →			44.5*
Siegrist, Witt and Boyle (1976)	15	22	23	25*	← 11* →		19*	
Water Encyclopedia (1970)	16	42	38	7	← 11 →		2	
Bostian: EPA (1973)	17	27-45	22-36	16	← 13 →		2	
US PHS (1967)	18	30	35	20	← 15 →			
Chanlett (1973)	19	43	38	7	← 11 →		11	
Univ. of Wisc. (1973)	24	49	30	15	← 10 →		5	50
Bailey, et al (1969)	25	49	32	4	← 12* →		3*	
AVERAGE		40	30	15	6	5	4	60

*Rural figures.

Table 5

Residential Water Use
As a Percentage of Average Daily Use

Reference	Reference No.	Category of Residential Use		
		Interior	Exterior	Total Flow gpcd
Linaweaver, Geyer, and Wolff (1966)	10	74	26	80
California DWR (1976)	2	56	44	
Bailey, et al (1969)	25	93	7	
Omaha Urban Study (1976)	27	85	15	
PA Average from DER Data (1975)		93	7	
USGS (1962)	12	96	4	55
AVERAGE		94	6	64

WATER-DEMAND COEFFICIENTS

Interior residential water demand (D_{IR}) has been found to be approximately uniform for metered urban areas throughout the country. Thus, the other five water demand categories can be expressed by multiplying D_{IR} by coefficients depending on various factors.

Exterior Residential Demand

$$D_{ER} = K_{ER} D_{IR}$$

where:

D_{ER} = Exterior residential demand

K_{ER} = Coefficient or ratio of exterior to interior residential demand

The coefficient K_{ER} is dependent on geographic location, climate, population density, property value, and water price.

Commercial Demand

$$D_C = K_C D_{IR}$$

where:

D_C = Commercial demand

K_C = Coefficient for commercial demand

The coefficient K_C for a particular city depends on the size of the commercial area and the intensity of commercial activity.

Industrial Demand

$$D_I = K_I D_{IR}$$

where:

D_I = Industrial water demand

K_I = Coefficient for industrial demand

The coefficient K_I depends on industrial activity, industry type, reuse, and urban water supply utilization.

Public Demand

$$D_P = K_P D_{IR}$$

where:

D_P = Public water demand

K_P = Coefficient for public demand

The coefficient K_P depends on the city size and the availability of public resources.

Unaccounted-For

$$D_{UF} = K_{UF} D_{IR}$$

where:

D_{UF} = Unaccounted-for water

K_{UF} = Coefficient for unaccounted water

The coefficient K_{UF} is dependent on the age and maintenance of the distribution system and fire hydrant usage.

The total per capita urban water demand is then the summation of these categorized demands:

$$TWD = D_{IR} + D_{ER} + D_C + D_I + D_P + D_{UF}$$

$$= D_{IR} (1 + K_{ER} + K_C + K_I + K_P + K_{UF})$$

The values of the demand coefficients (K values) can be evaluated from water utility data as shown in Table 6. Table 7 gives the average coefficients obtained from the Pennsylvania Water Utility Survey, AWWA 1970 Statistical Report, and the Weston National Water Utility Survey. These demand coefficients depend on many variables and may vary widely; the ranges are included in the table.

These averages are listed as background to aggregating and disaggregating total per capita water demand. Each community must be analyzed independently to determine its coefficients.

Table 6

Pennsylvania Water Demand Data Analysis

Water Utility	Water Demand (gpcd)			Water Demand Coefficients				
	Total TWD	Residential $D_{IR} + D_{ER}$	Interior Residential D_{IR}	Exterior Residential K_{ER}	Commercial K_C	Industrial K_I	Public K_P	Unaccounted-for K_{UF}
Easton	94	52	50	0.04	0.41	0.43	--	--
Phoenixville	216	86	75	0.15	0.29	0.86	0.43	0.14
Media	109	78	69	0.13	0.11	0.03	0.08	0.22
Pottstown	139	40	40	--	0.49	1.08	0.03	0.87
North Wales	129	34	34	--	0.15	1.67	0.19	0.80
Coatesville	165	63	57	0.11	0.90	0.87	--	0.03
West Chester	116	68	62	0.10	--	0.47	0.28	0.02
Hanover	107	41	40	0.03	0.27	1.23	0.05	0.11
Keystone	292	35	35	--	0.67	4.76	0.58	1.33
Lewistown	174	56	54	0.04	0.26	1.29	0.03	0.61
Meadville	154	60	55	0.09	0.84	0.87	--	--
Highland	74	31	31	--	0.43	0.19	0.02	0.74
Western PA	82	28	28	--	0.32	0.70	0.62	0.29
Latrobe	216	52	50	0.04	0.43	2.12	0.04	0.69
Uniontown	126	58	54	0.07	0.82	0.21	0.12	0.12
Aliquippa	164	43	42	0.02	0.31	1.59	0.16	0.78
Allentown	200	68	62	0.10	0.48	1.03	0.29	0.32
Bethlehem	258	54	52	0.04	0.40	2.28	0.40	0.84
Chester	227	45	44	0.02	0.31	2.89	0.05	0.88
Lancaster	148	78	69	0.13	0.06	0.56	0.13	0.26
York	184	88	75	0.17	0.27	0.91	0.10	--
Erie	262	105	90	0.17	0.29	1.02	0.09	0.35
Wilkinsburg	137	60	55	0.09	0.12	0.60	0.17	0.50
West View	70	46	45	0.02	0.34	0.08	0.08	0.03
Pa. Gas & Water	204	49	47	0.04	0.13	1.48	0.04	1.74
Pittsburgh	177	73	65	0.12	0.49	0.68	0.14	0.57
W. Penn Water	130	52	50	0.04	0.39	0.46	0.22	0.54
AVERAGE	162	57	53	0.08	0.38	1.00	0.18	0.40

Table 7

Average Water Demand Coefficients

Coefficient	Pennsylvania Survey	AWWA Survey	Weston Survey Range	Weston Survey Mean	Weston Survey Standard Deviation
K_{ER}	0.08	0.27	0.05-1.5	0.75	0.62
K_C	0.38	0.56	0.05-1.4	0.57	0.25
K_I	1.00	0.70	0.05-3.0	0.47	0.41
K_P	0.18	-	0.05-0.6	0.18	0.12
K_{UF}	0.40	-	0.05-1.0	0.24	0.12

WATER QUALITY

In a multiple water supply system, several grades of water are supplied through distribution main to various consumers. For this study, three types of water have been classified: potable, subpotable, and non-potable. The balance between these three supplies will vary for each community, depending on water availability and demand components. For instance, the possibilities for potable water in residences range from supplying all needs except toilet flushing and exterior uses to providing only potable drinking and cooking water and using subpotable water for the rest of household uses. The breakdown of potable, sub-potable, and nonpotable water for each demand category in combination with the total levels of water usage will determine the character of the multiple-supply system.

The subpotable fraction of total per capita urban water demand (SPWD) can be estimated as follows:

$$SPWD = SP_{IR}\,D_{IR} + SP_{ER}\,K_{ER}\,D_{IR} + SP_{C}\,K_{C}\,D_{IR} + SP_{I}\,K_{I}\,D_{IR} + SP_{P}\,K_{P}\,D_{IR} + SP_{UF}\,K_{UF}\,D_{IR}$$

$$= D_{IR}\,(SP_{IR} + SP_{ER}\,K_{ER} + SP_{C}\,K_{C} + SP_{I}\,K_{I} + SP_{P}\,K_{P} + SP_{UF}\,K_{UF})$$

Similarly, the nonpotable fraction of urban water demand (NPWD) can be estimated as follows:

$$NPWD = NP_{IR}\,D_{IR} + NP_{ER}\,K_{ER}\,D_{IR} + NP_{C}\,K_{C}\,D_{IR} + NP_{I}\,K_{I}\,D_{IR} + NP_{P}\,K_{P}\,D_{IR} + NP_{UF}\,K_{UF}\,D_{IR}$$

$$= D_{IR}\,(NP_{IR} + NP_{ER}\,K_{ER} + NP_{C}\,K_{C} + NP_{I}\,K_{I} + NP_{P}\,K_{P} + NP_{UF}\,K_{UF})$$

The potable water demand can be determined by difference since the three fractions must add up to the total water demand.

$$PWD = TWD - SPWD - NPWD$$

WATER DEMAND PROJECTION

Consumption of municipal water is influenced by many factors, including population, price, consumer age and income, regional cultural behavior, and climatic considerations. Meeting the increased demands as urban populations and per capita consumption increase is a complex problem.

Many factors are pertinent to the problem of demand forecasting. Generally, the future demand is estimated over a specified time horizon based on an average per capita water consumption and population. Therefore, population prediction and per capita water demand projection during the planning period is necessary in order to predict future water demands.

There are various methods of population projections. On one side of the spectrum of population projection methods is extrapolation of historical population trends to predict the future population; at the other end of the spectrum is the use of complex mathematical models to predict future population considering many variables.

In this study no new method of population projection will be developed. Most states have developed population projections through their respective state departments (e.g., Department of Finance in California; Department of Budget and Control in South Carolina) and by private agencies or institutions (e.g., Bell Telephone or universities). In most cases, these state projections are disaggregated to the county level; in certain states, an agency or planning department has allocated the county projections to municipalities.

The nationwide source for population projections is the Bureau of Economic Analysis (BEA), formerly known by the acronym OBERS, which included the Office of Business Economics (OBE) of the U.S. Department of Commerce and the Economic Research Service (ERS) of the U.S. Department of Agriculture. The OBERS projections for Standard Metropolitan Statistical Areas (SMSA's), economic areas, and water resources regions and subareas are available for 1980, 1985, 2000, 2010, and 2020.

In the systems model to be developed in this study, population for future years will be given as input. With future interior residential demand as input, the other demands (commercial, industrial, public, unaccounted-for, and exterior residential) are determined by multiplying by appropriate coefficients. (Because of the great variability in coefficients, the default values for the demand coefficients should be used with caution.)

The other mode of operation for the dual water supply model is to use total water demand to project demand curves. Utilities often have total water demand projections which can be disaggregated to its components. This will be discussed further in the section on system model development.

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SECTION 3

WATER QUALITY AND TREATMENT REQUIREMENTS IN MULTIPLE WATER SUPPLY SYSTEMS

WATER QUALITY AND TREATMENT REQUIREMENTS
IN MULTIPLE WATER SUPPLY SYSTEMS

INTRODUCTION

In dual or multiple water supply systems, two or more qualities of water, one potable, the other subpotable and/or nonpotable, would be supplied through separate distribution systems.

The quality of potable water in the United States is regulated by the Safe Drinking Water Act. The Safe Drinking Water Act (P.L. 93-523) was enacted on December 16, 1974, giving the Administrator of the Environmental Protection Agency the power to control the quality of the drinking water in public water systems. A "public water system" has been defined in the Act as a system providing piped water for human consumption to the public, if such system has at least 15 service connections. The Act calls for the establishment of comprehensive regulations for drinking water quality in three stages:

1. Promulgation of National Interim Primary Drinking Water Regulations.

2. A study to be conducted by the National Academy of Science, within two years of enactment, on the human health effects of exposure to contaminants in drinking water.

3. Promulgation of Revised National Primary Drinking Water Regulations based upon the National Academy of Science Report.

National Interim Primary Drinking Water Regulations were promulgated and became effective June 24, 1977. The EPA also proposed that secondary regulations complement the primary regulations. While primary regulations are devoted to water components and regulations affecting the health of consumers, secondary regulations are those which deal with the esthetic qualities of drinking water. Secondary regulations are not federally enforceable and are intended as guidelines for the states.

QUALITY OF POTABLE WATER

The quality of ideal potable water should conform with the Primary and Secondary Drinking Water Regulations. The Interim Primary Drinking Water Standards contain maximum contaminant levels and monitoring requirements for microbiological contaminants, 10 inorganic chemicals (arsenic, barium, cadmium, chromium, lead, mercury, nitrate, selenium, silver and fluoride), six organic chemicals (four chlorinated hydrocarbons and two chlorophenoxys), radionuclides and turbidity. Recently, EPA added trihalomethane to the list. The maximum contaminant levels (MCL) of various contaminants as prescribed in the Interim Primary Drinking Water Regulations(1) are given in Table 1.

The Secondary Drinking Water Regulations contain maximum contaminant levels (MCL) for chloride, color, copper, corrosivity, foaming agents, hydrogen sulfide, iron, manganese, odor, pH, sulfate, total dissolved solids and zinc. The MCL values as prescribed by the Secondary Drinking Water Regulations(2) are given in Table 2. Sixteen metals (barium, beryllium, cadmium, chromium, cobalt, copper, lead, magnesium, manganese, mercury, molybdenum, nickel, tin, silver, vandium, and zinc) have been reviewed by the National Academy of Science with respect to their relative contribution to man's activities and to the concentrations found in water supplies, and have been rated as given in the following tabulation.

Additional Metals Reviewed by NAS(3)

Metals	Rating
• Cadmium, chromium, copper, mercury, lead and zinc.	Very great
• Silver, barium, molybdenum and tin.	High
• Beryllium, cobalt, manganese, nickel and vandium.	Moderate
• Magnesium	Low

Potable water quality in a multiple supply system should conform with the revised Primary Drinking Water Regulations and proposed secondary regulations. As more knowledge on these contaminants becomes available, these standards are supposed to change from time to time to include more chemicals into the list.

QUALITY OF SUBPOTABLE WATER

The quality of the subpotable supply should be maintained at such a level that its occasional inadvertent use for drinking would not cause any harmful effects. This necessitates that the water be free from harmful organisms and acutely toxic chemicals. The subpotable supply, providing the bulk of the water, could be of questionable quality in terms of trace chemicals, but would be bacteriologically safe through conventional treatment, including disinfection. In fact, the subpotable supply would be what some cities are now providing. It is important to note that both potable and subpotable water in a dual system would be safe for short term drinking. Potable water would be safe also for long-term continuous drinking.

Table 1

National Interim Primary Drinking Water Regulations (1)

Maximum Contaminant Levels (MCL)

Contaminant	MCL mg/L
Inorganic	
Arsenic	0.05
Barium	1.0
Cadmium	0.01
Chromium	0.05
Lead	0.05
Mercury	0.002
Nitrate (as N)	10.0
Selenium	0.01
Silver	0.05
Turbidity	1 unit
Fluoride	1.4 - 2.4
Organic	
a. Chlorinated hydrocarbons	
Endrin	0.0002
Lindane	0.004
Methoxychlor	0.1
Toxaphene	0.005
b. Chlorophenoxys	
2,4-D	0.1
2,4,5-TP Silvex	0.01
Microbiological Contaminant	One coliform bacterium per 100 ml as the arithmetic mean of all samples per month.
c. Trihalomethane	0.1 (proposed)

QUALITY OF NONPOTABLE WATER

The nonpotable supply in a multiple supply system would basically be recycled water from either secondary effluent or tertiary effluent. It would be used for irrigation of public parks and golf courses, air conditioning and industrial cooling, recharging of groundwater, and other low grade water uses. The effluent water would be chlorinated with large dosages and a long contact period provided to kill all coliform bacteria.

The quality of nonpotable water should be such that it is free from harmful bacteria, and viruses below detection level. It would be relatively clear and might contain nutrients.

WATER SOURCES

In this study the following sources of water have been considered for potable, subpotable and nonpotable supplies:

1. For Potable Supply

    a. Good quality protected ground water.

    b. Protected upland reservoir.

    c. Unprotected ground water after extensive treatment.

    d. Unprotected surface sources (polluted streams, rivers or lakes) after extensive treatment.

2. For Subpotable Supply

    a. Unprotected ground water after usual conventional treatment.

    b. Unprotected surface water after conventional treatment.

    c. Advanced wastewater treatment effluent for reuse after extensive treatment.

3. For Nonpotable Supply

    a. Secondary effluent for reuse.

    b. Advanced wastewater effluent for reuse.

WATER QUALITY CRITERIA AND TREATMENT

The National Interim Primary Drinking Water Regulations prescribed maximum contamination levels for 10 inorganic contaminants. Removal of inorganic ions from drinking water is done in most cases by conventional coagulation and lime softening. Recent EPA research for removal of inorganic contaminants is based on coagulation and lime treatment(4). These EPA studies show that no one treatment technique is effective for all contaminants. A summary of treatment methods for inorganic contaminants as suggested by EPA research is given in Table 3(4). Most of the methods listed in this table are conventional coagulation and lime softening. Other treatment techniques such as ion exchange and reverse osmosis may be equally effective.

In developing treatment systems for water containing trace metals, either chemical coagulation or lime treatment has been adopted. In the removal of high total dissolved solids, reverse osmosis has been adopted.

If it is necessary to remove all the inorganic pollutants described in Interim Primary Drinking Water Regulations, more than one unit process of treatment will be required.

The Interim Primary Drinking Water Regulations prescribed a maximum contaminant level for turbidity as 1 unit. Turbidity can be removed from water by conventional coagulation, sedimentation and filtration. In developing treatment systems for river waters, coagulation, sedimentation and filtration have been adopted for turbidity removal in this study.

The organic compounds for which maximum concentration levels have been established are the pesticides endrin, lindane, toxaphene, 2,4-D, 2,4, 5-TP and methoxychlor. A summary of techniques for removal of pesticides is given in (4). Detailed information on the removal of 2,4,5-TP and methoxychlor are not available. It is apparent, however, that carbon adsorption is more effective in removal of pesticides than any other known method. Therefore, the granular carbon adsorption method has been adopted for trace organics removal in this study.

Recently EPA, under the Safe Drinking Water Act, proposed to establish maximum contaminant levels for trihalomethane compounds as 100 microgram per liter. Eventually, EPA expects to reduce trihalomethane levels to 10 micrograms per liter. EPA also proposed that water agencies serving populations of 10,000 or more would be required to develop a water sampling program to determine the prevalence of trihalomethanes. In addition to sampling water supplies, water agencies serving populations of 75,000 or more would be required to add granular activated carbon in the treatment system for removing trihalomethanes. About 400 communities, or 52 percent of all of the people served in the United States by community water systems, will be affected by this regulation.

For disinfection of water, chlorine, ozone, and chlorine dioxide can be used effectively. In this study chlorine or ozone has been considered for disinfection.

Table 3

Most Effective Treatment Methods for Inorganic Contaminant Removal*

Contaminant	Most effective methods	Contaminant	Most effective methods
Arsenic:			
As^{+3}	Ferric sulfate coagulation, pH 6-8	Fluoride	Ion exchange with activated alumina or bone char media
	Alum coagulation, pH 6-7		
	Excess lime softening	Lead	Ferric sulfate coagulation, pH 6-9
	Oxidation before treatment required		Alum coagulation, pH 6-9
			Lime softening
As^{+5}	Ferric sulfate coagulation, pH 6-8		Excess lime softening
	Alum coagulation, pH 6-7	Mercury:	
	Excess lime softening	Inorganic	Ferric sulfate coagulation, pH 7-8
Barium	Lime softening, pH 10-11	Organic	Granular activated carbon
	Ion exchange	Nitrate	Ion exchange
Cd^{+3}	Ferric sulfate coagulation, above pH 8	Selenium:	
		Se^{+4}	Ferric sulfate coagulation, pH 6-7
	Lime softening		Ion exchange
	Excess lime softening		Reverse osmosis
Chromium:		Se^{+6}	Ion exchange
Cr^{+3}	Ferric sulfate coagulation, pH 6-9		Reverse osmosis
	Alum coagulation, pH 7-9	Silver	Ferric sulfate coagulation, pH 7-9
	Excess lime softening		Alum coagulation, pH 6-8
Cr^{+6}	Ferrous sulfate coagulation, pH 7-9.5		Lime softening
			Excess lime softening

*Manual of Treatment Techniques for Meeting the Interim Primary Drinking Water Regulations.

Based on Interim Primary Drinking Regulations and Secondary Regulations, five different treatment systems have been developed for potable water, three treatment systems for a subpotable supply and one for nonpotable supply. These systems are shown in Table 4.

COST FUNCTIONS

Treatment Systems and Distribution Costs

To develop a systems model for conventional and multiple water supply, the capital and O&M costs of various treatment and distribution units as functions of flow are required. In this study no new cost data are generated. Cost data for various units of treatment and distribution are taken from recent literature (5-12) and updated and formulated in mathematical functions valid for September, 1976, which is considered as the base for this study.

The capital cost functions include the piping cost (10 percent), electrical (8 percent), instrumentation (5 percent), site preparation (5 percent), engineering and construction supervision (15 percent) and contingencies (15 percent).

Operation and maintenance costs consist of labor, material, energy and chemical components. For each capital cost function there is a corresponding O&M cost function.

Labor costs include the manpower required to operate and maintain the system plus such support tasks as supervision and administration, chemical work, laboratory work, and yard work. Materials costs include the various materials required for routine maintenance of the system. All energy costs required in the system are included in the O&M cost. The cost of chemicals required in various unit processes such as two-stage lime treatment, carbon adsorption, chlorination, metal salt addition, etc., are also included in the O&M cost.

The various cost functions for capital and O&M costs for various unit water and wastewater treatment processes and components of distribution systems are given in Tables 5 and 6.

SUMMARY

In this section cost functions of various units of a water supply system have been developed. Treatment systems considered to produce potable, subpotable and nonpotable water from various water sources are tabulated. Quality requirements of potable, subpotable and nonpotable water have also been identified.

To compare a conventional supply system with the multiple supply system, the quality of the potable fraction of a multiple supply system will be considered to be the same as that of a conventional system.

Table 4

Treatment Systems for Potable, Subpotable and Nonpotable Water Supply

Treatment System Number	Type of Water Supply	Source	Quality	Treatment System Model
TS1	Potable	Ground Water (protected)	Good and Clean	Disinfection
TS2	Potable	Ground Water (unprotected)	Polluted with high TDS, Hardness, iron and manganese, and trace metals	Lime treatment (including recarbonation) + filtration + disinfection
TS3	Potable	Upland Reservoir (protected surface water)	Generally good quality except turbidity	Filtration + Disinfection
TS4	Potable	River, Stream, or Lake (unprotected surface water)	Polluted with trace organics (including carcinogens and suspended solids)	Chemical coagulation (with alum) + sedimentation + activated Carbon + Filtration + Disinfection
TS5	Potable	River, Stream, or Lake (unprotected surface water)	Polluted with trace organics, heavy metals, and high TDS	Lime Treatment (including Recarbonation) + Activated Carbon + Filtration + Disinfection
TS6	Subpotable	Ground Water (unprotected)	Polluted with trace organics, iron and manganese	Aeration + Filtration + Chlorination
TS7	Subpotable	River, Stream, or Lake (unprotected surface water)	Polluted with trace organics, heavy metals, and suspended solids	Chemical Coagulation (with Alum) + Sedimentation + Filtration + Chlorination
TS8	Subpotable	Advanced Waste Treatment[1] Effluent (for reuse)	Polluted with TDS	Reverse Osmosis of a portion of flow
TS9	Nonpotable	Secondary Effluent (for reuse)	Polluted with high residual level of organics, TDS, and virus	

[1] It is assumed that the advanced waste treatment includes biological nitrification/denitrification, phosphorus removal (by alum, ferric chloride, or lime), filtration, activated carbon, and disinfection.

Table 5

Summary of Cost Functions

Unit Process	Capital Cost, $	O&M Costs, $/Year
Reservoir	$24,300S^{0.54} + 1.30S^{0.87}K$	$243S^{0.54}$
Wells	See Table 7	4% of Costs in Table 7
Vertical Turbine Pumps	$1707 + 302Q^{0.453}H^{0.642}$	$15Q^{0.453}H^{0.642} + 45.8Q\ H$
Submersible Turbine Pumps	$414Q^{0.541}H^{0.658}$	$20.7Q^{0.541}H^{0.658} + 45.8Q\ H$
Low Lift Pump Station	$58,400 + 207,000Q^{0.746}$	$5,200 + 3,100Q^{0.829}$
Pipe Line (Underground)	$(1.01 \times D^{1.29})L$	$.00637\ D^{1.29}L$
Retrofitting Pipeline	$(2.0 \times 1.01 \times D^{1.29})L$	$.00637\ D^{1.29}L$
Aeration (For Iron Removal)	$11,800Q^{0.736}$	$5,640Q$
Flash Mix and Coagulation	$146,000Q^{0.685}$	$25,600Q$
Sedimentation	$113,000Q^{0.893}$	$18,100Q$
Filtration (4 gpm/sq ft)	$464,800Q^{0.63}$	$24,300Q^{0.68} + 2,000$
Reverse Osmosis	$150,000 + 890,000Q^{0.867}$	$10,000 + 210,000Q^{0.954}$
Chlorination Equipment	$5,800 + 7,700Q^{0.822}$	$2,900 + 7,100Q^{0.703}$
Chlorine Contact Tank	$3,000 + 28,000Q^{0.725}$	$300Q^{0.725}$
Ozonation	$126,000Q^{0.745}$	$6,300Q^{0.745} + 3,800Q$
Water Storage Tank (On Ground)	$468,000V^{0.721}$	$7,000V^{0.721}$
Water Storage Tank (Elevated)	$877,000V^{0.599}$	$13,200V^{0.599}$
Microscreening	$21,900 + 148,000Q^{0.971}$	$100 + 22,300Q^{0.973}$
Carbon Adsorption		
Without Regeneration (< 3 mgd)	$731,000Q^{0.743}$	$6,300 + 116,000Q^{0.80}$
With Regeneration (> 3 mgd)	$880,000Q^{0.772}$	$23,000 + 58,000Q^{0.75}$
Nitrification (With Clarifier)	$137,000 + 410,000Q^{0.821}$	$2,700 + 6,700Q^{0.74}$
Denitrification (With Clarifier)	$100,000 + 298,000Q^{0.655}$	$5,200 + 35,000Q^{0.90}$
Break Point Chlorination	$299,000Q^{.523}$	$140,000Q^{0.789}$
Dechlorination	$1505 + 26,700Q^{0.44}$	$2,000 + 4,900Q^{0.65}$
Phosphorus Removal by Alum Addition	$33,700 + 12,800Q^{1.10}$	$4,700 + 24,600Q$
Phosphorus Removal by $FeCl_3$ Addition	$36,100 + 7,020Q^{1.14}$	$4,700 + 16,900Q$
Two-Stage Lime Treatment	$99,200 + 515,000Q^{0.65}$	$10,000 + 35,500Q^{0.899}$
Alum Sludge	$130,000 + 320,000Q^{0.550}$	$17,000 + 14,000Q^{0.741}$
Lime Sludge	$188,000 + 293,000Q^{0.740}$	$14,000 + 58,000Q^{0.905}$
Nitrification Sludge	$98,000 + 64,000Q^{0.709}$	$3,300 + 7,800Q^{0.865}$
Denitrification Sludge	$52,400 + 35,200Q^{0.74}$	$1,900 + 4,000Q^{0.873}$
Alum Phosphate Sludge	$199,000 + 67,000Q^{0.843}$	$6,500 + 16,300Q^{0.854}$
Ion Phosphate Sludge	$254,000 + 85,600Q^{0.843}$	$8,300 + 20,800Q^{0.854}$
Recharge		
By Well	Same as Wells	
By Basin	Same as Reservoir	

Notes: 1. Costs are adjusted to September 1976, ENR Construction Cost Index = 2,480.

2. S = Storage Capacity of Reservoir, Acre-Feet.
K = Land Cost, $/Acre.
Q = Flow, mgd.
H = Total Dynamic Head, Feet.
D = Diameter of Pipe, Inches.
L = Length of Pipe, Feet.
V = Capacity of Water Storage Tank, Million Gallons.

Table 6

Cost Functions for Wells

Type of Well	Bottom Bore Hole Diam. (D) inches	Depth of Well (d) feet	Capital Cost Original (1966) $	Capital Cost Adjusted (Sept. 1976) $
A. Tubular Wells Finished in Sand and Gravel[2]:	6 - 10	35 - 250	$800\ d^{0.299}$	$1710\ d^{0.299}$
	12 - 15	50 - 220	$850\ d^{0.373}$	$1820\ d^{0.373}$
B. Gravel Packed Wells Finished in Sand and Gravel[1]:	16 - 20	50 - 350	$680\ d^{0.408}$	$1460\ d^{0.408}$
	24 - 34	50 - 220	$680\ d^{0.482}$	$1460\ d^{0.482}$
	36 - 42	35 - 320	$890\ d^{0.583}$	$1900\ d^{0.583}$
C. Shallow Sandstone, Limestone, or Dolomite Bedrock Wells:	6	140 - 400	$0.578\ d^{1.413}$	$1.24\ d^{1.413}$
	8 - 12	200 - 600	$0.839\ d^{1.450}$	$1.80\ d^{1.450}$
	15 - 24	160 - 450	$1.781\ d^{1.471}$	$3.81\ d^{1.471}$
D. Deep Sandstone Wells[2]:	8 - 12	600 - 2500	$0.029\ d^{1.870}$	$0.062\ d^{1.870}$
	15 - 19	900 - 2000	$1.314\ d^{1.429}$	$2.81\ d^{1.429}$

[1] For Gravel Packed Wells: Bore Hole Diameter = Well Diameter + 2 x Gravel Pack Annulus.
[2] Other Types of Well: Bore Hole Diameter = Well Diameter.

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SECTION 4

OPTIMIZATION IN DESIGN OF PUMPING SYSTEMS

OPTIMIZATION IN DESIGN OF PUMPING SYSTEMS*

INTRODUCTION

The use of a pumping system for conveying fluids is universal. A significant amount of energy is spent in transporting liquids or gases through pipelines in industries, power stations, farms, residences, water and wastewater treatment plants, water distribution systems, and many other uses. With the increase of energy cost in recent years and prospective increase in the future, optimizing a pumping system gained recognition.

In the design of a pumping system, the costs of the system consist of the cost of pumps, pipes, operation and maintenance, and energy. The size of the pipe is an important factor in the whole pumping system. The pump size is also largely dependent on the size of the pipeline. Smaller diameters of pipe will result in large friction heads, requiring larger pumps and more energy. Again with the increase of pipe size, energy cost will be decreased but the costs of pipelines will be increased. For any pumping system there exist optimum sizes of pipeline and pump. In this paper a comprehensive econo-mathematical model of a pumping system, incorporating capital, O & M, and energy cost functions, has been developed to seek a minimum total cost of the system.

*Deb, A.K., "Optimization in Design of Pumping Systems," Proc. ASCE, J. EED, Vol. 104, February, 1978.

ANALYSIS

In developing an econo-mathematical model of a pumping system, the total cost is divided into two parts: capital costs and O & M costs (i.e. maintenance and energy costs).

Total capital cost of an installed pipeline (including laying, jointing, etc.) can be expressed as a function of diameter:[1,2,3]

$$y_1 = k_1 D^{m_1} \tag{1}$$

in which y_1 is the capital cost of the pipeline and D is the diameter of the pipeline, k_1 is a coefficient, and m_1 is an exponent of the cost function. On the basis of 1976 cost data, when y_1 is expressed in dollars/ft length and D is in inches, the value of $k_1 = 1.01$ and $m_1 = 1.29$. The capital cost of an installed pump was expressed as a function of flow and total head:[5]

$$y_2 = k_2 H^{m_2} Q^{m_3} \tag{2}$$

y_2 is the capital cost of the installed pumps in dollars, k_2 is the coefficient, Q is the flow rate in gpm, H is the total head in feet (it is equal to summation of static and friction heads), and m_2 and m_3 are exponents of the cost function. On the basis of U.S. cost data updated to 1976, the values of $k_2 = 16.14$, $m_2 = 0.642$ and $m_3 = 0.453$. The annual cost of energy is related to the unit cost of energy, operating head, and flow rate; indirectly it is related to the horsepower of the pump. The horsepower (HP) of the pump can be expressed when transporting water as:

$$HP = \frac{Q(H_s + H_f)\,8.33}{33000\,E_p} \tag{3}$$

in which Q is the flow in gallons per minute; H_s is the static head in feet and H_f is the friction head; and E_p is the pump efficiency.

In water distribution system pumping, maximum daily flow is usually considered in calculating the horsepower of the pump.

Friction water head, H_f, is dependent on the flow through the pump, the pipeline diameter, and the length of the pipeline. The relationship between friction head loss in pipe and the flow through it can be given by many formulas. For water supply, the Hazen-Williams equation is widely used and can be given as

$$H_f = \frac{L\,Q^p}{0.0955\,C^p\,D^q} \tag{4}$$

H_f is the friction head loss in feet, L is the length of pipeline in feet; Q is the flow in gallons per minute; C is the Hazen-Williams coefficient; D is the diameter of the pipeline in inches; p = 1.85 and q = 4.86.

The value of Hazen-Williams coefficient, C, is not constant; for a new cast iron pipe the value of C may be about 130, and for old pipe C = 100. For the calculation of design horsepower, HP, of the pumps it is rational to consider the value of C at the end of the design life (that is, C = 100 for cast iron pipe). For other pipes appropriate values of C should be taken.

Since the pipe condition will deteriorate during the life period, it is appropriate to assume that the C value will decrease linearly with the age of the pipeline. Equation (4) can be rewritten to accommodate a variable C as follows:

$$H_f = \frac{L\,Q^p}{0.0955\,(C_N - r)^p\,D^q}$$

in which

C_N = Hazen-Williams coefficient for a new pipe

r = number of years of operation.

For the calculation of annual electrical energy cost the average of the C values during the design period may be taken. If a head loss equation other than the Hazen-Williams equation is used, the C value will be replaced by the appropriate friction coefficient. Combining Equations (2) and (4), the capital cost of pumps can be expressed as:

$$y_2 = k_2\,Q^{m_3}\left[\,H_s + \frac{L\,Q^p}{0.0955\,C^p\,D^q}\right]^{m_2} \tag{5}$$

Average annual energy cost can be calculated using the equation for horsepower of the pump (Equation 3) as

$$Y_3' = \frac{0.746\,Q\,24\,c\,365\,(8.33)\,(H_s + H_f)}{33000\,E_m\,E_p} \tag{6}$$

in which c is the cost of energy per Kw-hr and E_m is the mechanical efficiency of the pump.

Combining the Hazen-Williams equation for head loss (Equation (4)) with Equation (6), the annual energy cost Y_3' can be expressed as:

$$Y_3' = \frac{0.746\, Q\, 24\, c\, 365\, (8.33)}{33000\, E_m\, E_p} \left(H_s + \frac{L\, Q^p}{0.0955\, C^p\, D^q}\right) \qquad (7)$$

The operation and maintenance other than energy cost can be assumed to be f times the energy cost. Thus the total annual O & M cost, including energy cost, can be written as:

$$Y_3 = \frac{(1 + f)\,(0.746)\, Q\, 24\, c\, 365\, (8.33)}{33000\, E_m\, E_p} \left(H_s + \frac{L\, Q^p}{0.0955\, C^p\, D^q}\right) \qquad (8)$$

For water pumping systems, the value of f has been suggested by Linaweaver[1] as equal to 0.08. To obtain the total annual cost of the system, the capital costs are converted into annual capital recovery cost considering an interest rate of i and repayment period of n years (useful life of equipment) and added to the annual maintenance and operation costs. If the capital is to be paid in equal annual payments during the life period (n years) of the equipment at an interest rate of i, and considering s as the salvage value of the equipment at the end of the useful life as a ratio of actual value, the annual capital recovery factor is expressed as:

$$R = \frac{i\,(1 + i)^n}{(1 + i)^n - 1}\,(1 - s) + i\,s \qquad (9)$$

where R is the annual capital recovery factor.

Considering the useful life of a pipeline as n_1 years, a salvage value factor s_1, rate of interest i, and annual capital recovery R_1, then the initial cost of the pipeline Y_1 can be obtained by combining Equations (1) and (9):

$$Y_1 = y_1 \; L \; R_1$$

$$= k_1 \; D^{m_1} L \left[\frac{i\,(1+i)^{n_1}}{(1+i)^{n_1} - 1}\,(1 - s_1) + i\,s_1\right] \qquad (10)$$

Similarly, the initial cost of pumps can also be converted to annual capital recovery cost by combining Equations (5) and (9).

$$Y_2 = y_2 \; R_2$$

$$= k_2 \left[\frac{i\,(1+i)^{n_2}}{(1+i)^{n_2} - 1}\,(1 - s_2) + i\,s_2\right] \; Q^{m_3} \left(H_s + \frac{L\,Q^p}{0.0955\,C^p\,D^q}\right)^{m_2} \qquad (11)$$

Now, the total annual cost of the pumping system is the summation of Y_1, Y_2 and Y_3, which can be obtained from Equations (8), (10), and (11) as follows:

$$Y = Y_1 + Y_2 + Y_3$$

$$= R_1 \; k_1 \; D^{m_1} \; L$$

$$+ R_2 \; k_2 \left[Q^{m_3} \left(H_s + \frac{L\,Q^p}{0.0955\,C^p\,D^q}\right)^{m_2}\right] + \qquad (12)$$

$$+ \frac{(1+f)(0.746)\,Q\,24\,c\,365\,(8.33)}{33000\,E_m\,E_p} \left[H_s + \frac{L\,Q^p}{0.0955\,C^p\,D^q}\right]$$

Equation (12) can be rewritten as

$$Y = R_1 \; k_1 \; D^{m_1} \; L + R_2 \; k_2 \left[Q^{m_3} \left(H_s + \frac{XL}{D^q}\right)^{m_2}\right]$$

$$+ k_3 \; F \left(H_s + \frac{XL}{D^q}\right) \qquad (13)$$

where

$$F = \frac{Q}{33000\, E_p}$$

$$X = \frac{Q^p}{0.0955\, C^p} \text{ and } k_3 = \frac{(1+f)(0.746)\, 24\, c\, 365\, (8.33)}{E_m} = \frac{(1+f)\, 54436\, c}{E_m}$$

Equation (13) represents the total annual cost of the pumping system. For a given flow Q, length L, and static pumping head H_s, the total annual cost of the pumping system can be obtained in terms of pipeline diameter using Equation (13).

To obtain optimum diameter of the pipeline, Equation (13) can be differentiated with respect to diameter D. Equating this to zero yields:

$$\frac{dY}{dD} = R_1\, k_1\, L\, m_1\, D^{m_1 - 1}$$

$$- R_2\, k_2\, m_2 \left[Q^{m_3} \left(H_s + \frac{XL}{D^q}\right)^{m_2 - 1} \right] (q\, X\, L D^{-(q+1)}) \tag{14}$$

$$- q\, F\, k_3\, XL\, D^{-(q+1)} = 0$$

By solving Equation (14), the optimum diameter of the pipeline of the pumping system can now be obtained. But Equation (14) can be solved only by trial and error. By sensitivity analysis, however, it was found that the second term on the right-hand side of Equation (14) does not have a significant effect on the value for optimum diameter. Therefore, the second term of Equation (14) can be neglected to obtain the optimum diameter of the pipeline:

$$\frac{dY}{dD} = R_1\, k_1\, L\, m_1\, D^{m_1 - 1} - q\, F\, k_3\, XL\, D^{-(q+1)} = 0 \tag{15}$$

The value for optimum diameter obtained from the simplified equation (i.e., Equation (15)), is not significantly different from the value for optimum diameter achieved through trial-and-error solution of Equation (14). Using the optimum diameter from Equation (14) or Equation (15) will not affect the final selection of pipe size because pipe available commercially comes only in fixed sizes.

Equation (15) can now be solved for diameter D;

$$D = \left[\frac{q\,F\,k_3\,X}{R_1\,k_1\,m_1}\right]^{\frac{1}{m_1 + q}} \tag{16}$$

Replacing the values of F and X and expressing in terms of flow Q, Equation (16) can be rewritten as

$$D = K\,Q^{(p+1)/(m_1+q)} \tag{17}$$

in which

$$K = \left[\frac{q}{R_1\,k_1 m_1}\;\frac{1}{33000\,E_p}\;\frac{k_3}{0.0955\,C^p}\right]^{\frac{1}{m_1 + q}}$$

Equation (17) results in an optimum diameter of a pipeline of a pumping system, a diameter which will produce the minimum total cost (capital and O & M) of the system.

The optimum size of the pipe as calculated using Equation (16) may be a fractional size, and may not be available in the market place. In order to use the Equation (16) concept to select a practical optimum diameter of a water supply system pumping main, available pipe sizes will be given as input to the computer, and the least-cost practical pipe size will be selected.

The theoretical horsepower of the pump, obtained using Equation (3) will be converted to design horsepower by multiplication of a standby factor which can be determined from the following relationships[6].

$$\left.\begin{array}{rl} Q' \leqslant 2.0 & AJ = 2.08 - 0.18Q \\ 2.0 < Q' \leqslant 5.0 & AJ = 1.9666 - 0.1233Q \\ 5.0 < Q' \leqslant 10.0 & AJ = 1.42 - 0.014Q \\ 10.0 < Q' \leqslant 20.0 & AJ = 1.30 - 0.002Q \\ 20.0 < Q' & AJ = 1.25 \end{array}\right\} \quad (19)$$

where, Q^1 = Flow in million gallons per day.

SOLUTION

To illustrate the applicability of the method of analysis developed, one example of a pumping system with the following data is considered. Flow rate = 1500 gpm of water; length of pipe = 6000 feet; static head to be pumped = 160 feet of water; rate of interest i = 0.06; useful life of pipeline n_1 = 30 years; useful life of pumps n_2 = 15 years; salvage value ratio for pipeline, s_1 = 0.1; salvage value ratio for pumps s_2 = 0.1; Hazen-Williams coefficient = 100; p = 1.85; q = 4.86;

E_p = 0.8; E_m = 0.8; energy cost, c = \$0.03 per kilowatt-hr; additional cost of maintenance as a fraction of energy cost, f = 0.08; exponents of cost functions k_1 = 1.01; m_1 = 1.29; m_2 = 0.642; m_3 = 0.453; k_2 = 16.14.

Using these data in Equation (12), the annual costs of pipeline Y_1, pumps Y_2, and O&M Y_3 have been calculated for various values of pipeline diameter and plotted as costs versus diameter in Figure 1. The optimum diameter of the pipeline of the pumping system has been calculated using Equation (16) and found to be 13.8 inches. Figure 1 also shows that the optimum diameter is little less than 14 inches.

For a water transportation system using pumps and pipeline, the optimum diameter of a pipeline is found to be proportional to $Q^{(p + 1)/(m_1 + q)}$ (Equation 17). For p = 1.85, m_1 = 1.29 and q = 4.86, the optimum diameter is proportional to $Q^{0.463}$. Equation (16) has also been solved for various interest rate (i) values, and the optimum diameter of a pipeline is found to decrease with the increase of interest rate. The theoretical horsepower required, calculated by using Equation (3), is 17.78; to obtain the design horsepower, this is multiplied by the standby factor: 1.7 x 17.78 = 30.21 HP.

SUMMARY AND CONCLUSIONS

A method of pipe size optimization in a pumping system for known flow has been developed, incorporating various cost functions. The method

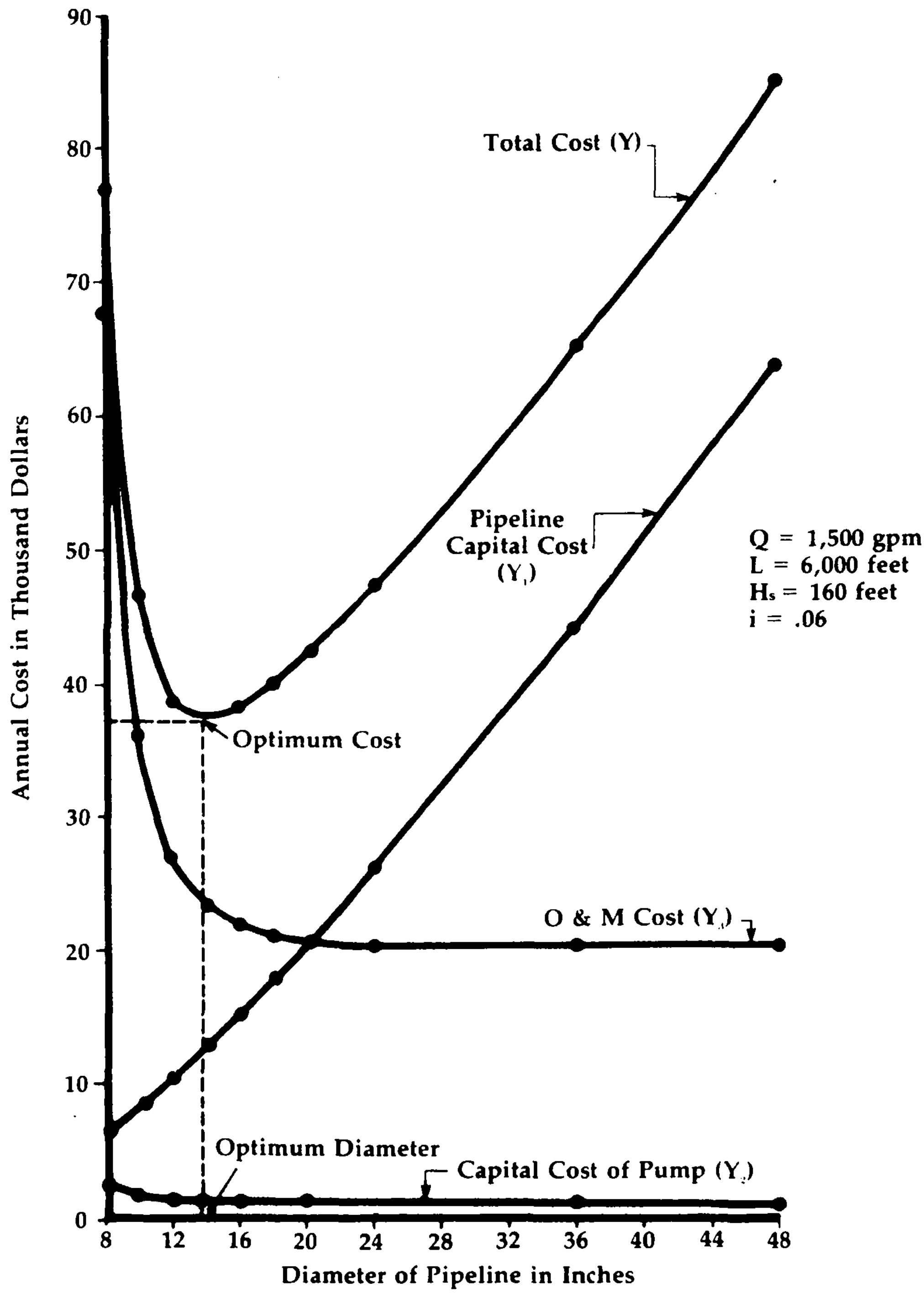

FIGURE 1 VARIATION OF COSTS WITH PIPELINE DIAMETER

has been extended to determine the optimum timing of phasing a pumping system with growing demand rate. The cost sensitivity of the system with the various parameters has been studied, and a comparatively simple solution for optimum size of a pipeline in a pumping system has been suggested.

In finding an optimum pipe size in a pumping system, it is always advisable to check the validity of the model as described in the paper under the circumstances of application. In this connection the following points may be noted: 1) the coefficients and exponents of the cost functions used to derive mathematical model should be valid for the locality of use; 2) values of variables such as interest rate, useful life of equipment, salvage value, and head loss equation coefficient should be properly selected; 3) in the case of pumping of fluids other than water, appropriate head loss equation, viscosity and density values should be taken.

APPENDIX -- NOTATION

c = cost of energy

C = coefficient of Hazen-Williams equation

C_c = rate of increase of capital cost

C_N = Hazen-Williams coefficient for new pipe

D = diameter of pipe

DF = demand factor

E_m = mechanical efficiency of pump

E_p = pump efficiency

f = ratio of O & M cost to energy cost

F = simplifying parameter

h_f = friction water head

H = total head

H_f = friction head

H_s = static head

HP = horsepower of a pump

k_1, k_2 = coefficients of cost functions

K = simplifying parameter

L = length of pipeline

m_1, m_2, m_3 = exponents of cost functions

n = useful life of equipment

n_1 = useful life of pipeline

n_2 = useful life of pumps

p, q = exponents of Hazen-Williams equation

Q = flow rate

r = number of years of operation

R = annual capital recovery factor

s = salvage value ratio

s_1 = salvage value ratio for pipeline

s_2 = salvage value ratio for pumps

t = number of years from installation

u = simplifying parameter

w = rate of increase in water demand

W_o = initial water demand

W_n = water demand at the n-th year

X - simplifying parameter

y_1 = capital cost of pipeline

y_2 = capital cost of pumps

Y_1 = initial cost of pipeline

Y_2 = total annual capital costs of pumps

Y_3 = total annual O & M costs

Y_3' = average annual energy cost

Z = cost of pipeline per unit length

Z_1 = present value of first pipe per unit length

Z_2 = present value of second pipe per unit length

Z_D = unit cost of two-pipe system

Z_r = ratio of unit costs of one- and two-pipe systems

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SECTION 5

SYSTEMS MODEL DEVELOPMENT

SYSTEMS MODEL DEVELOPMENT

INTRODUCTION

Increased demands for good quality water increase the necessity for better management and planning of water systems. Water pollution abatement requirements have resulted in the production of wastewater effluents of such quality that they may be considered sources of water for subpotable and nonpotable uses. Where good quality water is not available, potable water would need to be produced from unprotected sources after extensive treatment to meet primary and secondary drinking water regulations.

Therefore, in areas where good quality water is not available or there is an overall shortage of water, developing a long term economic water management plan will be complex. It might be necessary to consider more than one source of water (including reuse), and more than one distribution system to supply water for potable and other uses. This study intends to develop a comprehensive and easy-to-use systems model for analysis of long term urban water supply planning using single, dual or multiple supply concepts.

A conceptual multiple supply urban water system model is shown in Figure 1. A model of the quantity and quality aspects of an urban water network would incorporate three basic components:

1. Water sources including recycled water.

2. Water demand by quality.

3. Water treatment (including treatment of wastewater after secondary treatment) and distribution (including more than one distribution system).

The model outline, shown in Figure 1, may have numerous options, depending on the quality and quantity of sources, potable, subpotable and nonpotable water demand, and institutional and political considerations. This model is intended to provide a methodology of technical and economical analyses of urban water systems. It will also provide the user with a tool to analyze various water supply system alternatives, including multiple distribution, depending on the composition of sources and demands. At present, this model can only be used as a planning tool rather than a design tool.

Depending on the sources and demand requirements, the user must identify the physical systems to be analyzed, proceeding from source to distribution. Several alternative systems should be analyzed in order to determine a feasible solution.

WATER RESOURCES

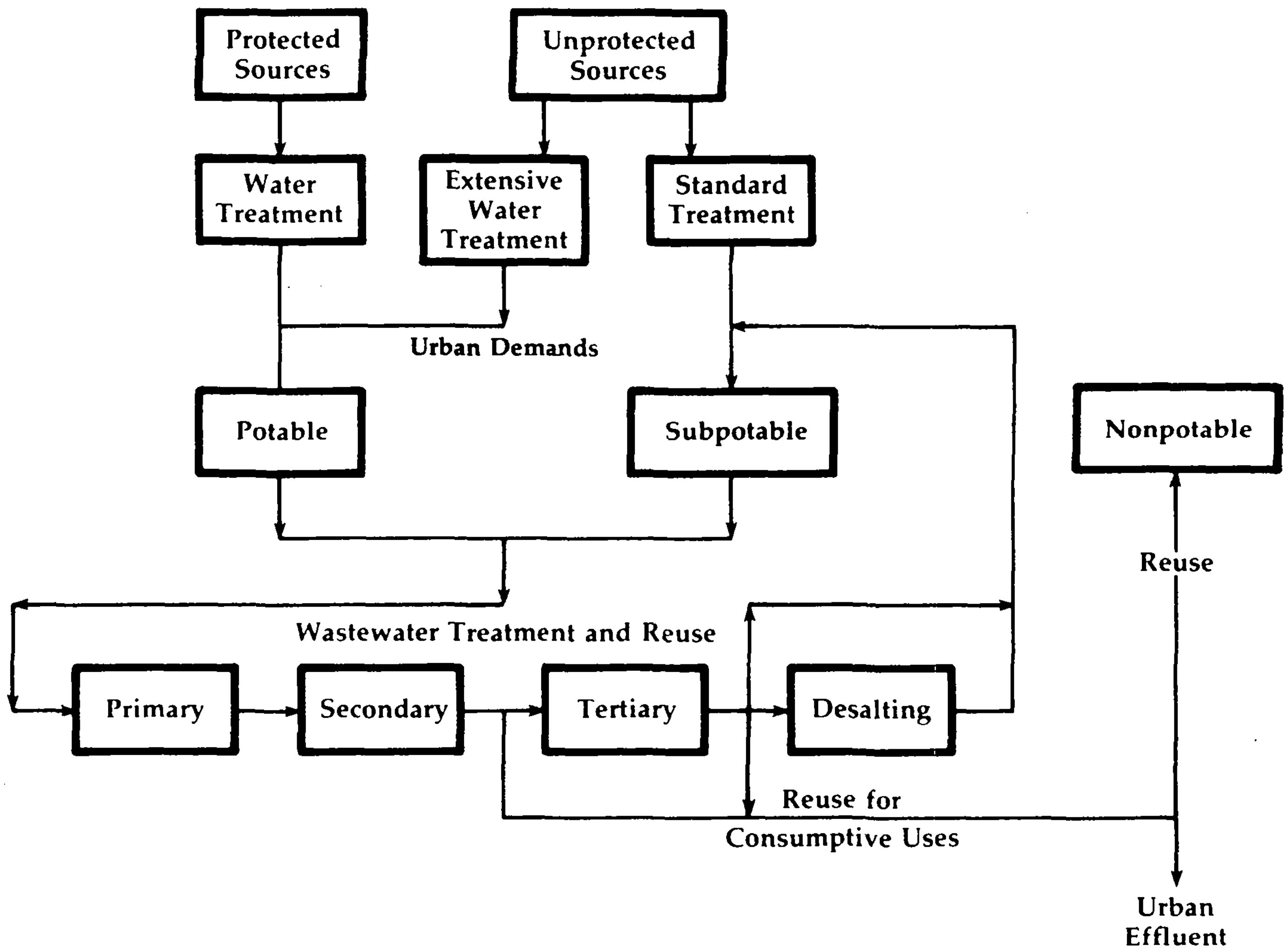


FIGURE 1 MULTIPLE SUPPLY URBAN WATER SYSTEM

Components of the systems model have been outlined in Figure 2.

In analyzing a multiple supply system, based on the sources and treatment requirements, the potable and subpotable system may be completely or partially separate. Total present worth of collection, transmission, treatment and distribution of a water system of any configuration for single or multiple supply can be calculated using this model.

In formulating the mathematical model, parameters such as potable to total flow ratio, interest rate, annual capital and operation and maintenance (O&M) cost increase rates, and cost function are considered as variables. In order to compare the present worth of the conventional system with the multiple system, the quality of water from a single supply is assumed to be the same as that of the potable supply from a multiple system.

DEMAND DEVELOPMENT

In order to analyze a water system, it is essential to project water demand during the planning period. In case of a multiple supply system, the projected demand for potable, subpotable or nonpotable supplies is required. In a previous presentation, details of various demand projections were analyzed.

PHYSICAL COMPONENTS OF THE SYSTEM

Any water supply system, from source to the distribution system, consists of collection and transmission, treatment, and distribution units. A list of the units used in the systems model is given in Table 1, along with their service life periods. A water supply system of any configuration incorporating the units listed in Table 1 can be analyzed using the model.

Collection and Transmission

Unit C1 - Reservoir Impounded

The impounded reservoir cost function is related to the capacity of the reservoir. Capacity of the reservoir can be related to the net yield of the reservoir.

$$S = 587.26\ Q^{1.085}$$

where:

S = Capacity of reservoir in acre-ft.

Q = Net yield of reservoir in mgd.

FIGURE 2

BASIC COMPONENTS OF SYSTEMS MODEL

1. **Demand Development During Planning Period**
   - Potable Demand
   - Subpotable Demand
   - Nonpotable Demand
2. **Preliminary Design of Physical Components**
   - Collection and Transmission
   - Treatment
   - Distribution
     - Single
     - Multiple
     - Bottled
3. **Cost of Components**
   - Present Worth at Base Year
     - Capital Costs
     - O & M Costs
     - Replacements Costs
     - Salvage Value
   - Total Present Worth

Table 1

Various Unit Processes Considered

Collection and Transmission

		Service Life In Years
C1	Reservoir Impounded	60
C2	Wells	25
C3	Vertical Turbine Pumps	15
C4	Submersible Turbine Pumps	15
C5	Low-Lift Pump Station	15
C6	Transmission Pipeline & Pumping	30
C7	Intake Tower	30
	Treatment	
T1	Aeration	30
T2	Flash Mix and Coagulation	30
T3	Sedimentation	30
T4	Filtration	30
T5	Chlorination Equipment	15
T6	Chlorine Contact Tank	40
T7	Ozonation	30
T8	Reverse Osmosis	15
T9	Carbon Adsorption Without Regeneration	35
T10	Carbon Adsorption With Regeneration	35
T11	Microscreening	20

Table 1
(continued)

Treatment (continued)

		Service Life In Years
T12	Nitrification (with Clarifier)	40
T13	Denitrification (with Clarifier)	40
T14	Breakpoint Chlorination	15
T15	Dechlorination	15
T16	Phosphorus Removal by Alum Addition	20
T17	Phosphorus Removal by $FeCl_3$ Addition	20
T18	Lime Treatment	40
T19	Point of Use Treatment	10
	Residual Handling and Disposal	
R4	Alum Sludge	25
R2	Lime Sludge	25
R3	Nitrification Sludge	25
R4	Denitrification Sludge	25
R5	Alum Phosphate Sludge	25
R6	Iron Phosphate Sludge	25
	Distribution System	
D1	High Lift Pumping Station & Pumping Main	15
D2	Water Storage Tank (on Ground)	40
D3	Water Storage Tank (Elevated)	40
D4	Distribution Mains	30
D5	Additional Interior Plumbing and Meter Cost	25
D6	Recharge of Ground Water	
	By Wells	25
	By Basins	35

Unit C2 - Wells

Well cost varies with the type and depth of the wells. Four types are considered in this study, with the following input information required to calculate well cost:

- Type of well
- Well diameter
- Depth of well
- Long-term average well yield

In designing the number of wells, this rule has been applied:

If $N < 3$, Number of wells provided = $N + 1$

If $N > 3$, Number of wells provided = $N + 2$

Units C3 and C4 - Vertical Turbine Pumps or Submersible Pumps

These guidelines have been used in selecting pumps for the wells:

If yield of a well < 0.15 mgd, use submersible pump

If yield of a well > 0.15 mgd, use vertical turbine pump

The number of pumps required has been assumed to be equal to the number of wells. The following information is necessary in selecting size and energy cost:

- Long term pumping level below ground level
- Head for pumping from ground level to the storage or treatment

Units C5 and C7 - Low-Lift Pump Station and Intake Tower

Low-lift pump station and intake tower cost functions are directly related to the average flow, and therefore capital and O&M costs are directly calculated from cost functions.

Unit C6 - Transmission Pipeline and Pumping

An optimum design procedure for transmission pipeline and pumping has been developed and is discussed in a separate section.

Treatment Units

The costs of all treatment units, including residual handling and disposal units, are expressed in terms of average flow rates. Capital costs are calculated on the basis of design flow to be encountered at the end of service life of the unit during the planning period. If the unit service life is greater than the planning period, the design flow is considered as the flow at the planning period. For O&M cost calculation, average daily flow for the year is considered, and O&M costs increase with the increase of demand during the planning period.

Distribution System

Unit D1 - High-Lift Pumping Station and Pumping Main

The same optimum design procedure developed for raw water pumping and transmission is used for treated water pumping and pumping main design. A detailed discussion is given in a separate section.

Units D2 and D3 - Water Storage Tank (On Ground and Elevated)

Both capital and O&M cost functions for storage tanks are expressed as functions of storage capacity. Using a regressional analysis of data from various water authorities, the following relationship of storage required and average daily pumpage is obtained. The plot of data is shown in Figure 3.

$$S = 1.054\ P^{1.0834}$$

where:

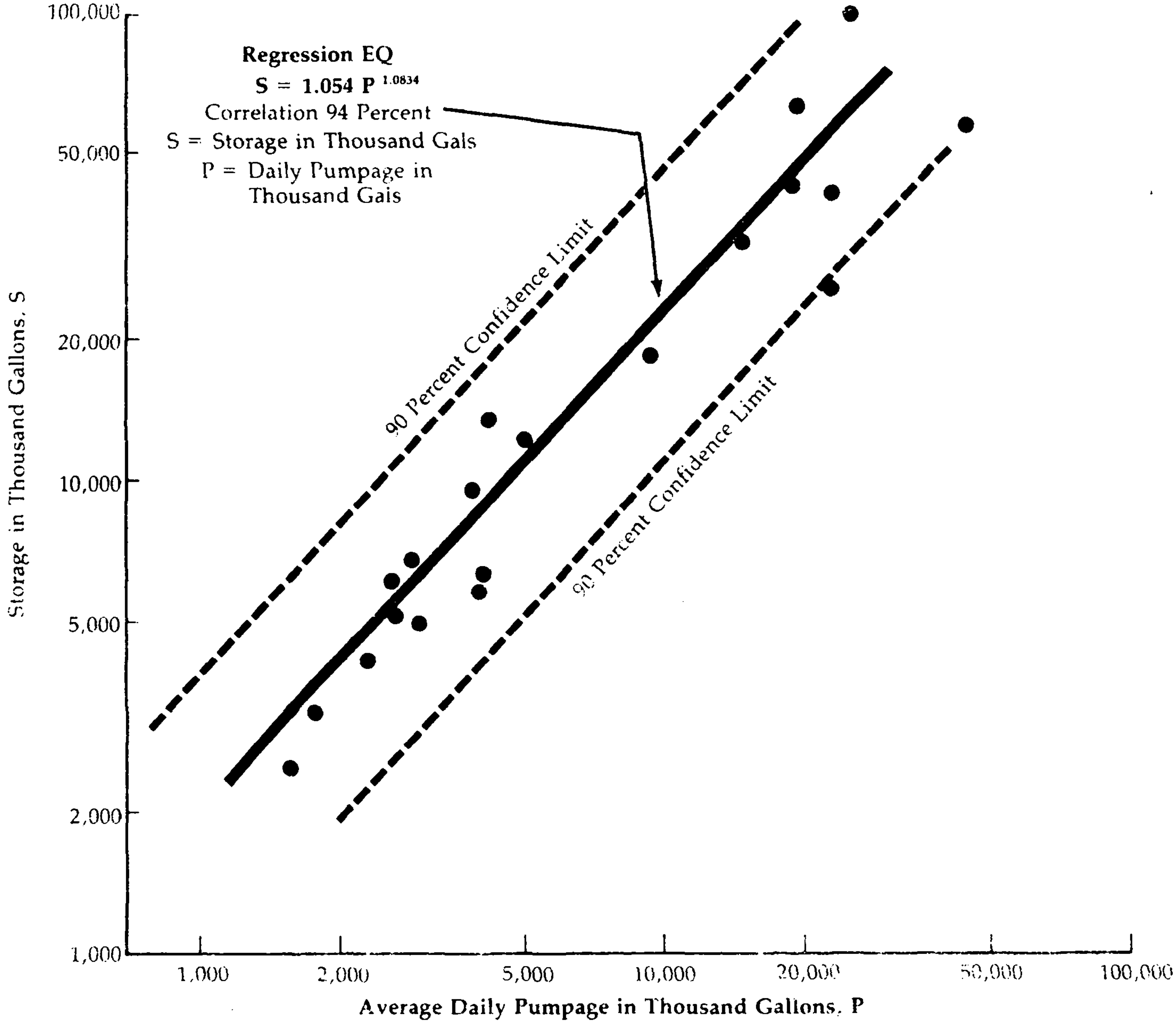
S = Storage in thousand gallons

P = Daily pumpage in thousand gallons

Unit D4 - Distribution Main System

The distribution main system consists of all pipes and appurtenances in the water distribution system from service reservoirs to consumers. If the lengths and diameters of all pipes in a system are known, capital cost of the total distribution system can be calculated. If the lengths and diameters of pipes are not known, as in the case of a new city, a simplified procedure has been developed to estimate the total lengths of pipes and average cost diameter for the city. This average cost diameter can be defined as the diameter of a pipe the length of which is equal to the total length of distribution main, and the cost is the same as for a total distribution system.

FIGURE 3 TREATED WATER STORAGE AS A FUNCTION OF AVERAGE DAILY PUMPAGE



The average cost diameter can be expressed as follows:

$$\text{Capital Cost} = 1.01\ d_{av}^{\ 1.29} L_m\ 5280$$

where:

d_{av} = Average cost diameter

L_m = Total length of distribution main in miles

In this study, a simplified method of estimating capital and O&M costs of a distribution system has been developed.

Relationship of Population Density With Water Main Length

The length of water main required in a community water supply is an important parameter in this study. It is expected that the length of water main in a community is dependent on population density of the community. The data of population density per square mile and lengths of water mains in mile per thousand of population of various water utilities surveyed are analyzed. It has been found that the larger the population density the smaller is the main length per thousand population. By analysis of the data the following relationship is obtained:

$$L_m = K_{Lm}\ P_d^{\ -0.458}\ POP_1 \qquad (1)$$

where:

K_{Lm} = Coefficient for length (by regression analysis K_{Lm} = 125.39)

L_m = Main length in miles

P_d = Population density, people/sq mile

POP_1 = Population in base year in thousand

Figure 4 shows plots of main length in miles per thousand population versus population density.

If the population density and population of a town are known, the total length of distribution main can be estimated.

Average cost diameter of the distribution system of a city is found to increase with the population of the city. A plot of average cost diameter of distribution systems of various size cities has been made in Figure 5.

FIGURE 4 MILE OF MAIN LENGTH/1,000 POPULATION VERSUS POPULATION DENSITY IN POPULATION PER SQ. MILE AREA

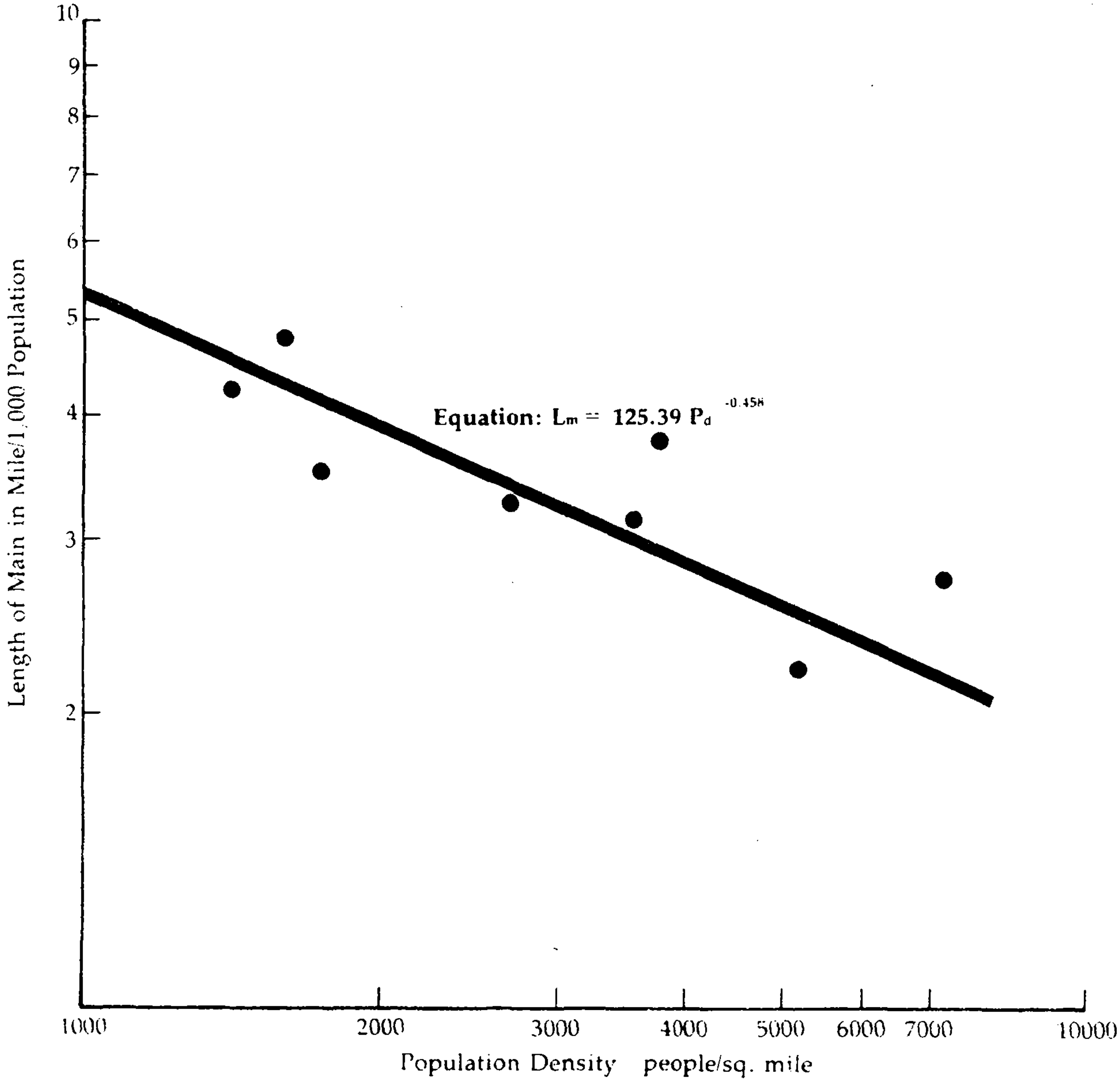
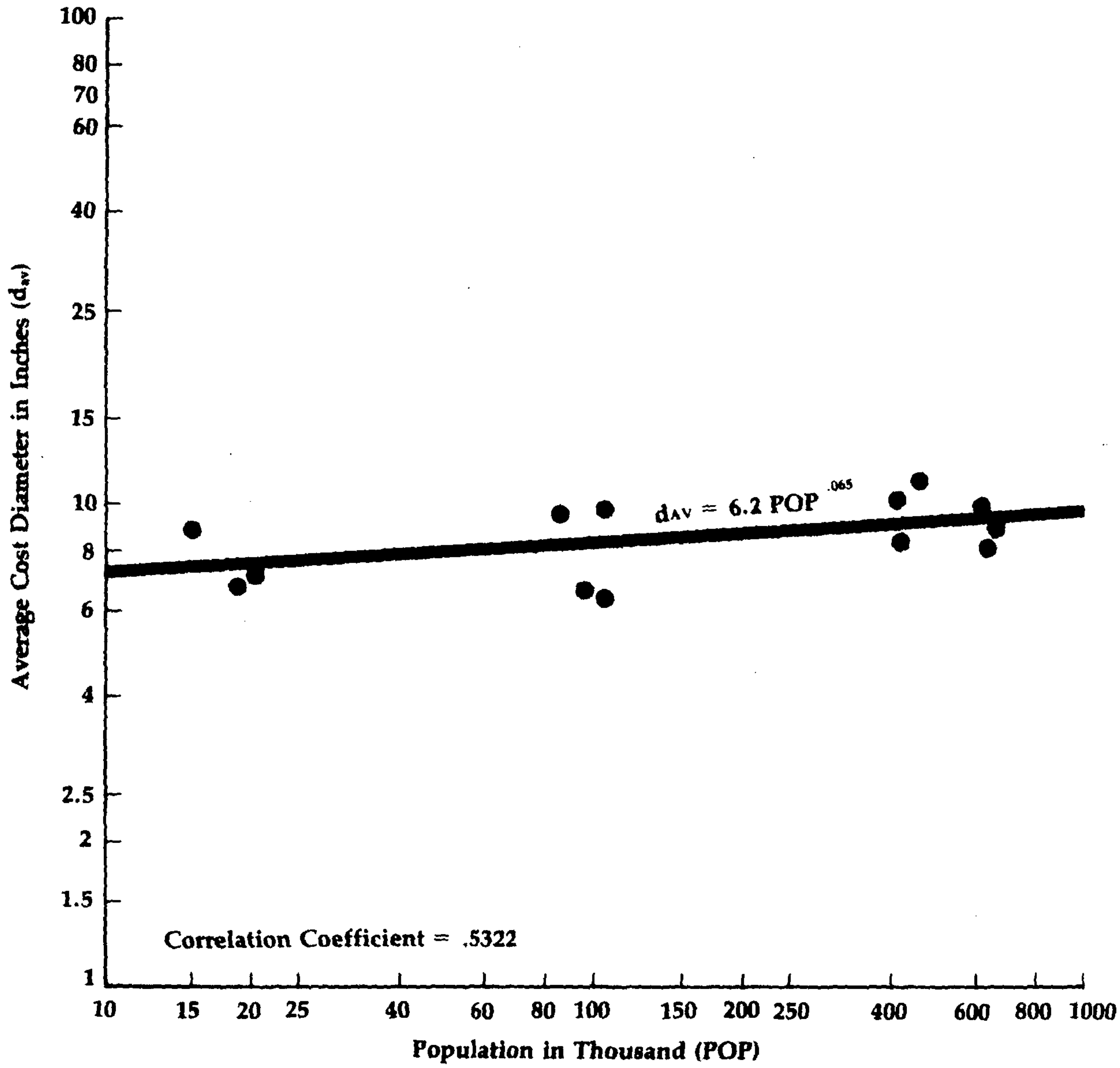


FIGURE 5 AVERAGE COST DIAMETER VS. POPULATION



By regression analysis the following relationship has been established:

$$d_{av} = K_d \, POP_1^{0.065} \tag{2}$$

where:

K_d = Coefficient for average diameter (by regression analysis $K_d = 6.2$)

d_{av} = Average cost diameter in inches

POP_1 = Population in thousands in the base year

Capital Cost of Distribution Mains

From known values of population and population density of a city and using equations (1) and (2), the total capital cost of a distribution system can be calculated using the relationship:

$$\text{Capital Cost} = 1.01 \times 5280 \, (K_d \, POP_1^{0.065})^{1.29} K_{Lm} \, P_d^{-.458} \, POP_1$$

$$= 5332.8 \, K_d^{1.29} \, K_{Lm} \, P_d^{-0.458} \, POP_1^{1.084} \tag{3}$$

In this study by regression analysis of data, the values of K_{Lm} and K_d are found to be 125.39 and 6.2, respectively.

Water mains from service reservoirs to consumers are assumed to be subject to the same hydraulic gradients in potable and subpotable water supply systems. Using the Hazen-Williams equation of pipe flow it can be shown that, for constant hydraulic gradient, the diameter of the pipe is proportional to $Q^{0.38}$.

For a conventional system, once the average cost diameter is calculated using equation (2), the average cost diameter for potable and subpotable systems can be calculated as proportional to respective flows. It is also assumed that for a complete dual supply system the length of potable mains is equal to the length of subpotable mains. Once the lengths and average cost diameters are calculated, the total cost of the distribution system is calculated using pipe cost function.

However, in a city if it is intended to have a partial dual or multiple distribution system, as in the case of supplying subpotable and/or nonpotable water for selected public, commercial and industrial uses, the lengths and sizes of various pipes required should be given as input into the model to calculate the capital cost of distribution system.

Retrofitting Cost of Distribution Mains

Retrofitting cost of laying of distribution mains for a dual or multiple system in an existing city will be higher than the cost of a new system. In order to calculate the capital cost of retrofitting distribution mains, the pipe cost function will be multiplied by a retrofitting factor. The value of this factor depends on the complexity of development of the area and should be given as input to the model. In England, it has been found that the cost of laying a pipe in developed areas is equal to twice the cost of laying the same pipe in open areas[2]. In this model a default value of 2.0 has been adopted as the retrofitting factor. Therefore, the model (Watman) can be used to analyze a complete or partial, dual or multiple distribution system, in a new or retrofitting condition.

Operation and Maintenance Cost of Distribution Mains

Operation and maintenance cost per mile of a distribution system is found to vary with the average cost diameter of the system. The following relationship was obtained using regression analysis of data from water utility companies.

$$\text{O\&M Cost (\$) per year} = 33.63\ K_d^{1.29}\ K_{Lm}\ P_d^{-0.458}\ POP_1^{1.084} \qquad (4)$$

where:

POP_1 = Population in the base year

d_{av} = Average cost diameter in inches

L_m = Length of mains in miles

Once the total length and average cost diameter of any supply system are known, the capital and O&M cost of distribution mains can be calculated. The O&M cost equation (equation 4) is essentially of the same form as capital cost equation (equation 3). In fact, O&M cost is 0.0063 times the capital cost.

Additional Yearly Cost for Extension Distribution Mains

The population growth of a city necessitates construction of new residential buildings involving extension of the existing distribution main lengths. Assuming the population density would remain constant during the planning period, the length and average cost diameter of a distribution system in any year can be calculated for known population using equations (1) and (2). Using equation (3), additional yearly capital costs required in any year, t, during the planning period to sustain population growth can be calculated during the planning period as:

$$\Delta \text{cost}_t = 5{,}332.8\ K_d^{1.29}\ K_{Lm}\ P_d^{-0.458}\ (POP_t^{1.084} - POP_{t-1}^{1.084}) \quad (5)$$

where:

POP_t = Population in the t-th year

POP_{t-1} = Population in the (t-1)-th year

Additional yearly O&M cost due to increases in main length can also be given as 0.0063 times the additional yearly capital cost for the mains.

All these additional yearly capital and O&M costs during the planning period are converted to present worth of the base year of the planning period, considering proper salvage value and inflation rates.

Unit D5 - Interior Plumbing Cost

A typical house with two baths has been used as a model to determine all labor and material cost for installation of water plumbing within the house for conventional (single) supply and dual supply systems. The following two divisions of potable and subpotable water for various household uses in a dual water supply system have been used for calculating the additional cost required for the plumbing change.

1. Cold water potable supply provided in kitchen only. Subpotable water (hot and cold) will be used for rest of the household usage.

2. Subpotable water used only for toilet flushing. Rest of household will use potable (cold and hot) water.

An estimate of costs is given in Table 2. It is apparent that option 1 would cost about \$318 per household and option 2 would cost \$253 more per household to have a dual supply system. An additional meter would also be required. Additional cost of meter including installation is calculated at \$100 per household.

These costs have been considered in the model in comparing costs of conventional and dual supply systems.

With the population increase more houses will be built and additional costs of interior plumbing and meters incurred during the planning period.

An analysis of the water utility survey data revealed that population per residential customer service varies from 3.1 to 5.94. The average number of persons per residential customer is 4.35 with a standard deviation of 0.98.

With the known values of average number of persons per residence and population, number of houses to be built can be calculated, and thus, the additional costs of interior plumbing and metering incurred every year can also be calculated.

These additional yearly costs with inflation and salvage value have also been considered in the systems model.

PRESENT WORTH METHOD OF COST CALCULATION

In calculating the costs of a single supply and a corresponding multiple supply system, all costs incurred during the planning period will be converted to the present worth cost. The total cost includes capital costs, O&M costs, replacement costs and salvage value. This procedure converts these figures over the project life into an equivalent cost representing the current investment required to satisfy all of the identified project costs for the planning period.

The present worth of a system unit cost, Y_t, that would be installed t years after the base year (1976) can be given as:

$$Y_{pw} = \frac{Y_t}{(1 + i)^t} \qquad (6)$$

where:

Y_{pw} = Present worth (base year) of the cost Y_t that would be incurred t years after the base year

i = The annual rate of interest.

Table 2

Interior Residential Plumbing Costs

● Conventional System	$1,122
● Dual System (10 Percent Potable and 90 Percent Subpotable)	
Potable to kitchen only	$ 318
Subpotable, hot and cold to entire household	$1,122
Total	$1,440
Additional Cost $1,440 - $1,122 =	$ 318
● Dual System (60 Percent Potable and 40 Percent Subpotable)	
Potable supply, both hot and cold to entire household	$ 834
Subpotable supply to toilets only	$ 541
Total	$1,375
Additional Cost $1,375 - $1,122 =	$ 253

To include the effect of inflation, assuming a capital cost inflation rate of C_c per year, the cost of this unit after t years would be:

$$Y_t = Y_o \, (1 + C_c)^t \qquad (7)$$

where:

Y_o = Present cost of this unit

Replacing the expression for Y_t of equation (7) in equation (6), the present worth of a future unit cost can be expressed as:

$$Y_{pw} = Y_o \left(\frac{1 + C_c}{(1 + i)} \right)^t \qquad (8)$$

$$= Y_o \, (IF_c)^t$$

where:

IF_c = The inflation factor for capital cost.

If the O&M annual cost increase rate is C_o, a similar expression for one future year's O&M cost can be converted to present worth as:

$$Y_{pw} = Y_o \left(\frac{1 + C_o}{(1 + i)} \right)^t \qquad (9)$$

$$= Y_o \, IF_o^{\,t}$$

where:

IF_o = Inflation factor for O&M cost

The present worth of the total O&M cost during the planning period can be given as:

$$Y_{pw} = \sum_{t=0}^{n} Y_o \left(\frac{1 + C_o}{(1 + i)} \right)^t \qquad (10)$$

where:

n = *Planning period*

Salvage Value

The salvage value represents the value remaining for all capital at the end of the planning period. Considering inflation, the salvage value of a unit cost converted to present worth can be given as:

$$SV_{pw} = \left(1 - \frac{UL}{DL}\right) Y_o \; IF_c^{\,n} \tag{11}$$

where:

SV_{pw} = Present worth of salvage value of a unit of which present day (base year) cost is Y_o

n = Planning period in years

DL = Design life period of the unit in years

UL = Used life period of the unit in years

Replacement Costs

All equipment found within a water-supply system has a finite service life. This service life represents a period of time when a particular equipment item must be replaced. Mechanical equipment such as pumps, chlorinators, chemical feeders, etc. tend to have a low service life, whereas structural equipment, such as sedimentation tanks, filtration units, buildings, etc., a long service life. Appropriate service life was established for all unit processes. The present worth of replacement cost of a unit considering inflation can be calculated using equation (8). This applies only for equipment with a service life shorter than the planning period.

CONCLUSIONS

A systems model for technical and economic analysis of an urban water system having multiple sources and multiple distribution systems has been developed. The model gives the user an effective tool for analyzing alternative water systems for economic long-term urban water management. The model can accept three grades of water in a multiple supply system of any arrangement of sources, transmission, treatment and distribution units.

In using the model for analyzing any urban water system, it is advisable to check the validity of the model as described under the applicable circumstances. In this connection, the following points may be noted:

1. The coefficients and exponents of all unit cost functions used in the system should be valid for the locality of use.

2. Values of variables such as interest rate, useful life of equipment, salvage value, and head loss equation coefficient should be properly selected.

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2. Miller, D.G., Burley, M.J., and Mawer, P.A., "A Survey of Water Supply Costs," Chem. and Ind. No. 21, pp. 673-683, 23 May 1970.

SECTION 6

BOTTLED WATER SUPPLY

BOTTLED WATER SUPPLY

The economic viability of a dual water supply system depends on the opportunities it provides for savings in treatment costs and on the extent of such savings relative to the additional distribution costs involved.

Home delivery of bottled water has been investigated as an alternative to pipe distribution of potable water. In a public system, bottled water would be distributed by trucks for drinking and cooking, and a subpotable supply would be distributed by water mains for all other residential uses.

INTRODUCTION

Bottled water is water that is sealed in glass bottles or other containers and intended for human consumption. At present, it is a privately produced commodity that comes under the Federal jurisdiction of the Food and Drug Administration (FDA). Private bottled water companies either deliver bottled water by truck to homes, businesses, or industries in five-gallon bottles or cartons of six half-gallon returnable bottles, or sell it in supermarkets or similar retail outlets in half-gallon or one-gallon non-returnable containers.

The bottled water industry is a collection of companies that bottle and sell waters of superior quality, principally for drinking purposes. The American Bottled Water Association (ABWA) represents the bottled water industry as its national trade association. ABWA has 250 member companies, who account for 90 percent of the bottled water being sold in this country.

The ABWA members have been working for 18 years under a set of voluntary water quality guidelines. In 1974, FDA established water quality standards similar to the Safe Drinking Water Act, and "Good Manufacturing Practices."

Bottled water customers generally choose to buy bottled water on the basis of taste preference, as an alternative to the municipal water normally supplied to them. Eighty seven percent of the United States bottled water sales are in California, Florida, Illinois, New York, and Texas, with California accounting for more than 50 percent of the overall national sales. Southern California is the largest single market, with one of every six people drinking bottled water; by way of comparison, the national average is about one bottled water user per 2,000 population (1).

The average price of domestic bottled water generally varies from an average of 45 to 70 cents per gallon, with 3 brands having higher prices (79 cents to $1.25). The cost of home-delivered bottled water is about $2 for five gallons, or approximately 700 times the cost of municipal water. European bottled water sold in U.S. markets is in the $1.50-$2.00 per gallon price range.

Processed bottled water when home delivered is generally placed on a stand ready to use. The resident is responsible only for drawing off the water as needed, by means of a push button faucet. The stand may provide cooling and/or heating. Air-evaporative cooling is free, an electric cooler rental costs about $4 per month, and an electric cooler and heater rental costs about $7.25 per month including all maintenance.

BOTTLED WATER DISTRIBUTION ASSUMPTIONS

Few details have been published about the operation of the bottled water industry. In conducting this study, the American Bottled Water Association and 38 large bottled water producers were contacted for information. ABWA has supplied background information, but does not keep economic data on bottling and distribution. Only five bottled water companies, two in detail, responded to the questionnaire sent in April 1977.

The information gathered on bottling equipment, manpower, and truck distribution was used to develop cost functions for a planning analysis.

At the present time, there is no public bottled water distribution system. Thus, the results of the analysis are order-of-magnitude costs and not definitive, because many assumptions were necessary. A public bottled water truck distribution system could vary substantially from place to place, depending on the local conditions and exact design basis.

Table 1 shows the basic assumptions made in the analysis of bottled water distribution by trucks to households. The costs of water aquisition from the source and water treatment are not included.

BOTTLED WATER SUPPLY MODEL

A computer model was developed to facilitate detailed evaluation of the economics of bottled water distribution. This model determines annual labor, bottling, and truck operating and maintenance costs, and total present-worth bottling, delivery, and truck capital costs.

The basic parameters used in the analysis are listed in Table 2, and the flowchart is presented as Table 3. Table 4 is a summary of the bottled water demand and cost functions.

Table 1

Bottled Water Distribution Assumptions

- Deliver 0.5 gpcd x $\frac{4.3 \text{ people}}{\text{residence}}$ x $\frac{7 \text{ days}}{\text{week}}$ = 15 gallons/residence/week
- One 5-gallon bottle of water delivered 3 times per week by truck
- Truck capacity = $\frac{1{,}500 \text{ gallons}}{\text{(Three hundred 5-gallon containers)}}$
- Each truck has 2 workers delivering its capacity daily.
  - Delivery: $\frac{300 \text{ residences}}{\text{day}} \times \frac{1 \text{ min}}{\text{residence}} \times \frac{1 \text{ hr}}{60 \text{ min}}$ = 5 hrs
  - Full bottle loading and empty bottle unloading at plant = 2 hrs
  - Truck travel time from plant to delivery route (38 mile average) = 1 hr

  Total = 8 hrs
- Labor rate including fringe benefits = \$7.50/hr or \$60/8-hr work day
- Truck capital cost = \$18,500
- Truck depreciation = \$ 2,300/year
- Uniform Series Capital Recovery Factor:

$$\frac{A}{P} = \frac{2{,}300}{18{,}500} = \frac{i\,(1+i)^{LIFE}}{(1+i)^{LIFE} - 1}$$

For i = 6% or 7%

Truck Life = 11 years

- Cost of truck operation and maintenance = \$.30/mile
- Cost of bottle = \$2.86 for 5-gallon bottle having an average life of 26 recycle trips or $\frac{\$2.86}{26 \times 5}$ = \$0.022/gallon
- Total unit cost to wash, fill, and cap the bottle, including capital and O&M costs:

$$\frac{\$0.132}{5 \text{ gal.}} = \$0.0264/\text{gallon}$$

Table 2

Bottled Water Supply Model Parameters

Symbol	Parameter	Unit
* NPLAN	Planning period	years
* t	Time	years
* PIRB	Potable, interior residential demand for bottled water	gpcd**
* INT	Annual interest rate	% ÷ 100
* INFL	Annual inflation rate	% ÷ 100
* LIFE	Truck life	years
* SIZE	Truck capacity for bottled water	gallons
* CPCOST	Present truck capital cost	$/truck
* NDELIV	Number of truck deliveries per week per residence	(integer)
* TRK	Truck routing coefficient	(real number)
* DIST	Average roundtrip distance to bottled water plant from truck route	miles
* OMRATE	Rate of truck operation and maintenance	$/mile
* OMCOST	Annual truck operation and maintenance cost	$/truck
* SALARY	Labor rate	$/day
* LABOR	Average number of workers per truck	(integer)
* BCOST	Bottled container cost	$/gallon
* WASH	Total unit cost to wash, fill, and cap the bottle	$/gallon
* POP_t	Population in year t	(integer)
* $DISTRB_t$	Length of water-main distribution in year t	miles
ATRUCK	Total annual truck O&M cost	$/year
ALABOR	Total annual labor costs	$/year
ABOTTL	Total annual bottling cost	$/year
ATOTAL	Total present-worth bottling and delivery costs	$/year
CTRUCK	Total present-worth truck capital cost	$
COST	Total bottled water delivery cost	$/1,000 gallons
CDELIV	Total present-worth bottled water delivery cost for the total planning period	$

* Computer model input
** gallons per capita per day

TABLE 3
SIMPLIFIED BOTTLED WATER SUPPLY MODEL FLOW CHART

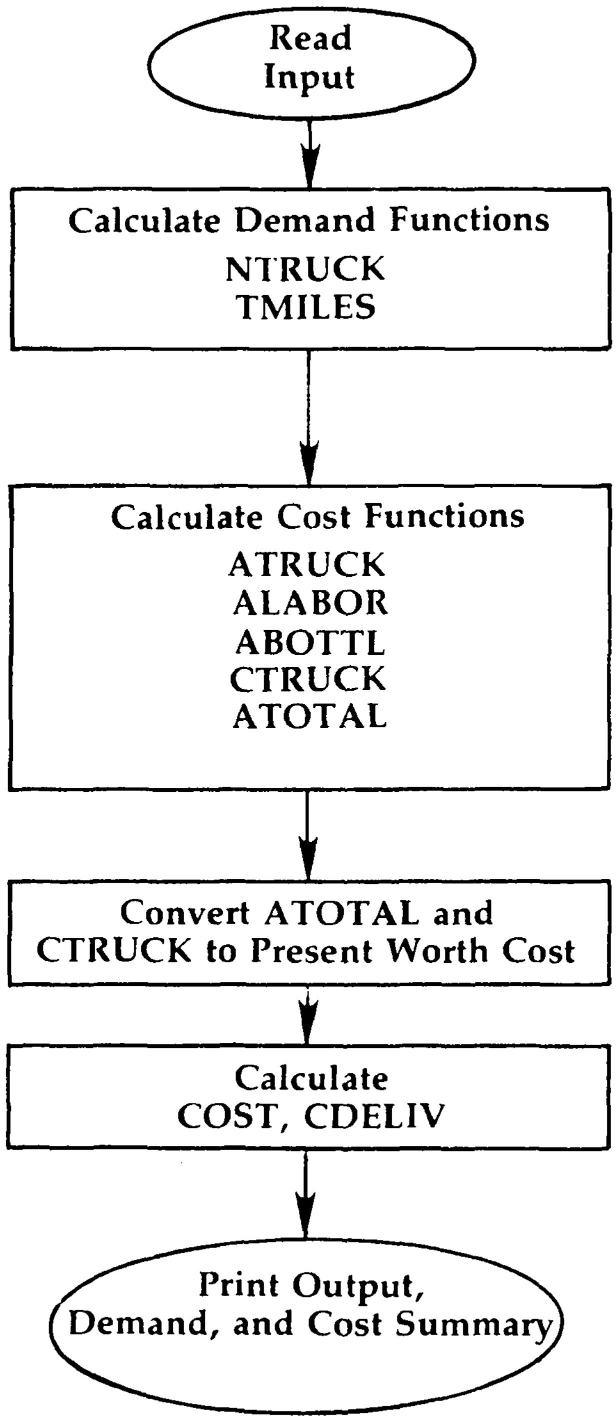


Table 4

Bottled Water Demand and Cost Functions

$$NTRUCK_t = \frac{POP_t \, (PIRB)}{SIZE}$$

$$TMILES_t = DIST + \frac{TRK \, (DSTRB_t)}{NTRUCK_t} \left[\frac{NDELIV}{7} \right]$$

$$ATRUCK_t = NTRUCK_t \, (TMILES_t)(OMRATE)(365)$$

$$ALABOR_t = NTRUCK_t \, (LABOR)(SALARY)(365)$$

$$ABOTTL_t = POP_t (PIRB)(365)(BCOST + WASH)$$

$$ATOTAL = \sum_{t=1}^{NPLAN} (ATRUCK_t + ALABOR_t + ABOTTL_t) \left[\frac{1 + INFL}{1 + INT} \right]^t$$

$$* \; CTRUCK = NTRUCK_1 \, (CPCOST) + \sum_{t=1}^{NPLAN} \left(NTRUCK_{t+1} - NTRUCK_t + \frac{NTRUCK_t}{LIFE} \right) (CPCOST) \left[\frac{1 + INFL}{1 + INT} \right]^t$$

$$CDELIV = ATOTAL + CTRUCK$$

$$COST = \frac{CDELIV \, (1000)}{\sum_{t=1}^{NPLAN} (POP_t)(PIRB)(365)}$$

*Approximation of expression used in computer program.

Based on a given population to be served, the computer model calculates the number of trucks of a given capacity required to deliver the average daily water demand. The length of the truck distribution route, round-trip distance to the route from the bottling plant, and the number of deliveries per week are used to calculate the average daily truck travel.

This provides the capital cost and associated operating and maintenance and labor costs for each truck to deliver its capacity, e.g. 1,500 gallons per day. The other components of cost are empty bottles and bottle washing, filling, and capping in preparation for delivery. Adding these costs and converting to present worth provides a basis for comparison between systems. The cost per gallon is the total present-worth delivery cost divided by the total volume of water delivered over the planning period.

ECONOMICS OF BOTTLED WATER DISTRIBUTION

The bottled water supply model was run with various input values for a 25-year planning period, to determine the total present worth of truck delivery. This can then be compared with pipe distribution of the potable fraction of water, from the dual water supply model.

Three test-case community sizes (20,000, 100,000, and 500,000) were initially examined for residential bottled water demands of 0.5, 1.0, and 3.0 gallons per capita per day (gpcd). (The test cases used the default values listed in Table 5, which were derived from the basic assumptions.) The linear variation of total present-worth costs is illustrated in Figure 1.

The population was assumed to increase 1 percent each year for the three cities during the 25-year planning period: i.e., 20,000 to 25,400, 100,000 to 127,000, and 500,000 to 635,000 respectively. Also, the length of water-main distribution was assumed to increase from 100 miles to 127 miles, 500 to 635, and 2,000 to 2,540 respectively.

The model was used to perform a sensitivity analysis, for varying each parameter individually within the ranges indicated in Table 5. The results indicated that truck capacity has a significant influence on total cost, because it is the basis for calculating the number of trucks needed and the labor force. Increasing the size will decrease the cost in this analysis if the basic assumption (that the two workers for each truck can deliver its capacity daily) is met.

More than half the total cost is for labor; the number of workers and their salary have a direct effect. Next in importance are bottle (container) cost and the washing, filling, and capping operation. The total present worth was relatively insensitive to the other six input parameters for the ranges tested.

Table 5

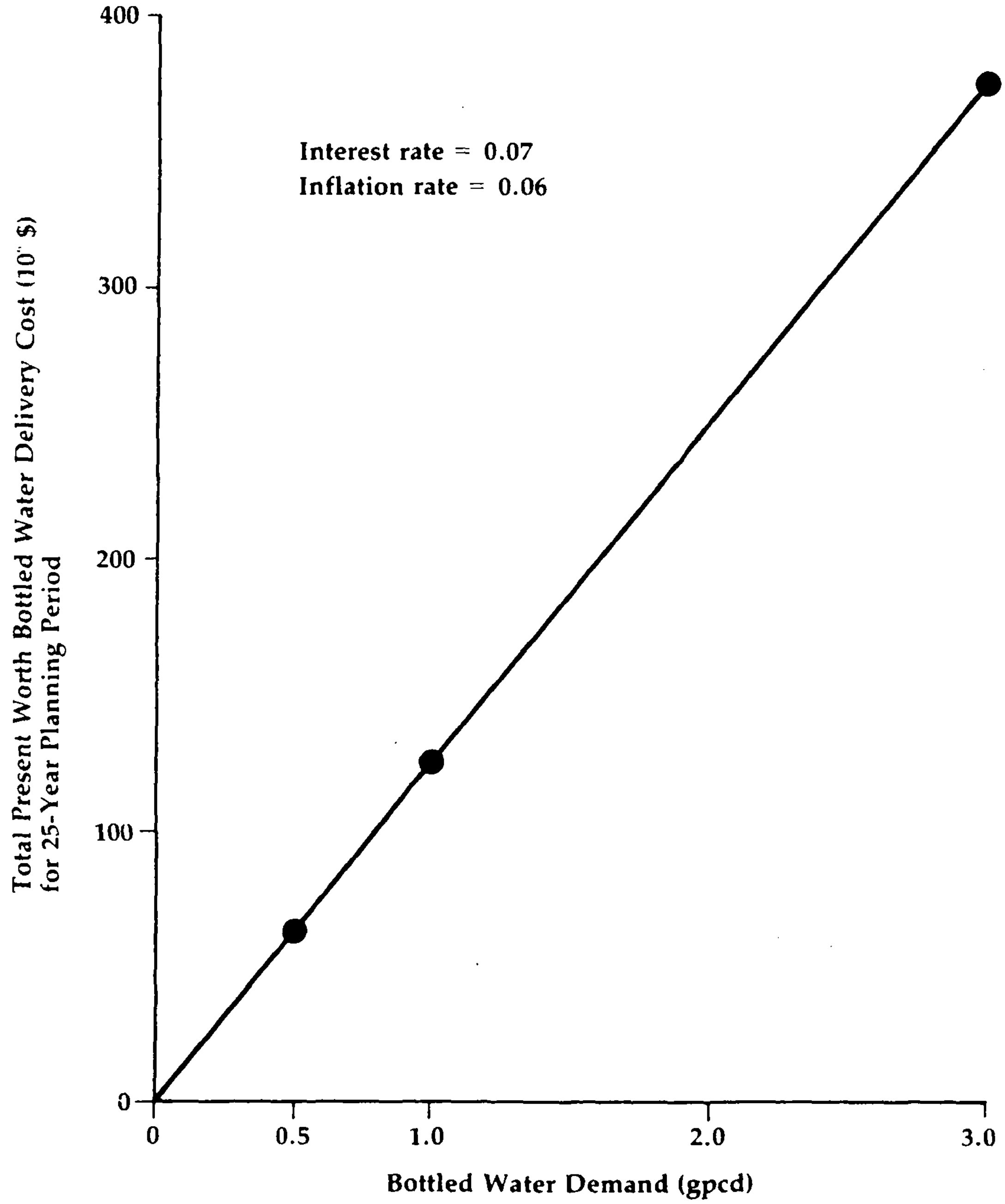
Bottled Water Supply Model

Sensitivity Analysis

Parameter	Default Value	Range Tested	Total Present Worth Variation (%)
LIFE	11	4-12	- 3
SIZE	1,500	1,000-3,000	-63
CPCOST	18,500	15,000-25,000	+ 2
DELIV	3	1-5	+ 2
TRK	1.5	1-2	+ 1
DIST	25	10-50	+ 7
OMRATE	0.30	0.20-0.40	+ 3
SALARY	60	40-80	+38
LABOR	2	1-3	+57
BCOST	0.022	0.01-0.03	+14
WASH	0.026	0.015-0.04	+18

FIGURE 1 BOTTLED WATER SUPPLY MODEL: 100,000 POPULATION

Test Case Using Default Values



The model showed no economies of scale. The total unit bottled water delivery cost for an interest rate of 7 percent and inflation rate of 6 percent was a constant \$123/1,000 gallons. The reason is that the costs are directly proportional to the number of trucks and to the labor needed to deliver water from each of these trucks. Also, the unit costs for the bottles, washing, filling, and capping were kept constant for the various sizes of bottled water plants.

The total present-worth cost of \$123/1,000 gallons (\$0.123/gallon) for delivery is approximately 25 percent of the total price of home-delivered bottled water from private producers today. The total present-worth delivery cost varies from \$63 to \$180/1,000 gallons as the overall inflation rate increases from 0.93 to 1.03 (Figure 2).

Table 6 is a summary of the comparison of the total present-worth cost of truck distribution to pipe distribution for the three selected community sizes. At 0.5 gpcd, truck distribution of bottled water costs about 20 times piped potable water distribution; at 3.0 gpcd, the cost increases to approximately 55 times, independent of city size.

Even though the cost of public home delivery of bottled water greatly exceeds that of water distribution through mains, there may be instances where bottled water is a feasible alternative in a dual distribution system. The developed computer model can give a preliminary economic analysis of a public truck distribution system for the bottled water. In using the flexible model, all input parameters should be properly selected for the particular locality and situation.

FIGURE 2 BOTTLED WATER SUPPLY MODEL: 100,000 POPULATION

Test Case Using Default Values

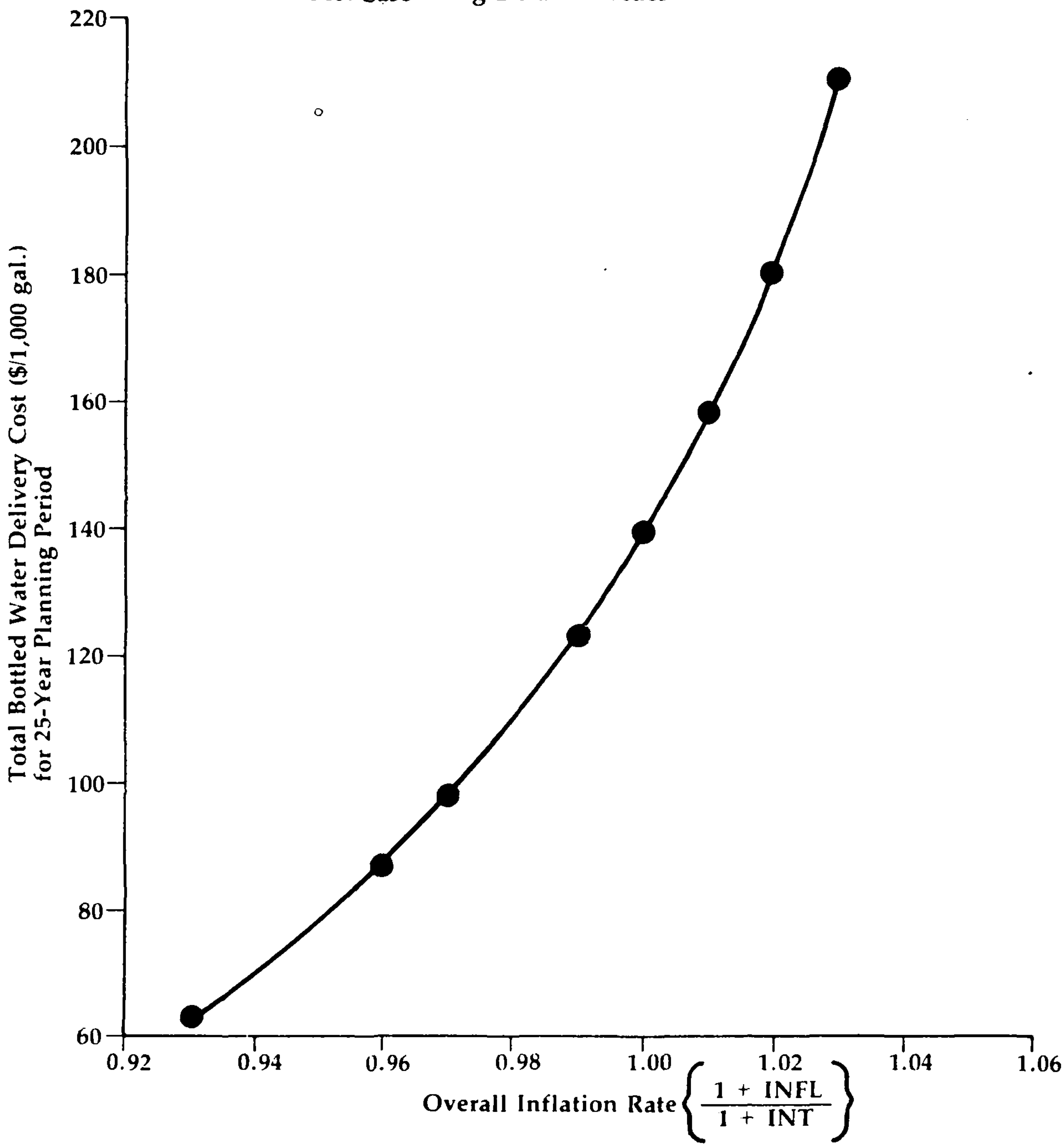


Table 6

Bottled Water Truck Distribution vs. Distribution Main

Population	gpcd	Distribution Length in 1976 and 2000 (miles)	Total Present Worth Cost[1] for 25-Year Planning Period (10^6 $)	
			Distribution Main[2]	Bottled Water Truck Distribution
100,000	0.5	500-635	2.88	63.5
	1.0		4.17	126
	3.0		6.93	376
20,000	0.5	100-127	.503	12.0
	1.0		.729	23.7
	3.0		1.21	70.8
1,000	0.5	10-13	.039	.643
	1.0		.057	1.27
	3.0		.094	3.77

[1] Interest Rate = 0.07
Inflation Rate = 0.06

[2] Cost from Dual Water Supply Model Subroutine

SECTION 7

APPLICATION METHODOLOGY AND RESULTS

APPLICATION METHODOLOGY AND RESULTS

APPLICATION

With the growing needs of urban water demand and decreasing availability of good quality sources, it is apparent that efficient and economical management of water is an essential step in long-range water planning. In water problem areas, in order to develop an optimum water plan and to safeguard the public health, it might be necessary to consider more than one raw water source and more than one supply system.

This concept would produce a large number of alternative plans for supplying water for a city or a region. Many of these possibilities can be readily dismissed on the basis of engineering judgment and local conditions. However, a substantial number of alternatives will require full analysis and evaluation before selection of a final plan.

The systems model that has been developed in this study is a powerful tool for analyzing the various alternatives of long term urban water management. The model is also very flexible and can accommodate any configuration of water supply system. It can handle: 1) a conventional system; or 2) a dual or multiple system with more than one source of water (including reuse) and supplying more than one grade of water to various demand centers.

The model has been designed also to accommodate regional management of water, with up to 10 cities in the region. In a regional water supply management analysis one limited good quality source may be considered to supply all potable demands of the region, and subpotable and nonpotable demands can be drawn from local unprotected sources or from effluents of sewage treatment plants. Again, the model is applicable to a new system as well as to an existing system.

Bottled water may be a viable alternative for potable supply in a multiple system. Hence, a bottled water distribution system model has also been developed.

Possible Application Areas

This systems methodology can be used for any urban water supply management and planning effort where a large number of alternatives is to be analyzed. Specifically, this systems model can be applied to evaluate various alternative plans using conventional and multiple supply concepts in areas where: 1) the quality of the raw water source is poor; 2) the availability of good quality water is limited; and 3) where there is an overall shortage of water.

New Systems

In the design of new water supply systems, it is advisable to consider the multiple supply concept as a viable alternative for a long-term safe water supply plan. From social, engineering, economical and institutional points of view, it would be easier to establish multiple supply in a new city.

Depending on the sources and quality of raw water, 14 dual water supply systems and corresponding single (conventional) supply systems using various treatment systems have been identified as typical cases. Table 1 lists these 14 hypothetical dual supply systems along with the sources of raw water and treatment systems required to produce water of desired qualities. It should be emphasized, however, that there may be many other cases where dual or multiple supply can be economically adopted.

Existing Systems

With the introduction of Primary Drinking Water Regulations, many water systems will have to provide further treatment or look for alternative good quality sources in order to meet requirements. In cases where the cost of additional treatment for the bulk water is high, it might be appropriate to consider a dual supply system, using either the same source or an alternate source for potable supply. Figures 1 to 4 describe typical existing systems and outline conventional or multiple supply alternatives.

In cases where a good quality, limited water supply is available but distant (Figures 2 and 3), it might be preferable to conserve good quality water and use recycled water or local lower quality sources for subpotable and/or nonpotable uses. In cases (Figures 1 and 4) where existing source quality is poor it might be less expensive to treat a small fraction of the water to meet drinking water regulations and distribute it through a separate distribution system.

In an existing system where large quantities of water of nonpotable quality are required by industries, a dual water supply system can be used in long-term planning with and without reuse as shown in Figure 5.

In regional management of water, good quality limited sources can be conserved and used for potable supply only through a regional water treatment and distribution system. The water for subpotable uses can be obtained from local polluted sources (Figure 6). In a region where a limited quantity of protected water is available, existing communities obtaining their water supply from nearby polluted sources will have a choice. They will have the option of having additional treatment for removing trace chemicals, or using a limited protected source for potable supply to all communities in the region using a dual distribution system.

Table 1

Various Dual Water Supply Systems.

System Series	System Number	Supply System	Quality of Supply	Source of Raw water	Treatment* System to be Used
I. Potable Source of Dual Supply: Protected Groundwater	I-1	Dual	Potable	Protected Ground Water	TS1
			Subpotable	Unprotected Surface Water	TS7
		Single	Potable	Unprotected Surface Water	TS5
	I-2	Dual	Potable	Protected Ground Water	TS1
			Subpotable	Wastewater Effluent	TS9
		Single	Potable	Unprotected Surface Water	TS5
	I-3	Dual	Potable	Protected Ground Water	TS1
			Subpotable	Unprotected Surface Water	TS7
		Single	Potable	Unprotected Surface Water	TS4
	I-4	Dual	Potable	Protected Ground Water	TS1
			Subpotable	Wastewater Effluent	TS9
		Single	Potable	Unprotected Surface	TS4
II. Potable Source of Dual Supply: Unprotected Groundwater	II-1	Dual	Potable	Unprotected Ground Water	TS2
			Subpotable	Unprotected Ground Water	TS6
		Single	Potable	Unprotected Ground Water	TS2

*See Section 3 for corresponding treatment systems.

Table 1

(continued)

System Series	System Number	Supply System	Quality of Supply	Source of Raw water	Treatment* System to be Used
	II-2	Dual	Potable	Unprotected Ground Water	TS2
			Subpotable	Wastewater Effluent	TS9
			Potable	Unprotected Ground Water	TS2
III. Potable Source of Dual Supply: Protected Surface Water	III-1	Dual	Potable	Protected Surface Water	TS3
			Subpotable	Unprotected Surface Water	TS7
		Single	Potable	Unprotected Surface	TS5
	III-2	Dual	Potable	Protected Surface Water	TS3
			Subpotable	Wastewater Effluent	TS9
		Single	Potable	Unprotected Surface Water	TS5
	III-3	Dual	Potable	Protected Surface Water	TS3
			Subpotable	Unprotected Surface Water	TS7
		Single	Potable	Unprotected Surface Water	TS4
	III-4	Dual	Potable	Protected Surface Water	TS3
			Subpotable	Wastewater Effluent	TS9
		Single	Potable	Unprotected Surface Water	TS4
	III-5	Dual	Potable	Protected Surface Water	TS3
			Subpotable	Unprotected Ground Water	TS6
		Single	Potable	Unprotected Ground Water	TS2

*See Section 3 for corresponding treatment systems.

Table 1

(continued)

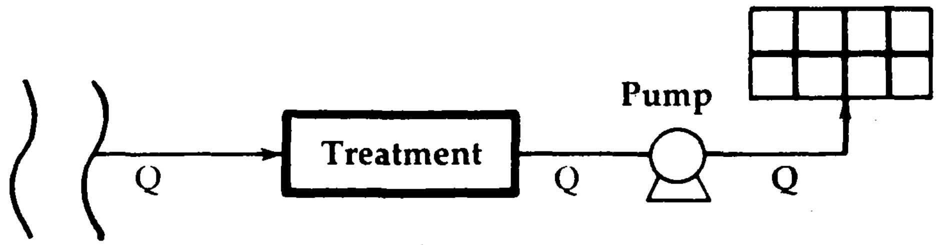
System Series	System Number	Supply System	Quality of Supply	Source of Raw water	Treatment* System to be Used
	III-6	Dual	Potable	Protected Surface Water	TS3
			Subpotable	Wastewater Effluent	TS9
			Potable	Unprotected Ground Water	TS2
Potable Source of Dual Supply: Unprotected Surface Source	IV-1	Dual	Potable	Unprotected Surface Water	TS5
			Subpotable	Unprotected Surface Water	TS7
		Single	Potable	Unprotected Surface Water	TS5
	IV-2	Dual	Potable	Unprotected Surface Water	TS4
			Subpotable	Unprotected Surface Water	TS7
		Single	Potable	Unprotected Surface Water	TS4

*See Section 3 for corresponding treatment systems.

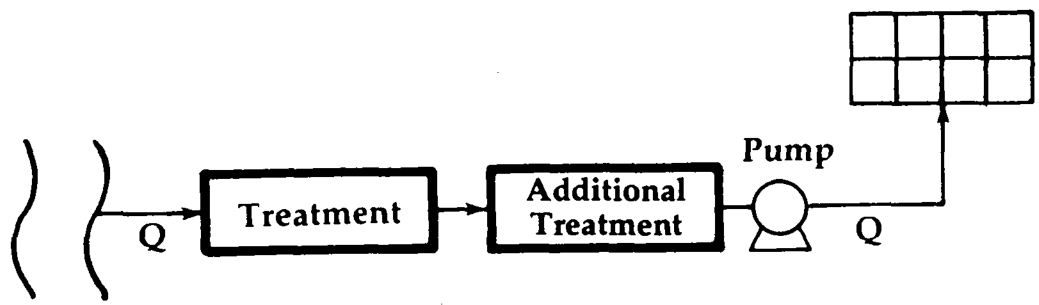
FIGURE 1

EXISTING SYSTEM 1

Source: Polluted River

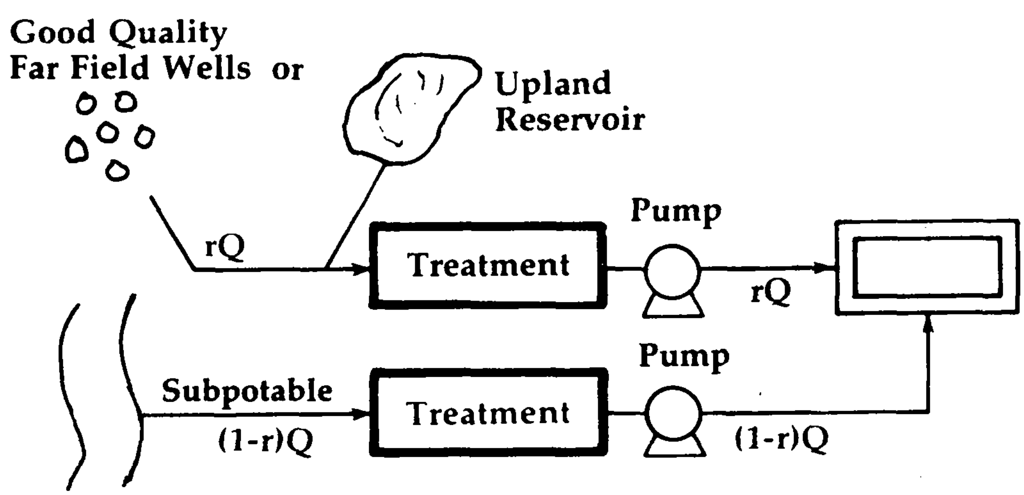


CONVENTIONAL ALTERNATIVE



DUAL SUPPLY ALTERNATIVES

1.



2.

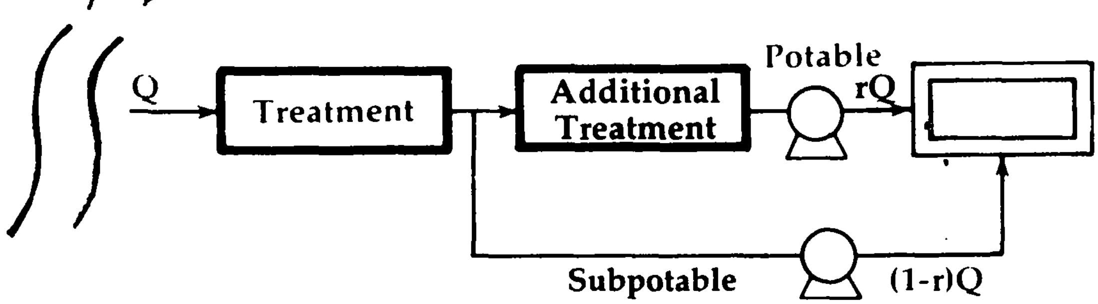


FIGURE 2

EXISTING SYSTEM 2

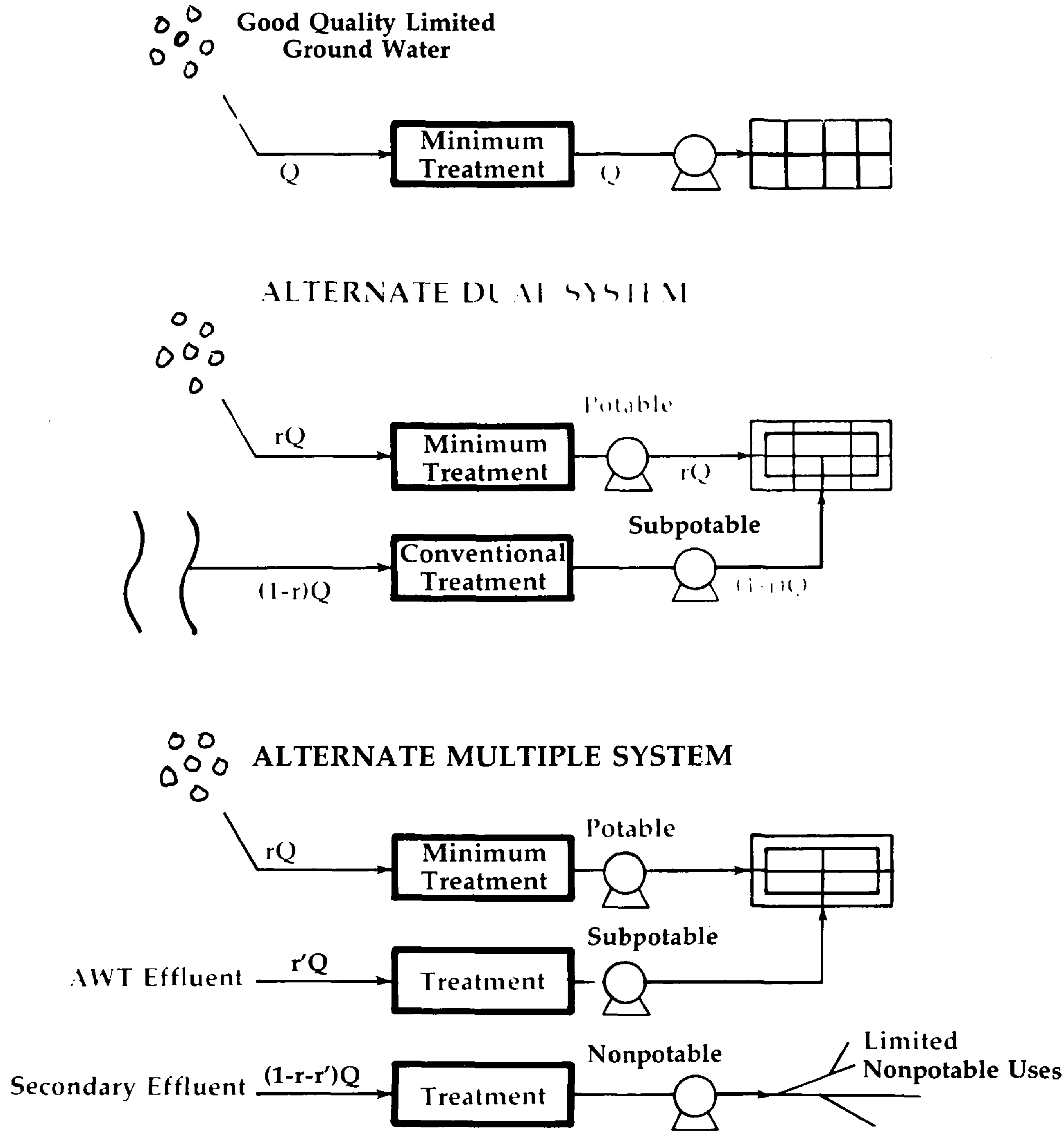


FIGURE 3

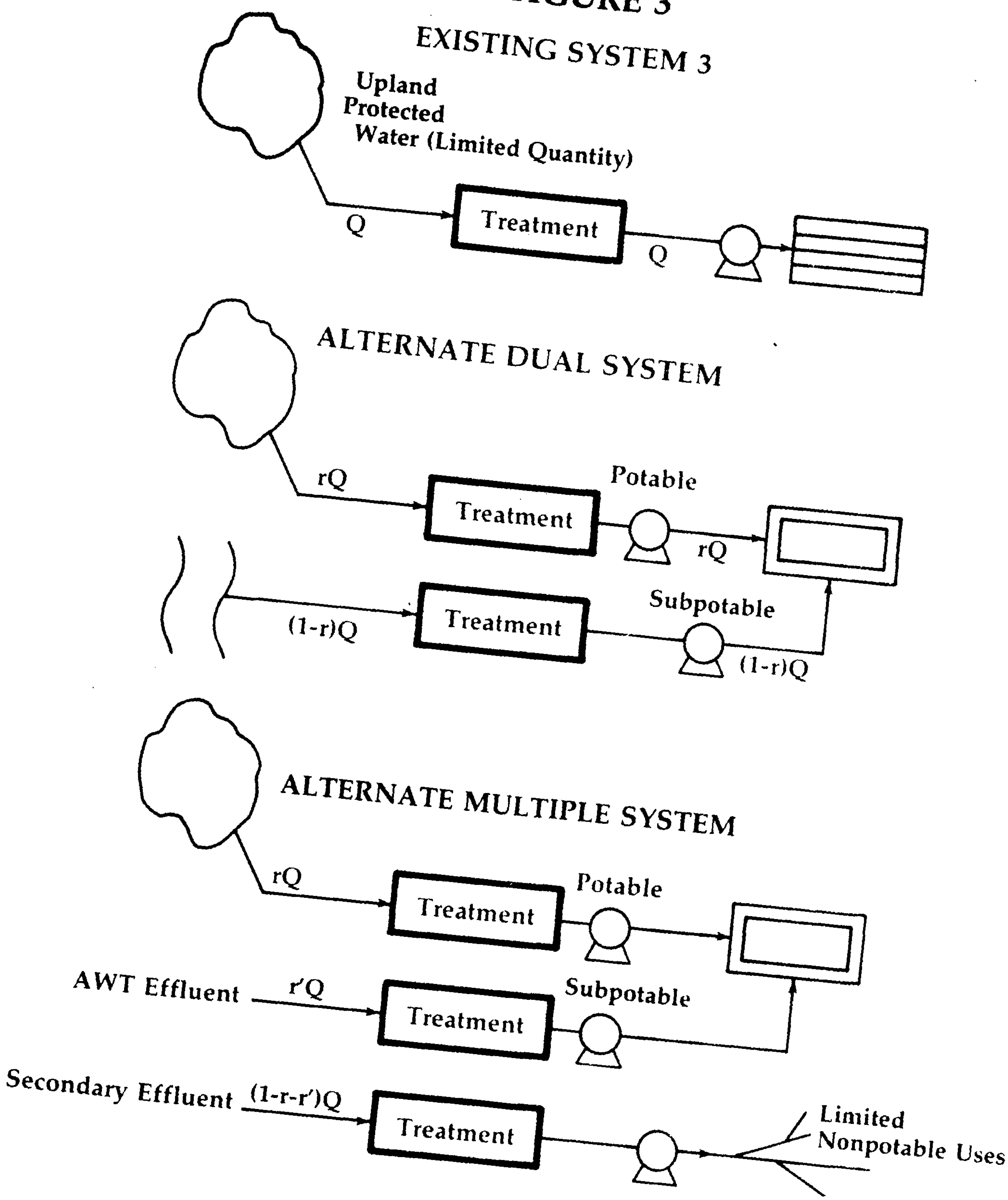
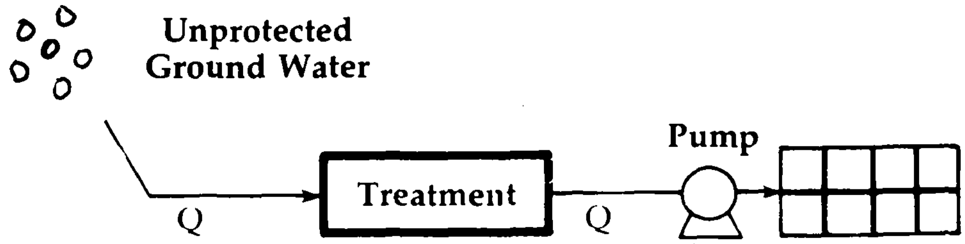
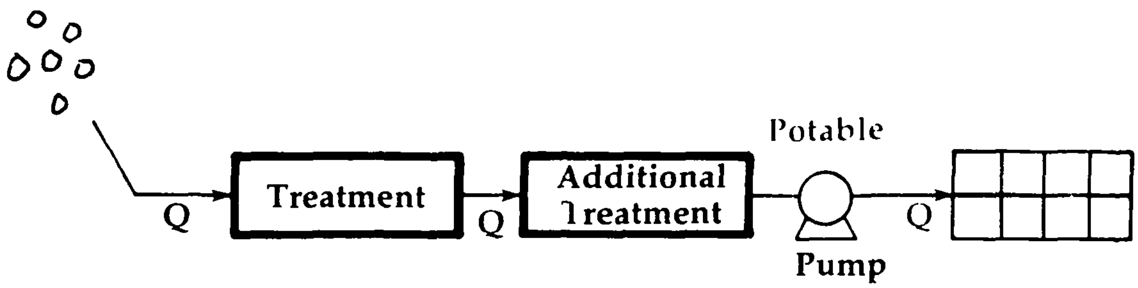

EXISTING SYSTEM 3
Upland
Protected
Water (Limited Quantity)
Q
Treatment
Q
ALTERNATE DUAL SYSTEM
rQ
Treatment
Potable
rQ
(1-r)Q
Treatment
Subpotable
(1-r)Q
ALTERNATE MULTIPLE SYSTEM
rQ
Treatment
Potable
AWT Effluent
r'Q
Treatment
Subpotable
Secondary Effluent
(1-r-r')Q
Treatment
Limited
Nonpotable Uses

FIGURE 4

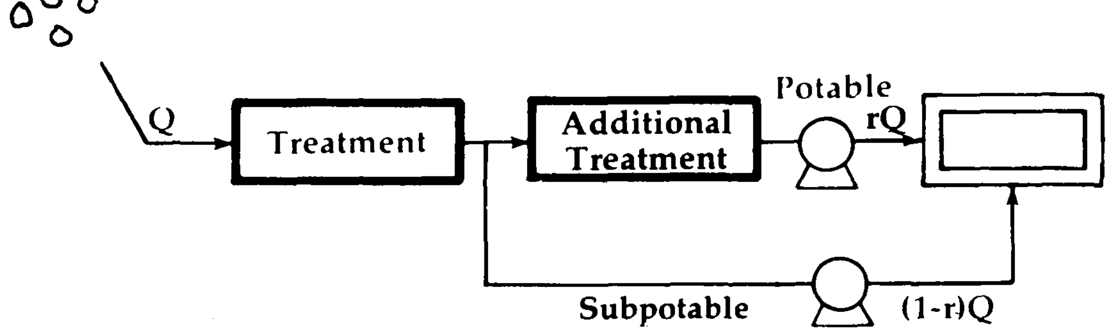
EXISTING SYSTEM 4



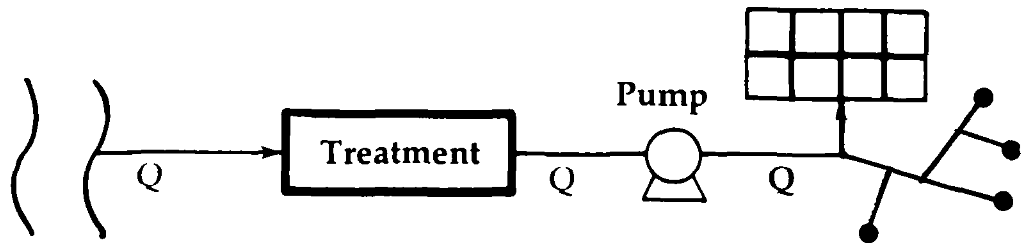
CONVENTIONAL ALTERNATIVE



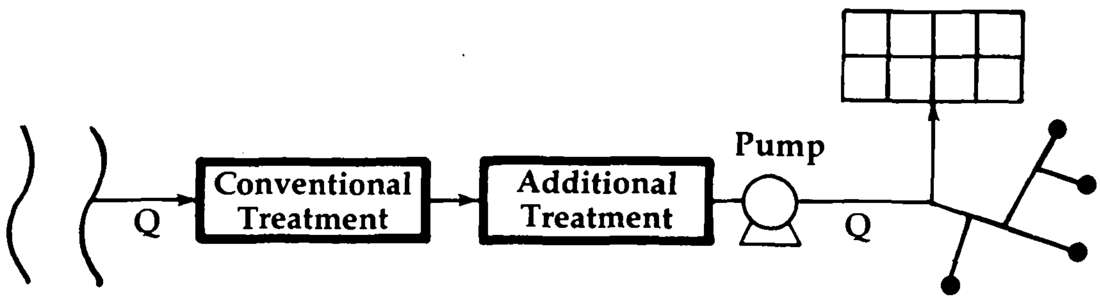
DUAL SUPPLY ALTERNATIVE



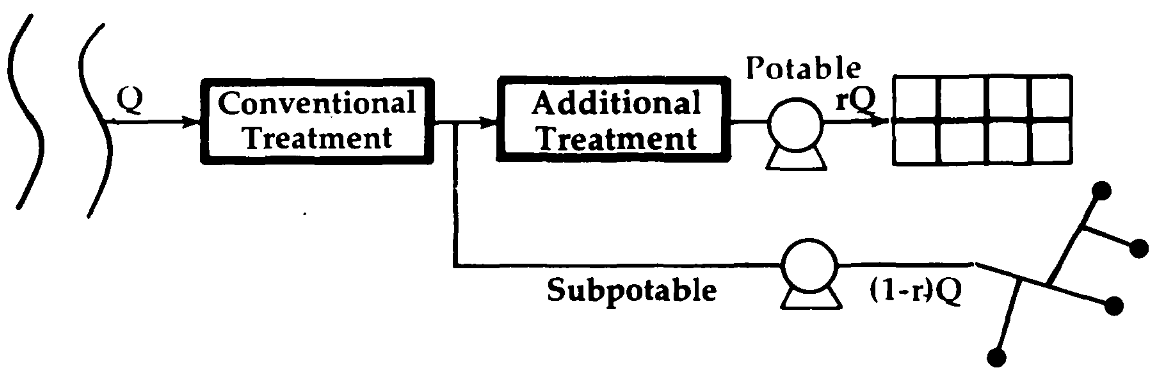
Source: Polluted River



SINGLE SUPPLY ALTERNATIVE



DUAL SUPPLY ALTERNATIVE



DUAL SUPPLY ALTERNATIVE WITH REUSE

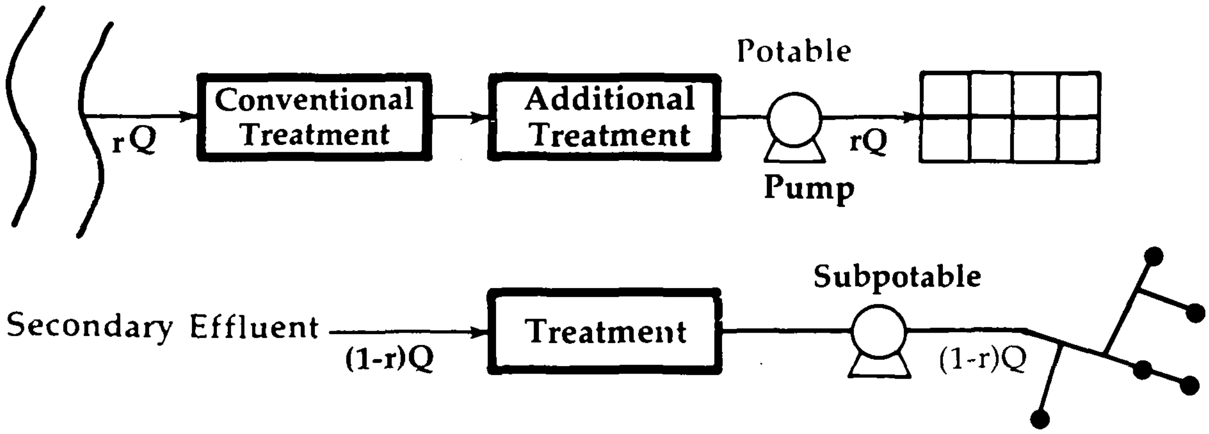
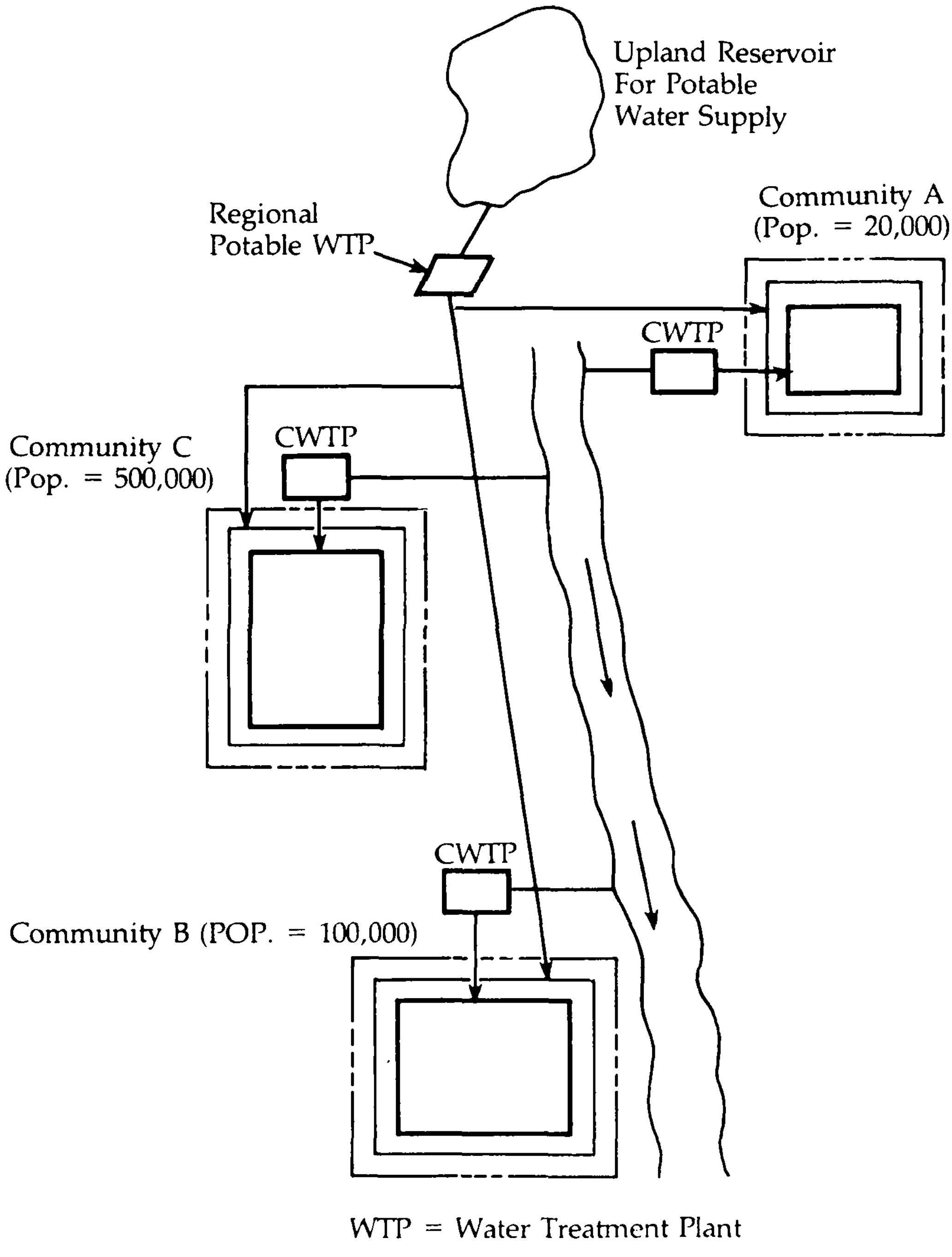


FIGURE 6 REGIONAL WATER SUPPLY MANAGEMENT USING DUAL SUPPLY CONCEPT



WTP = Water Treatment Plant

CWTP = Conventional Water Treatment Plant

Application Procedures and Data Needs

In order to use the system model in city or regional water supply planning, the following steps are required:

- Understanding of the computer system.
- Identification of available water sources, including recycled water.
- Identification of quantity and quality of raw water from each source.
- Identification of quantity and quality of various demands (potable, subpotable and nonpotable).
- Identification of raw water treatment requirements to meet various demands.
- Identification of transmission, treatment and distribution from each source and demand.
- Identification of various alternative water systems, including multiple systems.
- Collection of data for input into the model.

Data Requirements

Data requirements for the systems model can be divided into the following categories:

- Physical and design data.
- Demographic and water demand data.
- Economic data (interest rate, inflation, etc.), cost data.

In order to simplify the model for use by an engineer or planner with little computer background, data requirements have been minimized by providing default values for many data. The details of the data requirements will be discussed in the next section.

However, as a note of caution, it is always advisable to check the validity of model and data under the circumstances of application. The coefficients and exponents of cost functions of various units used should be valid for the locality of use.

Application Problems

Whenever the concept of dual water supply is raised, there is always a concern regarding the possibility of cross connection. From past experience, it has been observed that a dual system with safe water for drinking and unsafe water for other household uses poses a serious health threat, either from drinking unsafe water by mistake or from cross connection of the systems.

Such a dual supply system should not be considered seriously. However, the dual water supply considered in this study would offer two safe supplies at the home. The potable water would conform with the drinking water regulations; the subpotable water would also be safe, but of inferior trace-chemical quality. In case of a cross connection no one would be harmed by drinking the cross-connected water.

If proper precautions are taken, it is expected that the cross connection could be detected quickly, but as Haney (1) in his recent article on dual water systems mentions, "The importance of cross-connection control in all types of systems, however, cannot be downgraded. Cross connections are insidious. They must be ferreted out and eliminated."

In a residence served by a dual system, plumbing would be installed or modified to conform to the dual system. Each house or building with a dual system would require two service lines and two meters.

In an existing system of supply, it would be difficult to convert the whole single supply system to a dual system at one time. In the initial stages, however, dual supply could be offered economically to large consumers, and gradually extended to new housing developments at the outskirts of the city.

RESULTS

Of the 14 multiple water systems described in Table 1, five systems (I-3, II-1, III-1, III-5 and IV-2) are studied, using the computer model for various potable/total flow ratios, capital cost inflation rates, O&M cost inflation rates, and population densities. The program has been run using three population-sized cities (20,000 population, 100,000 population and 500,000 population). The computer output includes input data verification, details of costs and present worth of capital, O&M and salvage values, and total present worth of each system.

Sensitivity of Dual/Conventional, Supply Cost Ratio with Potable/Total Flow Ratio

The ratios of total costs of dual supply to conventional supply for all the five systems, and for various potable to total flow ratios have been presented in Figures 7 to 11.

FIGURE 7
SYSTEM I-3
CITY: 100,000 POPULATION

Conventional System

Source:	Unprotected River
Treatment:	Coag. + Sed. + Filt. + Gac + Disin.

Dual System

Potable Source:	Protected Ground Water
Treatment:	Disinfection
Subpotable Source:	Unprotected River
Treatment:	Coag. + Sed. + Filt. + Disin.

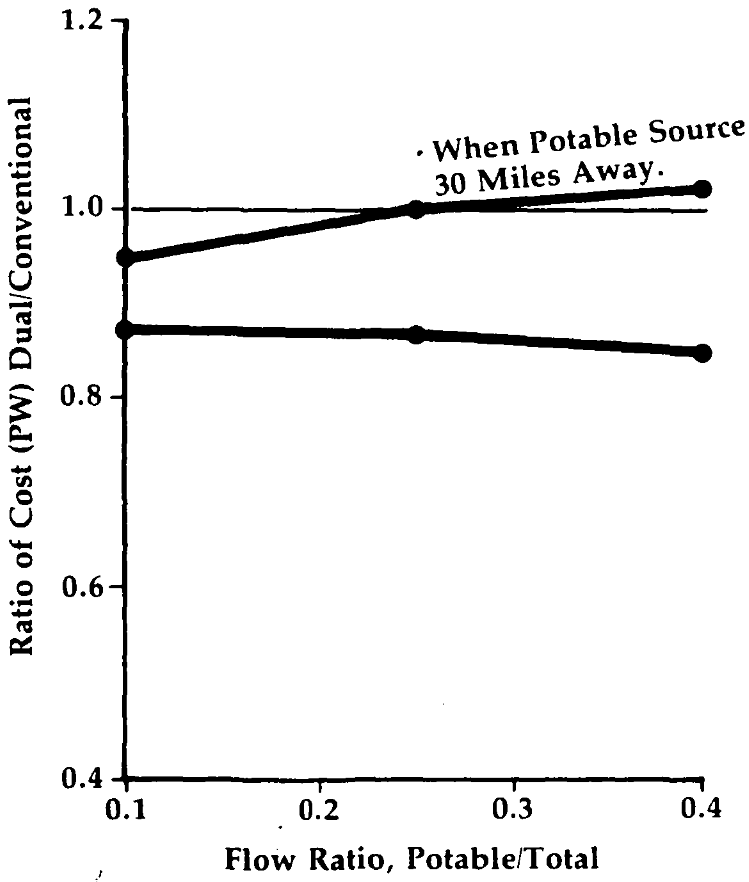


FIGURE 8
SYSTEM II-1
CITY: 100,000 POPULATION

Conventional System

Source:	Unprotected Ground Water
Treatment:	Lime + Filt. + Disinf.

Dual System

Potable:	Unprotected Ground
Treatment:	Lime + Filt. + Disinf.
Subpotable:	Same Source
Treatment:	Aeration + Filt. + Chlorination

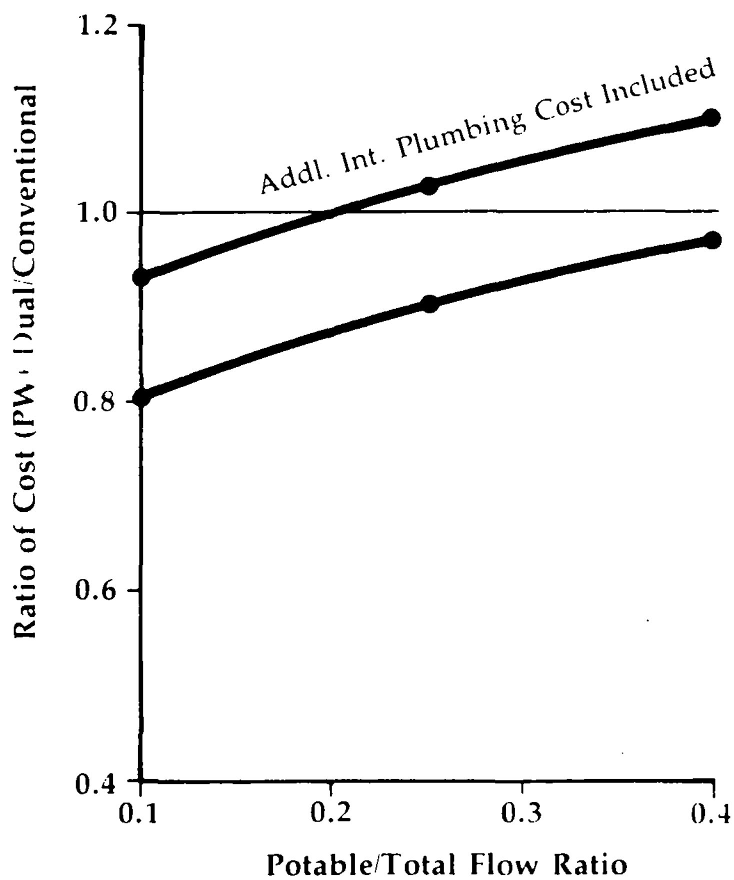


FIGURE 9
SYSTEM III-1
CITY: 100,000 POPULATION

Conventional System

Source: Unprotected River
Treatment: Lime + Filter. + Gac + Disinf.

Dual System

Potable: Protected Upland Reservoir
Treatment: Filtration + Disinfection

Subpotable: Unprotected River
Treatment: Coag. + Sed. + Filt. + Chlorination

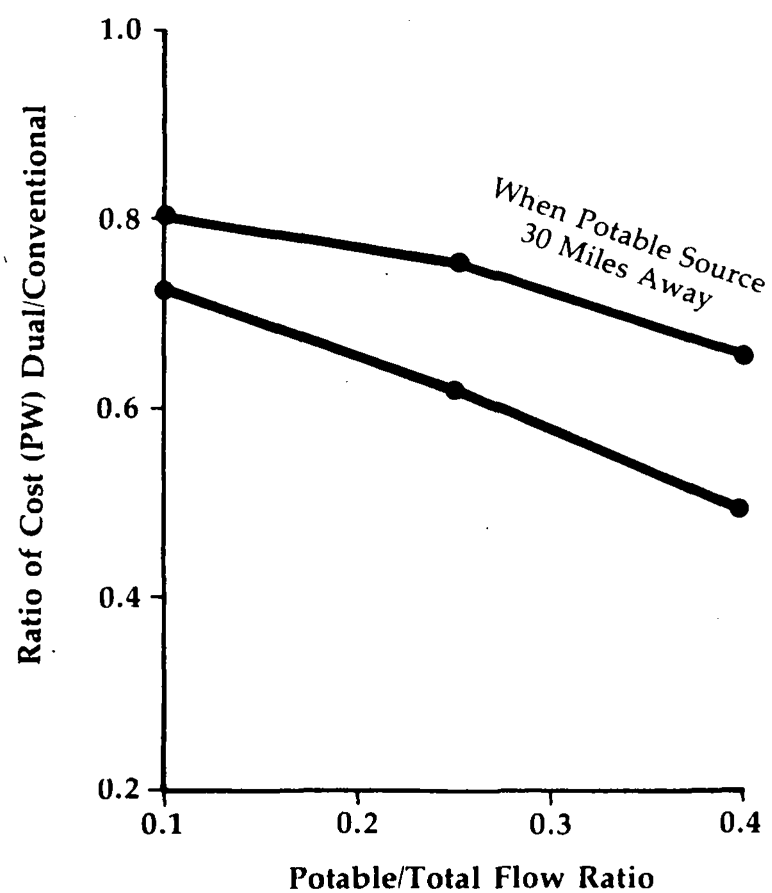


FIGURE 10
SYSTEM III-5
CITY: 100,000 POPULATION

Conventional System

Source:	Unprotected Ground Water
Treatment:	Lime + Filt. + Disinf.

Dual System

Potable:	Protected Upland Reservoir
Treatment:	Filtration + Disinfection
Subpotable:	Unprotected Ground Water
Treatment:	Aeration + Filt. + Chlorination

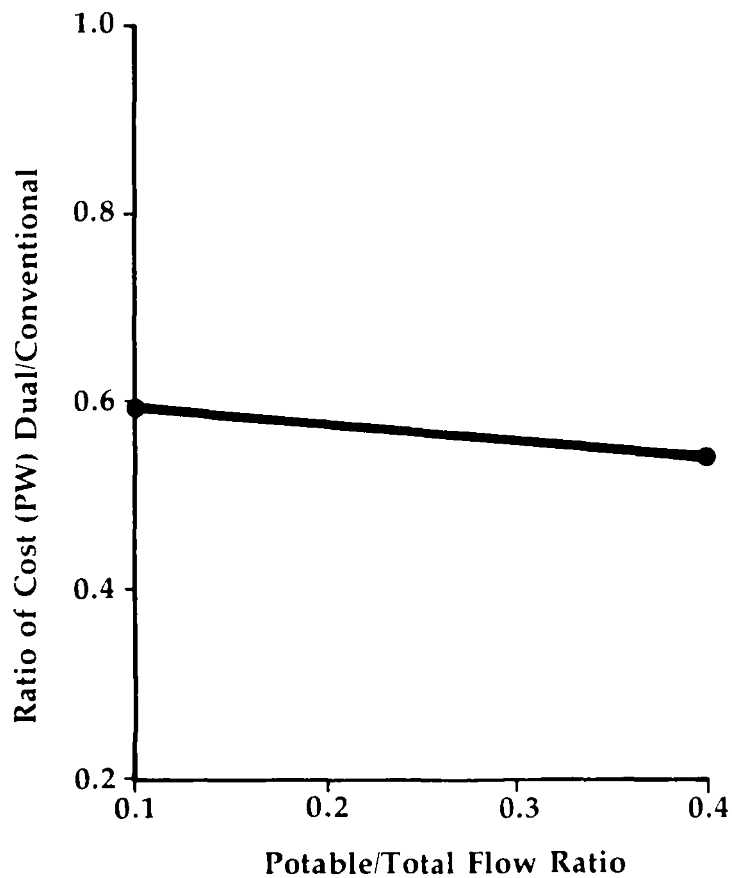


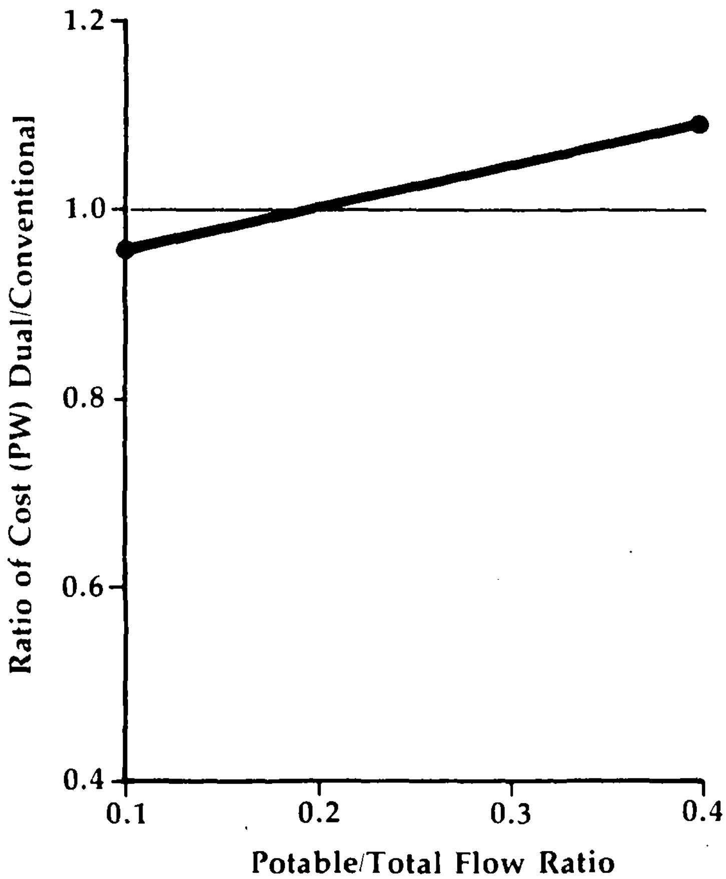
FIGURE 11
SYSTEM IV-2
CITY: 100,000 POPULATION

Conventional System

Source:	Unprotected River
Treatment:	Coag. + Sed. + Filt. + Gac + Disinf.

Dual System

Potable:	Unprotected River
Treatment:	Coag. + Sed. + Filt. + Gac + Disinf.
Subpotable:	Same Source
Treatment:	Coag. + Sed. + Filt. + Chlorination



In System I-3, the source of water is an unprotected river. In order to produce potable water from this source, granular activated carbon (GAC) has been added with the standard treatment of coagulation and filtration. In the corresponding dual system, the potable source is good quality groundwater, and the unprotected river source furnishes subpotable water.

It is apparent from Figure 7 that the dual water supply cost is less than 85 percent of that of a conventional supply in a new system. If, however, it is assumed that the potable well source for dual supply is 30 miles away, dual supply becomes more expensive than conventional supply, when the flow ratio exceeds 0.25.

The results of System II-1 are presented in Figure 8. In this system the conventional and the dual supply have the same source (unprotected groundwater). The potable supply dual system will have the same treatment as the conventional system. The subpotable fraction of the dual system will have standard treatment. The results show that for a new system, dual supply is cheaper than conventional supply. However, if additional interior plumbing changes and metering cost are included in the dual system, a break-even point in cost will be reached at a flow ratio of 0.2.

In Systems III-1 and III-5, it is assumed that local good quality water of limited supply is available for the potable fraction of the dual system. The cost curves show (Figures 9 and 10) that a dual system is always more economical than a conventional system.

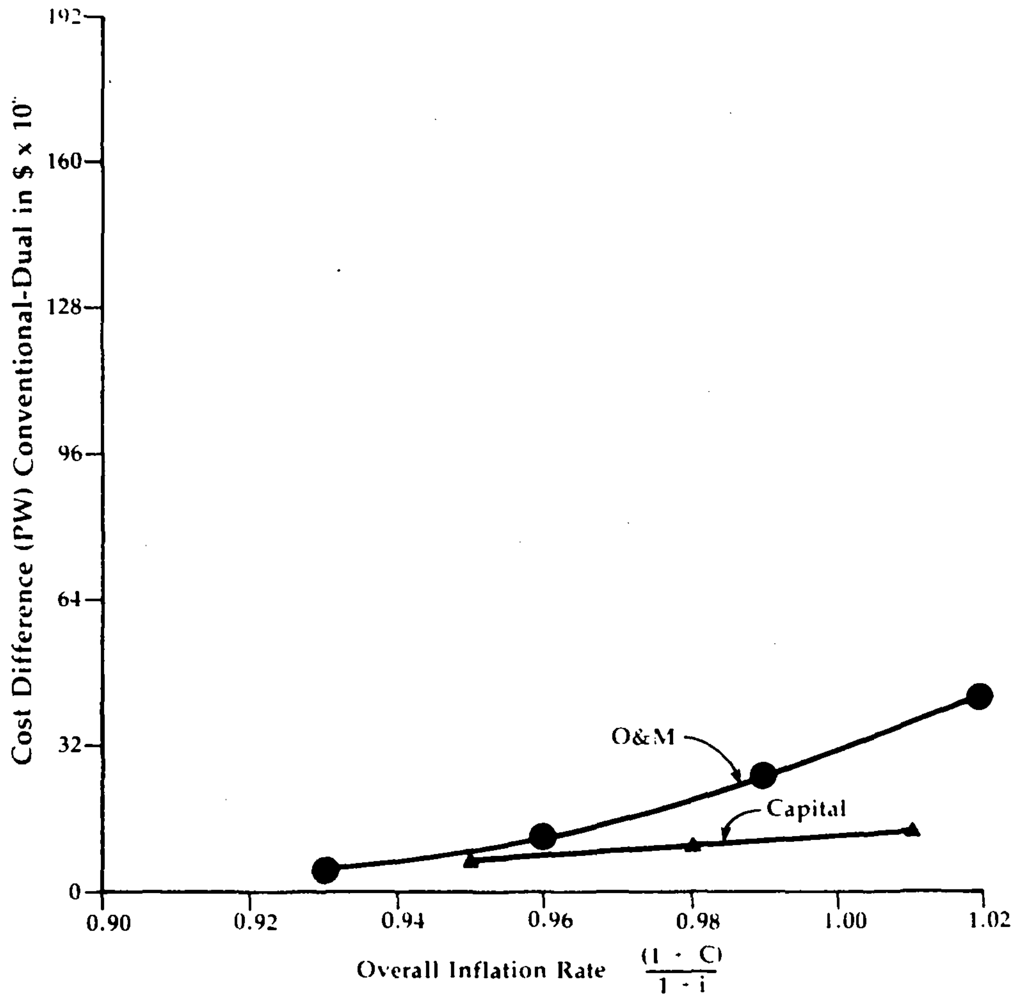
In System IV-2, unprotected river water is the only source for either a conventional or a dual system. An activated carbon treatment has been considered along with the standard treatment to produce potable water in the dual and conventional systems, whereas only standard treatment is given to produce subpotable water in the dual system. The result has been presented in Figure 11. A conventional system will be more economical when the flow ratio exceeds 0.2. However, if plumbing change cost is included in the dual system, a conventional system will be cheaper, even at a flow ratio of 0.1.

In all the systems studied, it has also been found that if in any treatment system reverse osmosis is needed to produce potable water the dual system is always more economical.

<u>Sensitivity of Inflation Rate on Cost</u>

The sensitivity of capital inflation rates $\frac{1 + C_c}{1 + i}$, and O&M inflation rates $\frac{1 + C_o}{1 + i}$, on cost difference (conventional system cost - dual system cost) has been studied. The results are plotted in Figure 12. From Figure 12 it can be seen that the O&M cost inflation rate is more

FIGURE 12 COST RATIO VERSUS OVERALL INFLATION RATE SYSTEM I-3 POPULATION 100,000



sensitive to cost difference than the capital inflation rate. This is due to the fact that O&M costs are distributed over the entire planning period and are more affected by inflation.

Sensitivity of Population Density and Potable Total Flow Ratio on Distribution Main Cost

Distribution main cost is largely dependent on the length of mains required. If the population density of a city is high, the total length of mains required is relatively low and as a result the cost of distribution is less. Figure 13 shows that with an increase of population density, cost of distribution mains for conventional and dual systems decreases and the difference in distribution cost of dual and conventional systems also decreases.

With an increase of potable to total flow ratio in dual distribution systems, the cost of distribution mains increases; the cost reduces to minimum when the flow ratio is zero; i.e., when the system becomes conventional (Figure 13).

Point-of-Use Water Treatment

Point-of-use water treatment refers to a home "under-the-sink" unit for purification of drinking water. Depending on the type of water, several different types of point-of-use equipment are available.

The granular activated carbon (GAC) home units filter and adsorb trace chemicals and organics and remove the taste of chlorine. This can be accomplished in a replaceable packed bed or a loose bed which requires regeneration. Silver ions can be deposited on the activated carbon to inhibit bacteria growth in the units, or a mechanical filter can be added after carbon adsorption to remove any bacteria from the water.

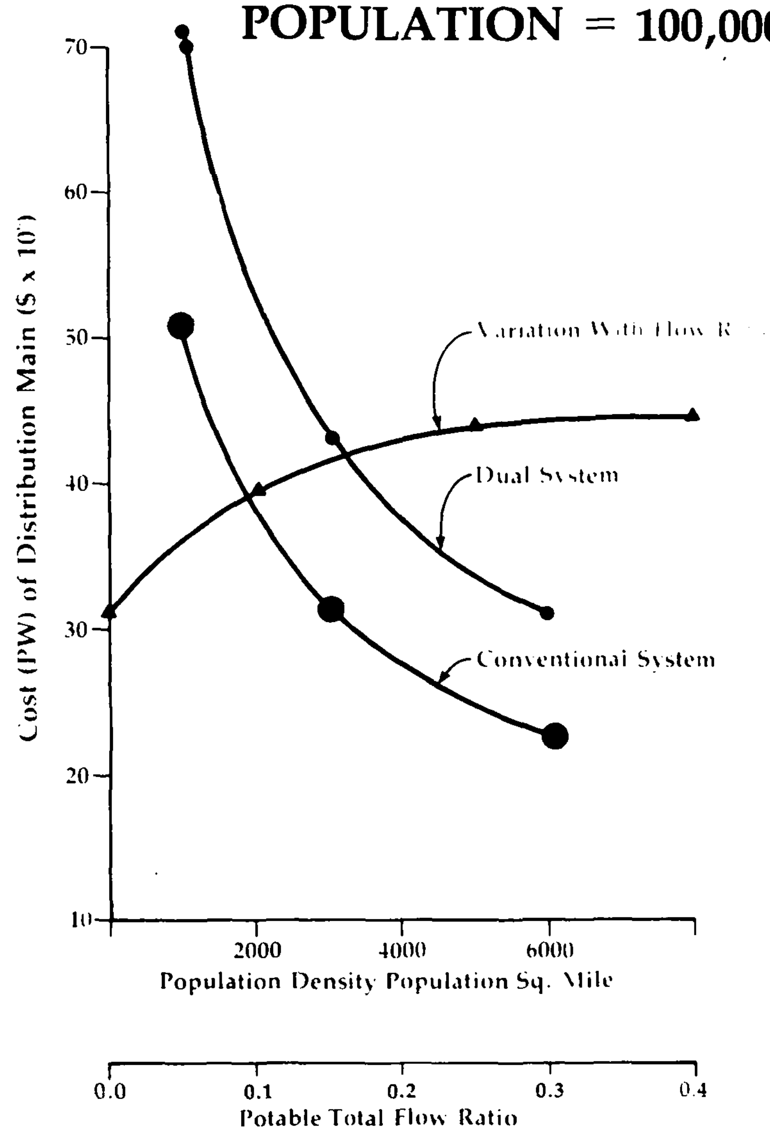
The other major, but less common clarification of point-of-use treatment is reverse osmosis. These units remove chlorine taste, chemical salt, and other trace impurities. No electricity is required but a bubble tank is necessary to increase the pressure to 40 psi.

The cost of point-of-use water treatment varies depending on the quality of water and wholesale or retail basis. The total cost for treating 1.5 gpcd or about 2,500 gallons per year per residence ranges from $.04/gallon to $.12/gallon for activated carbon and $.06/gallon to $.16/gallon for reverse osmosis home units.

CONCLUSIONS

The systems model developed in this study has been applied to five hypothetical test cases.

FIGURE 13 VARIATION OF COST OF DISTRIBUTION MAINS WITH POPULATION DENSITY AND POTABLE TOTAL FLOW RATIO POPULATION = 100,000



It has been found from these test case results that, if an activated carbon bed is required to produce potable water, the cost of a dual water supply system will be comparable to the cost of a conventional system. In cases where reverse osmosis is required in the treatment system to produce potable water, the cost of a dual water supply will be cheaper than the conventional single supply system. The effect of the rate of change of capital cost inflation rate on economic advantages of dual supply over the conventional system is less than that of O&M inflation rate.

These results undoubtedly reflect the assumptions made in the analysis regarding the sources of water for the potable and nonpotable supplies. Although certain general conclusions are possible, it would be unwise to accept these conclusions for all circumstances. It must also be stressed that the results that have been presented assume the development of new supplies.

The systems model developed in this study is a powerful tool for technical and economical analysis of various alternatives for long-term water supply management for urban areas. This model can analyze any configuration of water supply system including reuse using single, dual or multiple supply systems.

The important contribution of this study is that a general methodology has been developed for comparing the costs of single and multiple supply. This method can be used in any specific case by putting the proper values of the various costs and other economic parameters in the systems model.

REFERENCE

1. Haney, P.D. and Beatty, F.K., "Dual Water Systems-Design," JAWWA, July 1977, pp. 389.

SECTION 8

COMPUTER USAGE AND INPUT DATA PREPARATION

COMPUTER USAGE AND INPUT DATA PREPARATION

COMPUTER SYSTEM MODEL (WATMAN)

The computer system model developed for this study for WATer MANagement (WATMAN) provides the user with an economic and powerful tool for planning water supply management systems. The system's primary function is to derive capital costs, operation and maintenance costs, salvage value, and present worth costs. The present worth costs are based on the user defined base or starting year for each unit process in the water supply management system. The system also provides summary cost information for the entire operation. Currently, the system contains a library of 36 unit processes that may be included in the design of a water supply management system.

SYSTEM OVERVIEW

Design Considerations

In designing the Watman System, many factors were taken into consideration. Key items include:

1. System Capability - Of major concern, care was taken to insure that the final product would have the capability and flexibility to effectively model a variety of water supply management system configurations.

2. Data Preparation/Structure - A simple and straightforward approach to creating job decks and running the system was essential from the user viewpoint.

3. Data Verification - Extensive data input verification was required to insure that the computer system would not be wasting time and resources by producing meaningless results or recognizing a system error resulting in abnormal termination.

4. Maintainability/Modularity - It was important to recognize the distribution logistics involved with the Watman System and provide an easy and effective approach to maintaining the system. The system is designed on a modular basis to support this thinking.

5. Computer System Resources/Compatibility - Every attempt was made to limit memory and peripheral device requirements such that a minimal computer system would be required. The language chosen was based on industry compatibility (ANSI FORTRAN IV).

System Capabilities

Based on the design considerations, the following capabilities exist:

1. The system is capable of handling a conventional water supply management system configuration as well as a multiple water supply management system configuration.

2. The system is capable of handling a regional water supply management system configuration serving up to 10 cities.

3. The design flow of a unit process may be derived from total average water demand or from interior residential water demand and population data.

4. Up to 50 unique unit processes may be included in the design of a water supply management system.

5. Multiple water supply management system designs may be processed in a submittal.

6. Presentation of various types of output may be suppressed.

7. Extensive input data verification is performed with easily understood error messages.

System Design Constraints

As with any system, constraints are placed upon it due to design considerations and the requirements of the computer language used. The most significant constraints are given in Figure 1.

Hardware Requirements

Computer hardware requirements are given in Figure 2.

Functional Aspects

The design considerations, system capabilities, and design constraints of the Watman System define or establish a "general system environment," and the user must work within the confines of this environment.

FIGURE 1

SYSTEM DESIGN CONSTRAINTS

- 3 Water Supply Types
- 10 Communities/Cities
- 35 Years of Population Data Per City
- 50 Unit Processes
- 28 Variables Per Unit Process
- 10 Sets of Variables Per Unit Process
- 4 Unit Process Types
- 30 Year Planning Period
- Defined Order of Input Data
- Fixed Format Input (Generally 10 Columns)
- Batch Environment

FIGURE 2

HARDWARE REQUIREMENTS

- 132 Column Line Printer
- Card Reader
- Disc File
  - Direct Access
  - 100 Records
  - Record Length: 200 Bytes
- Memory
  - Program DEFCRT: 50,000 Bytes
  - Program WATMAN: 65,000 Bytes

The first step the user must take (phase 1) is to define the "user system environment." This may be the same as the "general system environment" or a subset. The user defines his environment by specifying information about cities (such as name, population data), general information defaults, and unit processes with associated default values for variables.

User specifications are read from cards and placed in a "system file" by program DEFCRT. Phase 1 is referred to as the "system file creation" phase. This step is generally only performed once, but the user may redefine the "user system environment" at any time.

The remaining step (phase 2) is to define the "water management system design (WMSD) environment." This may be the same as the user system environment or a subset. In essence, the WMSD environment specifies a unique system that the user employs to derive associated costs. The user defines his unique system by specifying information about cities (such as water demand), general information, and unit processes with associated data. User specifications are read from cards, associated data is read from the system file, water demand curves are derived, unit processes are called to calculate various costs, and information is output by program WATMAN. Phase 2 is referred to as the "model execution" phase. This is a repetitive step for each unique water management system design the user wishes to consider. The system execution environment is depicted in Figure 3.

PHASE 1: SYSTEM FILE CREATION

Functional Description

The program used to define the user system environment is DEFCRT. The program accepts input from cards, verifies the input data, and writes the data to the system file and to the line printer (see Figure 4). The program will continue to verify input if an error occurs unless it is a fatal error. In this case the program will terminate to avoid erroneous error messages. All error messages are easily understood with no additional documentation required. The user should maintain the output in a safe place as it is his reference document reflecting his user system environment.

Input Preparation

The data required to execute the DEFCRT program can be grouped into four categories:

- General input
- Community/city general input
- Community/city demand type input
- Unit process input

FIGURE 3

SYSTEM EXECUTION ENVIRONMENT

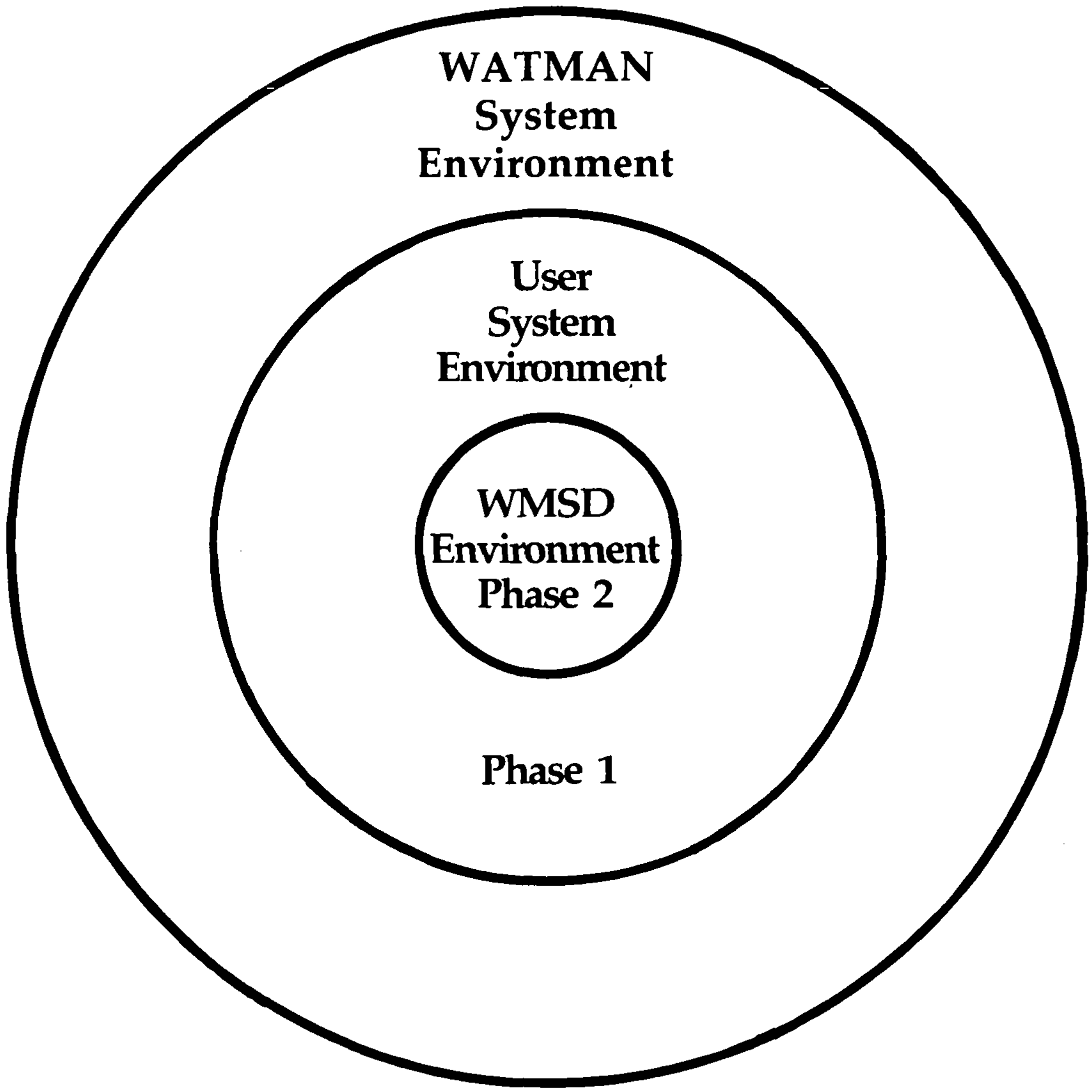
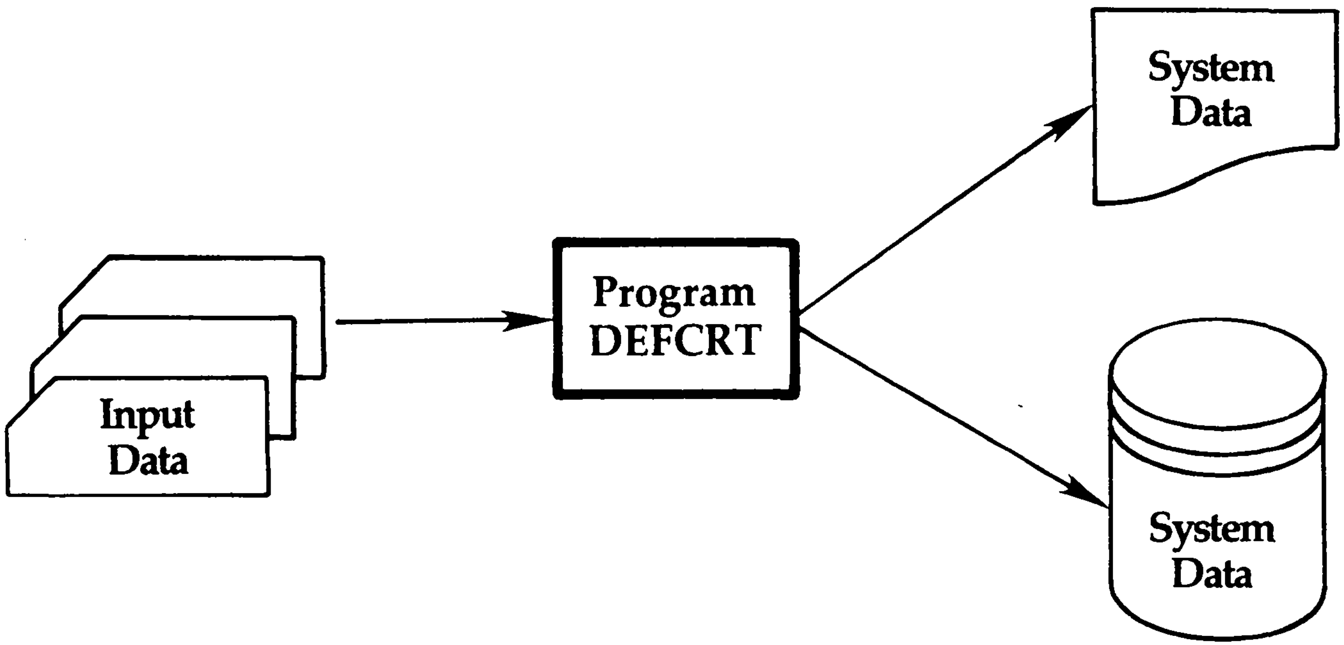


FIGURE 4

WATMAN SYSTEM OVERVIEW — SYSTEM FILE CREATION



The order of input is shown in Figure 5.

1. General Input

The general input is used to specify default values for the following:

- Planning period
- Annual interest rate
- Capital cost inflation rate
- Operation and maintenance cost inflation rate

In addition, the ENR index value and date that unit process cost default values are based on must be defined.

2. Community/City General Input

The community/city general input is used to specify general data for each city to be included in the user system environment. It consists of a set of cards for each city.

The general data card is used to specify the following:

- Unique city code
- City name
- Maximum to average daily flow ratio default value
- Number of years of population data
- Population data starting year
- Number of people per residence default value

This card is followed by one or more cards containing population data for the city.

The general data definition set for each city is placed one after the other, the last set being followed by an End of Community/City General Input card.

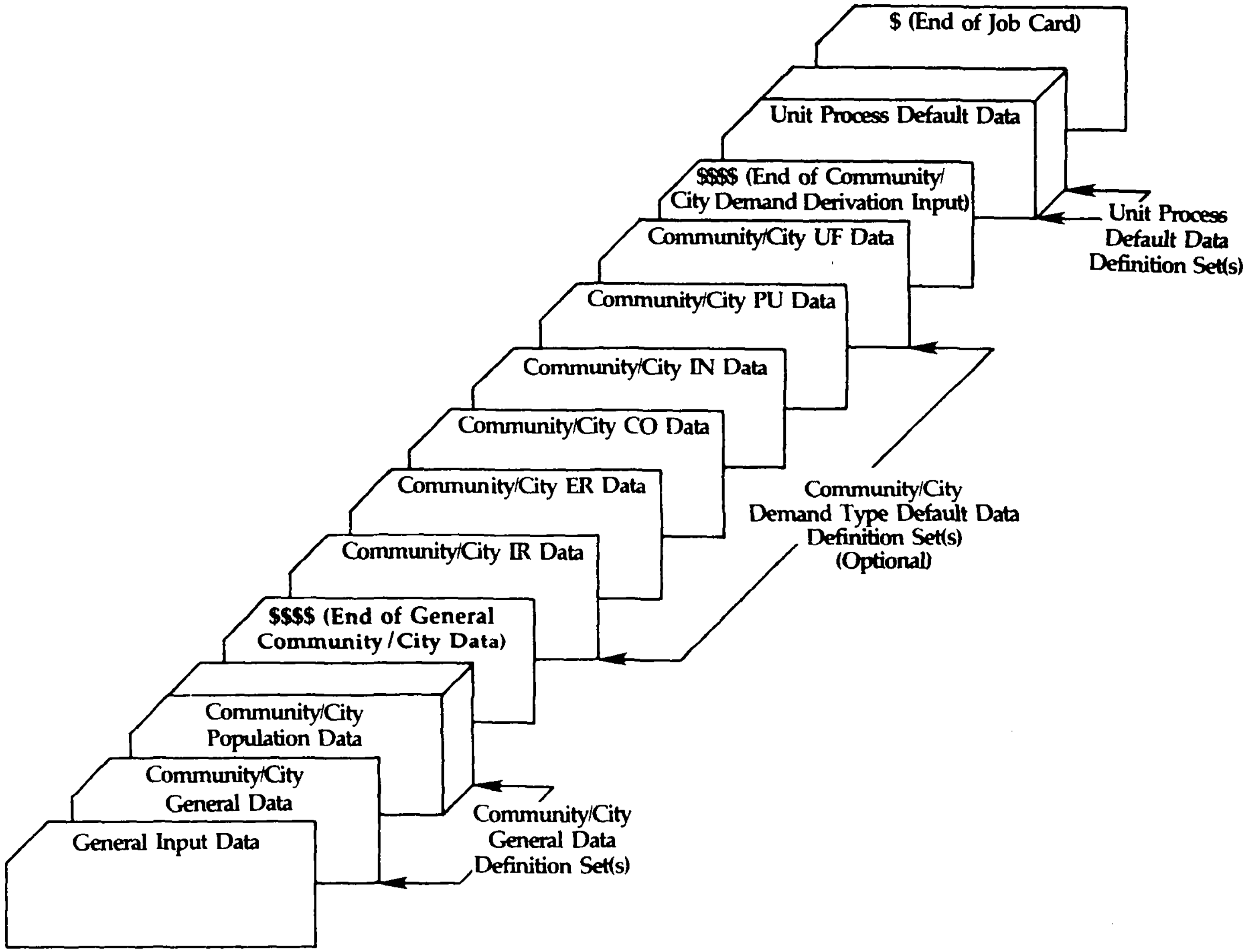
3. Community/City Demand Type Input

This input is used to specify the potable, subpotable, and nonpotable default fractions and the interior residential default coefficient for a city for the following demand types:

- Interior residential
- Exterior residential
- Commercial
- Industrial
- Public
- Unaccounted for

FIGURE 5

DEFCRT INPUT DATA



A card is required for each demand type.

Only cities specified in the community/city general input are valid and this input is optional. The last set is followed by an End of Demand Input card.

4. Unit Process Input

This input is used to specify the unit processes to be included in the user system environment and default values to be assigned to each variable of the unit process. Cards for each unit process are placed after each other, followed by an End of Job card.

PHASE 2: MODEL EXECUTION

Functional Description

The program used to define and analyze the water supply management system environment is WATMAN. The program proceeds as follows:

- Reads community/city general input from system file.
- Reads unit process names from system file.
- Creates several unit process tables.
- Reads run heading card.
- Reads output requirements card.
- Calls GENIN subroutine which reads the general input data card replacing blank fields read with defaults.
- Calls DEMDIN subroutine which reads demand data for each city to be included in the design and derives eight demand curves.
- Reads unit process definition set.
- Derives design flow curve(s) for the unit process.
- Calls appropriate unit process subroutine.
- Summarizes costs.
- Output of results.

An overview is given in Figure 6. Any input error will result in immediate program termination. All error messages are easily understood with no additional documentation required. Two important functional considerations the user must keep in mind are that cities and unit processes defined as a part of the WMSD environment must be part of the user system environment.

Input Preparation

The data required to execute the WATMAN program can be grouped into the following three categories:

- General system input.
- Community/city demand derivation definition set(s).
- Unit process definition set(s).

The order of input is shown in Figure 7.

1. General System Input

Input consists of three cards. The first card is a heading card. This is provided so that the user can supply a text heading to identify the run. This heading may be comprised of any alphanumeric characters in columns 1-80. Only one heading card is permitted and the heading will be printed on every page of output for the run.

The second card is an output requirement card. Five types of output may be selected as follows:

- General input data table.
- Demand curve tables for each city.
- Unit process input verification table.
- Unit process cost calculation tables.
- Cost summary table.

The third card is the general input data card consisting of the following:

- Starting year of planning period.
- Planning period.
- Annual interest rate.
- Capital cost inflation rate.
- O&M cost inflation rate.
- ENR index based on starting year.
- ENR index date.

FIGURE 6

WATMAN SYSTEM OVERVIEW — MODEL EXECUTION

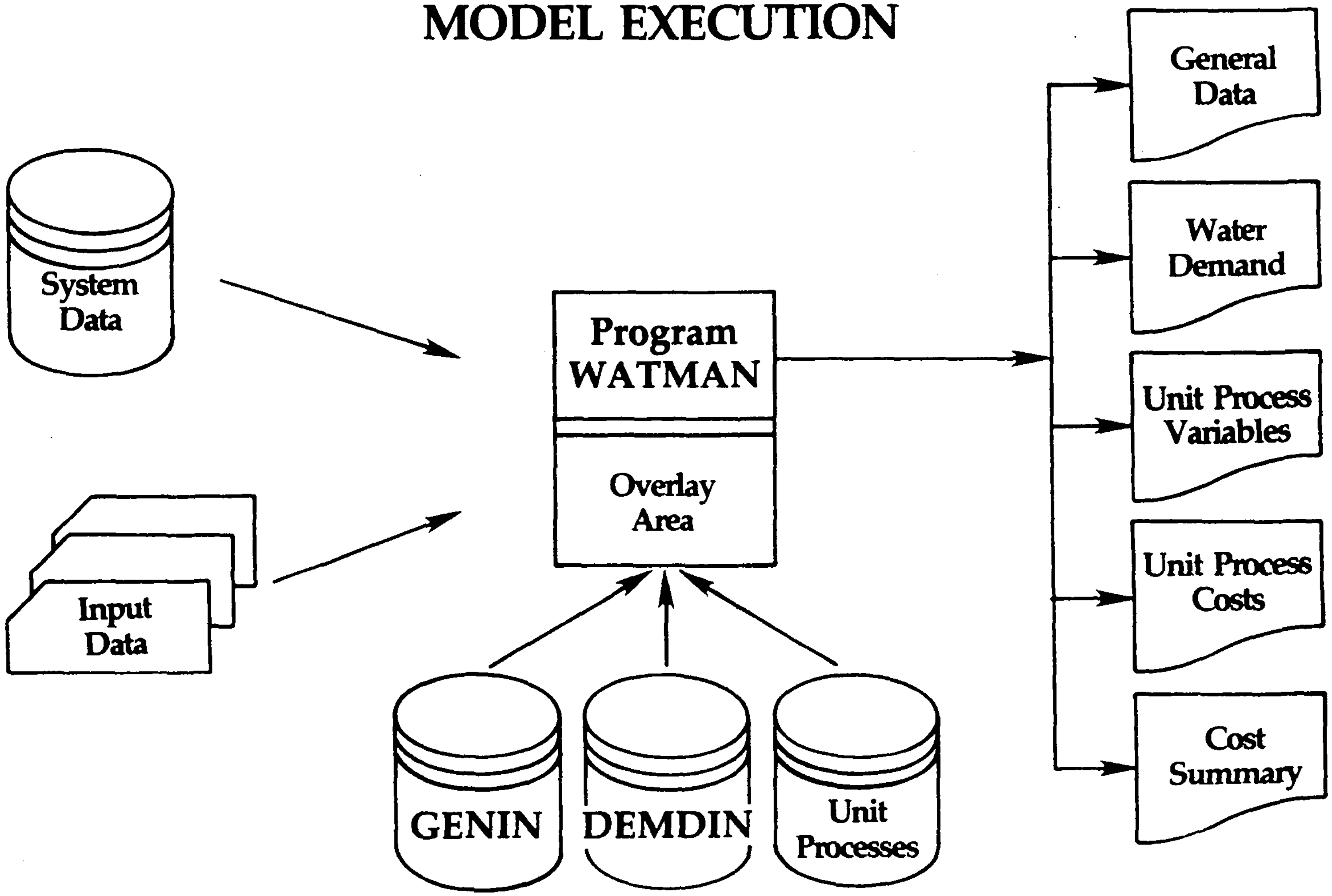
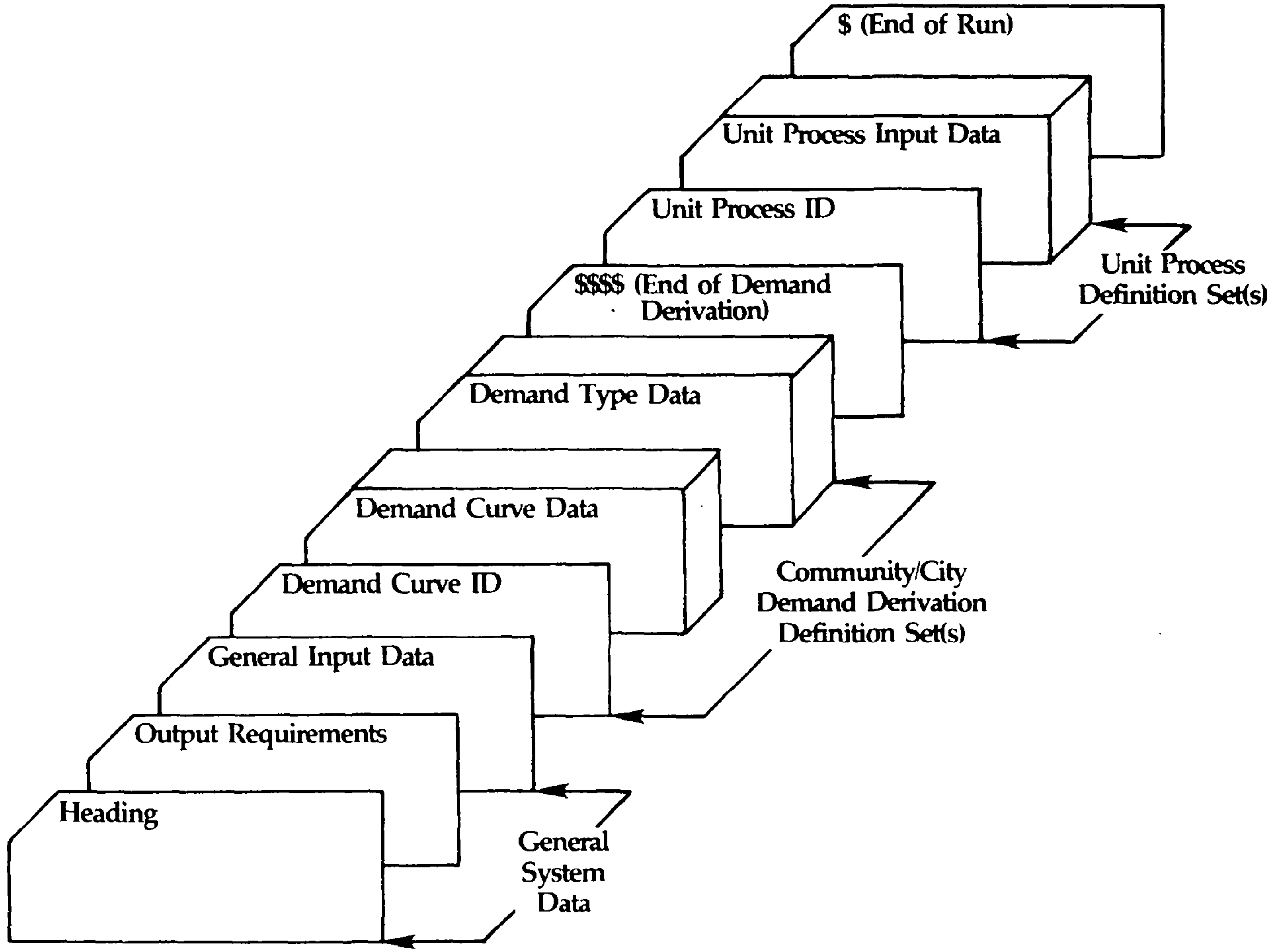


FIGURE 7

WATMAN INPUT DATA



2. Community/City Demand Derivation Definition Set Input

This input set is used to specify water demand data for each city included in the water management system design environment. Water demand derivation may be based on:

a. Total average daily demand (mgd) with potable and subpotable fraction (CODE = 1).

b. Interior residential demand (gpcd), demand type potable and subpotable fractions and IR coefficients, and population data (CODE = 2).

The first card in the set is a demand curve ID card consisting of:

- City code.
- Demand derivation basis.
- Maximum to average daily flow ratio.
- Potable fraction (if CODE = 1).
- Subpotable fraction (if CODE = 2).

The next group of cards is used to specify the demand data for each year of the planning period. If CODE = 1, units would be (mgd). If CODE = 2, units would be (gpcd). One card is required for every 10 years of planning period.

The next group of cards is required if CODE = 2, specifying the demand type, potable and subpotable fractions, and IR coefficient. Six cards are required.

A community/city demand derivation definition set should be included for each city considered in the configuration. This section of input defines the cities to be included in the water management system design environment.

3. Unit Process Definition Set Input

This input set is used to specify a unit process that is to be included in the WMSD environment, the water supply type (quality), communities/cities utilizing the unit process, and values associated with the unit process variables.

The first card in the set is a unit process ID card consisting of:

- Unit process type.
- Unit process number.
- Water supply type code (see following table).
- One or more community/city codes.

Water Supply Type Codes

Code	Description
1	Total (Potable + Subpotable + Nonpotable)
2	Potable only
3	Subpotable only
4	Nonpotable only
5	Potable + Subpotable
6	Potable + Nonpotable
7	Subpotable + Nonpotable

The next group of cards is used to assign values to unit process variables. One card is required for every eight values. The number of values required varies from one unit process to another.