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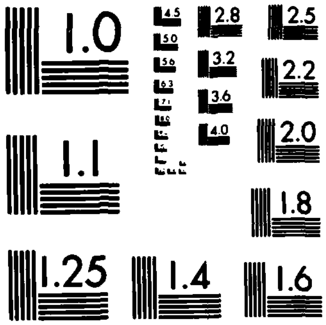

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WATER SUPPLY ANALYSIS FOR THE GUAM COMPREHENSIVE STUDY

by

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This report presents an analysis of five types of water supply alternative plans for public water use on Guam. These include (a) northern lens aquifer plus military sources, (b) northern lens aquifer only; (c) northern lens aquifer plus southeastern river; (d) southeastern river development only, and (e) southeastern river development plus military sources.

There are two parts to the report. The first part, "Water Distribution

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20. ABSTRACT (Continued).

System Analysis," presents the results of a water balance for the five types of alternatives under three water use projections, and documents the collection of data and development and calibration of the MAPS (Methodology for Areawide Planning Studies) water distribution system for Guam. The model is intended to be turned over to the Government of Guam.

In the second part, "Economic Analysis of Alternatives," conceptual designs are presented for each type of alternative for three water use projections. These designs include source, treatment, and major distribution facilities. The MAPS computer program was used to prepare cost estimates and convert capital and operation and maintenance (O&M) estimates into average annual cost for economic evaluation.

In general, alternatives relying on the northern lens aquifer were less expensive because of the large capital cost associated with large dams. The large dams with centralized treatment should produce better quality water. Use of several types of sources should reduce the stresses on the northern lens aquifer.

PREFACE

This report presents the results of the water supply task of the Guam Comprehensive Study (GCS). This work was conducted by the U. S. Army Engineer Waterways Experiment Station (WES), Vicksburg, Miss., for the U. S. Army Engineer Division, Pacific Ocean (Honolulu District), under InterArmy Order PODSP-CIV-81-39.

This report was prepared by Dr. Thomas M. Walski, Water Resources Engineering Group (WREG), Environmental Engineering Division (EED), Environmental Laboratory (EL), WES. He was assisted by Ms. Cheryl M. Lloyd, WREG. Technical review was provided by Dr. Joe Miller Morgan, WREG. Chiefs of the WREG and EED were Messrs. Michael R. Palermo and Andrew J. Green, respectively. Chief of the EL was Dr. John Harrison.

The study manager for the GCS at the Honolulu District was Mr. Gene P. Dashiell, Project Formulation Section of Planning Branch. The principal engineer was Mr. James D. Emerson of the Hydraulics Section. Division Engineers during this study and publication of this report were BG Henry J. Hatch and COL Robert M. Bunker.

Commander and Director of WES during conduct of the study was COL Tilford C. Creel, CE. Technical Director was Mr. F. R. Brown.

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CONTENTS

PART I

PART II

PART I

WATER DISTRIBUTION SYSTEM ANALYSIS

1. Introduction

Background

The U.S. Army Engineer Division, Pacific Ocean (POD), Honolulu District, is conducting the Guam Comprehensive Study for water and related land resources (GCS). The U.S. Army Engineer Waterways Experiment Station (WES) was requested to provide technical assistance to the Honolulu District in carrying out the water supply portion of the GCS.

While the primary interest of the Honolulu District is the possibility of providing additional sources of water, it was necessary in the study to also analyze the treatment and distribution of water in Guam since different sources of water require different treatment and distribution systems. Therefore, in order to properly determine the economic benefits and costs of the alternatives (since the benefits of Federal water supply projects are measured using the costs of the most likely non-Federal alternative), it was necessary for WES to calculate the costs of treatment and distribution systems other than for the Federal Plan.

Overview

A considerable portion of the WES effort was spent developing an understanding of the existing Public Utility Agency of Guam (PUAG) water supply system. This was done on two levels. First, water balances were performed on a village basis for several alternative development scenarios under several growth projections to identify source development requirements. These water balances did not take into consideration system hydraulics, but merely the volumes of water required at the village level and the availability of water from various sources. It was assumed that an adequate distribution system could be constructed for any alternative.

Secondly, an analysis was performed by WES using the Hardy-Cross method portion of the Methodology for Areawide Planning Studies (MAPS) computer program developed at WES. In this portion of the study a model of the distribution system was constructed and calibrated for four subareas on Guam. The model was then used to locate and investigate problem areas in the distribution system. The model was found to be very useful and will be given to the Government of Guam to assist in the future management of the system.

This is the first part of a two-part final report. This part contains the results of the water balance analysis and a discussion of the development of and results from the water distribution analysis. The second part consists of an economic analysis of the alternatives.

Section 2 of this part contains the results of the water balance. Section 3 describes the data collection effort required to develop and calibrate the water distribution model. Section 4 contains a description of the calibration of the model. Section 5 presents a discussion of anticipated problems in the distribution system under future water use. Appendices A and B contain the User's Guide and Documentation of the MAPS Water Distribution Program. Appendix C contains sample results of the calibration runs. Appendix D contains maps of the distribution system model, while Appendix E contains a description of the program being given to the Government of Guam along with some instructions for its use.

Description of System

The PUAG water supply system is a composite of many types of sources, treatment, storage, transmission lines, and operating strategies. The PUAG relies on wells in the northern part of the island as the primary source of water, although it also operates surface, spring, and well sources in other areas and purchases water from the U.S. Navy.

Treatment generally consists solely of chlorination at the source (well or spring), although more conventional treatment is used at surface sources. Ground-level tanks are generally used for storage, although there are some elevated tanks.

Very little booster pumping is used as sufficient pressure head is generally provided by well pumps or gravity flow from storage. The distribution system includes a wide variety of pipe materials.

The PUAG system is divided into four regional systems. The regional water system boundaries are shown in Figure 1-1. The areas not included in the PUAG system are undeveloped or served by either the U.S. Air Force or U.S. Navy systems.

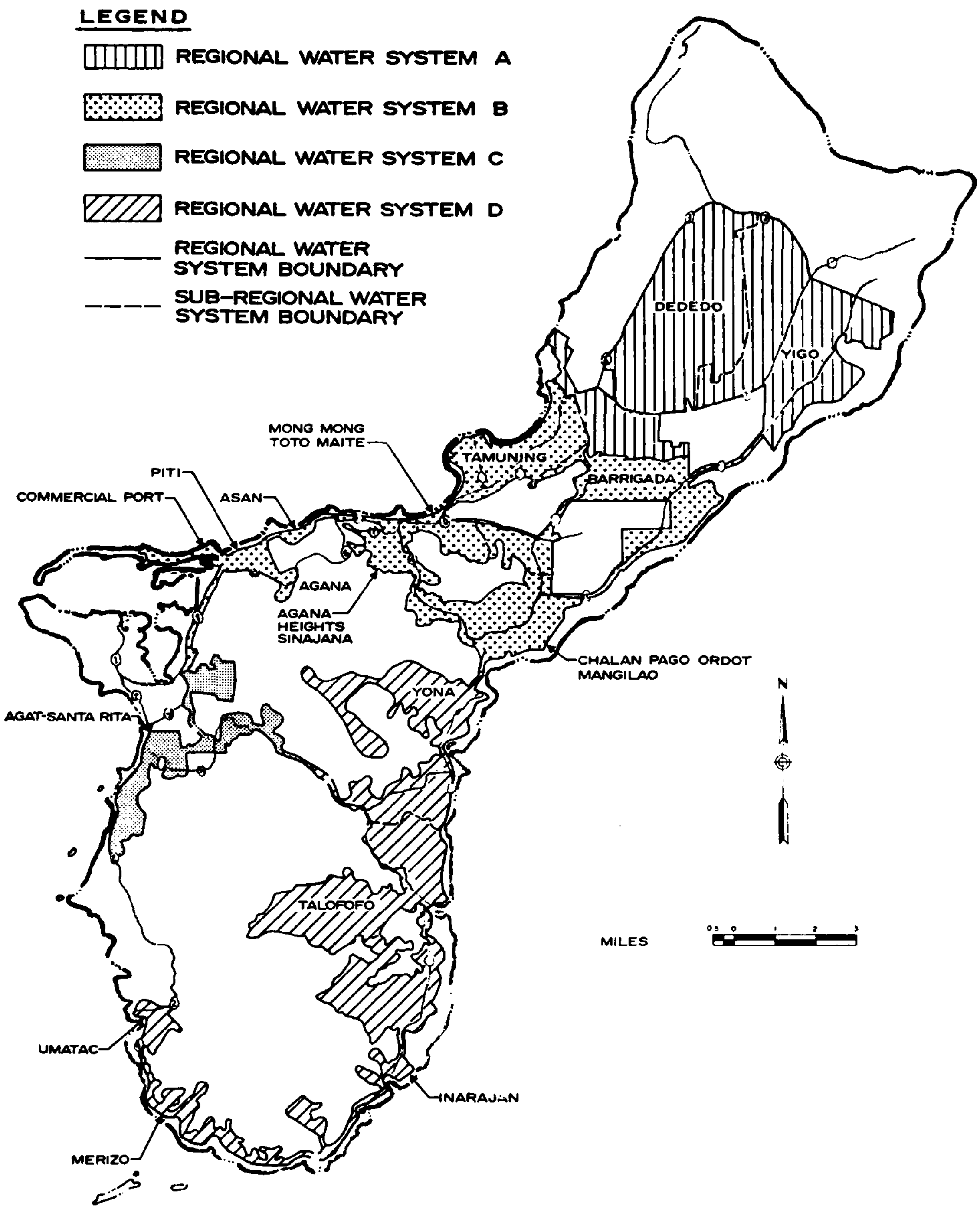


Figure 1-1. Service areas in Guam

2. Water Balance

Introduction

A great deal of information related to water supply problems and their potential solutions can be developed fairly easily by performing a water balance for the PUAG System. This balance is based on average water use and source yield for an array of different water demand projections and distribution and source development scenarios.

Sources

There are essentially three sources of water on Guam which can be used by the PUAG. They are (1) the northern groundwater lens, (2) the Navy system using the Fena reservoir and treatment plant, and (3) a new surface water reservoir in one of the southeastern river valleys. (In this report, this option will be referred to as the Ugum River Dam, although other sources are feasible.)

Scenarios

For the water balance, the existing PUAG sources are assumed to continue producing water throughout the study period. Five scenarios were formulated for the most likely combinations of additional source development. These are:

1. Groundwater development plus Navy source.
2. Groundwater development only.
3. Groundwater and Ugum River development.
4. Ugum River plus other southeastern rivers.
5. Ugum River plus Navy source.

The results of the water balance for each of these scenarios is discussed in detail later. The facilities associated with each of these scenarios are presented in Section 1, Part II, of this report.

Water Use

Water use estimates for the water balances are based on the population projections provided by the Guam Bureau of Planning. The population projections were converted to water use based on per capita water use estimates from the Master Plan (Water Facilities Master Plan;

Barrett, Harris & Associates, Inc. 1979) as shown below for each service area.

Service Area	Per Capita Use
Yigo, Dededo (Service Area A*)	80 gpcd**
Remainder of Island (Service Area B)	145 gpcd
Agat, Santa Rita (Service Area C)	100 gpcd
Umatac, Merizo, Inarajan, Talofofo, Yona (Service Area D)	105 gpcd

* These designations correspond to those used in the Master Plan.
** Gallons per capita per day.

The water use for each village, based on the above per capita rates, is shown in Table 2-1 for the three time windows considered (1976, 2000, and 2035). The total water use is projected to double from 1976 to 2000 and increase by 17 percent in the following 35 years. One problem made evident from Table 2-1 is that total use in 1976 is calculated to be only 9.72 mgd, while in the Master Plan water production plus purchase is reported as 17.7 mgd. The differences are due to "unaccounted for" water and large commercial and industrial users. In order to include these water sinks in the water requirements to be used in the mass balance, the values in Table 2-1 must be modified.

The uncertainty in the use and population projections can best be accounted for by performing the water balance for a range of water requirements. In this study three sets of water uses are examined in the water balance:

1. Low
   a. 2000 - Water use from Table 2-1 plus 4.1 mgd added for agricultural/commercial use as per Table 5-25 of Master Plan
   b. 2035 - 2000 use times 1.17
2. Medium
   a. 2000 - Taken from Master Plan Table 5-25 (28.9 mgd)
   b. 2035 - 2000 use times 1.17
3. High
   a. 2000 - Water use from Table 2-1 times 1.97, which is ratio of 1976 production to domestic use
   b. 2035 - 2000 use times 1.17

Table 2-1

Water Purchased by Village

Village		1976 (gpm)	2000 (gpm)	2035 (gpm)
1.	Dededo	1215	2014	2356
2.	Yigo	339	672	786
3.	Tamunig-Tumon	1193	2769	3240
4.	Barrigada, Mangilao, Mongmong-Toto-Maite, Chalan Pago-Ordot	1857	4130	4832
5.	Agana	64	257	300
6.	Agana Hgts-Sinajana	501	881	1030
7.	Asan	145	272	318
8.	Piti	158	266	312
9.	Yona	299	617	722
10.	Santa Rita	222	351	410
11.	Agat	294	653	764
12.	Talofofo	157	195	228
13.	Umatac	51	117	136
14.	Inarajan	130	202	236
15.	Merizo	119	188	220
	Total	6,744	13,584	15,890
		(9.72 mgd)	(19.57 mgd)	(22.9 mgd)

The average day water use for the PUAG system in million gallons per day is given below.

	Low	Medium	High
2000	23.7	28.9	38.6
2035	27.7	33.7	45.1
Per capita use (gpcd)	141	172	230

The per capita use rates are based on a civilian population of 167,500 in 2000.

Existing Sources

For the water balance, new sources are brought on line only when the capacity of existing PUAG sources is exceeded. The capacity of surface water and spring sources is given in Table 4-4 of the Master Plan and is shown below as Table 2-2.

Table 2-2

Source Capacity

Source	Capacity (gpm)
Asan Spring	125
Santa Rita Springs	50
Ylig River	250
Geus Dam	70
Siligen Spring	10
Laelae Spring	65
La Sa Fua River	30
Total	600 (0.86 mgd)

Groundwater source capacities were taken from Appendix D of the Master Plan and are listed in Table 2-3 by the Village in which they are

Table 2-3

Well Capacity

Village	Capacity (gpm)
Yigo (AG*+Y)	541
Dededo (D+F)	2705
Barrigada et al. (A+M)	3675
Talofofo (T)	152
	7073 = 10.1 mgd

* Capital letters refer to well series as defined in the Master Plan.

located. Note that the numbers in Table 2-3 are 80 percent of the values of Appendix D. This is to account for downtime and manual operation of the wells.

The total surface water capacity in Table 2-2 of 0.86 mgd agrees roughly with Table 5-3 of the Master Plan which gives surface and spring production of 0.92 mgd. The total well capacity in Table 2-3 is somewhat lower than the 14.19 mgd well production given in the Master Plan. This is probably due to the fact that capacity is not given in Appendix D of the Master Plan for nine of the wells reflected in Table 2-3. This figure of 14.19 mgd requires each of these wells to have a capacity of 308 gpm which is higher than that reported for any of the existing wells.

Inconsistencies in the data on source capacity, production, and water use should be kept in mind when interpreting the numbers reported in the results of the water balances. In general, a range of values has been given and it is left to the reader to decide which value is more reasonable. At the very least this should serve to cause the reader to appreciate the uncertainty associated with the water balance calculations.

Results of Water Balances

The results of the water balances for the five scenarios investigated are presented in the following sections. The results are shown graphically and flows at critical points in the system are given in matrix form for several sets of conditions. The three rows of the matrices correspond to the low, medium, and high water use projections given earlier and the two columns represent the 2000 and 2035 time frames. For example, in Scenario 1, the flow between Village 4 (Barrigada et al.) and 9 (Yona) for the medium use projection in 2035 is shown in the second column, second row (2.29 mgd). An arrow along a line indicates direction of flow. A negative flow indicates flow in the direction opposite the arrow.

Scenario 1: Groundwater Development Plus Continued Use of Navy (Figure 2.1). This scenario represents the status quo, with the military (chiefly the Navy) providing 2.6 mgd, the PUAG providing 0.9 mgd from surface and spring sources, and the remainder coming from wells. In this scenario, the Agat-Santa Rita area, which is presently served by the Navy, will continue to be so served and will not be connected to the remainder of the system except through Navy lines. By 2035 the Navy will supply from 1422 gpm (2.05 mgd) to 2327 gpm (3.35 mgd) to the areas it serves. The advantage of continuing use of Navy sources is that the Navy takes its water from the Fena Reservoir in the southern portion of the island and any water taken from this source reduces the stress on the northern groundwater lens. Even so, this scenario calls for from 5240 gpm (7.55 mgd) to 16,453 gpm (23.71 mgd) of additional groundwater to be pumped from the northern lens. The present pumping rate is 18.3 mgd, according to the Master Plan, and the estimated yield is approximately 50 mgd. Therefore, adequate water is available, although there will be little safety margin. Continued use of Navy facilities also will eliminate the need for the Cross Island pipeline along Route 17 and will allow elimination (or reduction in size) of the line connecting Asan and Agana. The southern portion of the island will receive from 1270 gpm (1.83 mgd) to 2192 gpm (3.16 mgd) from the north to supplement its surface sources by 2035.

Scenario 2: Groundwater Development Only (Figure 2-2). This scenario corresponds to the proposed Master Plan. In this plan net purchase from Navy sources will be zero, although water may be traded.

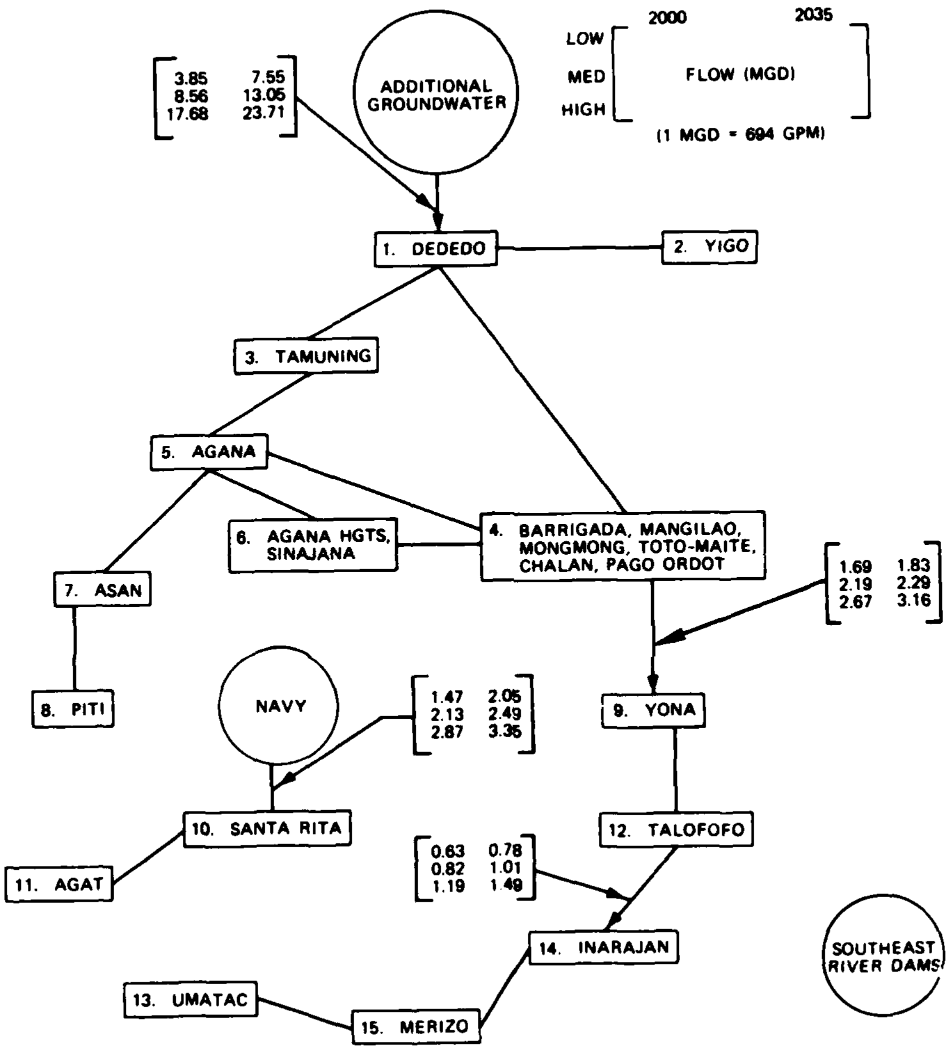


Figure 2-1. Scenario 1 - Groundwater + Navy

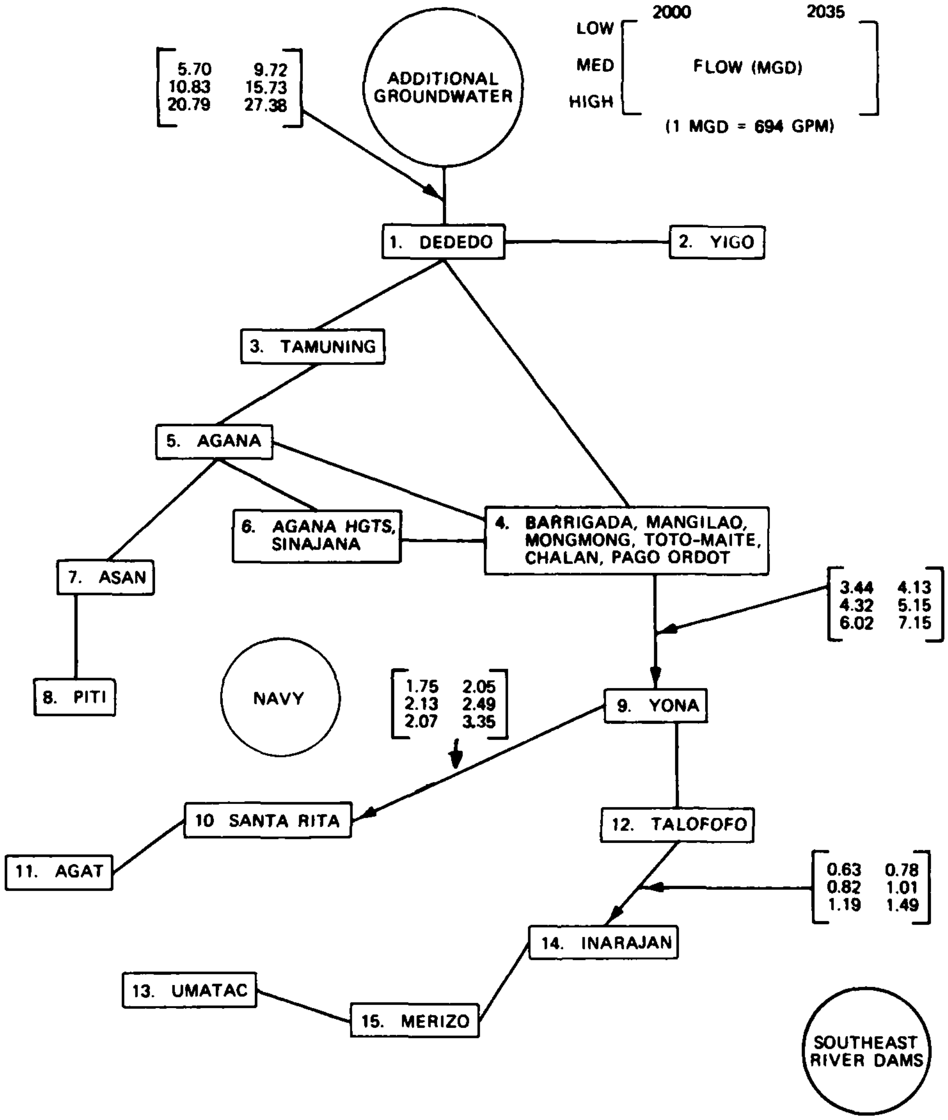


Figure 2-2. Scenario 2 - Groundwater Only

For this alternative, all of future development must be met from the northern lens, and the Asan-Piti-Ninitz Hill and Agat-Santa Rita areas will be connected to the remainder of the PUAG system. The northern lens must provide from 6747 gpm (9.72 mgd) to 79,000 gpm (27.38 mgd) additional water by 2035. Unless the distribution system is repaired to eliminate losses and/or conservation is implemented, the northern lens will be stressed near its limits. This scenario calls for an additional 100 wells (assuming approximately 200 gpm/well) and will probably result in significant operation and maintenance problems as well as possible water quality problems if current operation is indicative of future operation. Rather than chlorinate the water at each well and pump it directly into the system, it may be better to collect water at a central point in each wellfield, treat it there, and then pump it into the system. This should improve water quality control and simplify operation. It may even be economical since the pumps at the wells can be smaller and chlorinators will not be required at each well. (The previous statements are true for all scenarios using wells, but are mentioned here since this scenario relies on wells most heavily.) In this scenario, the water transported to the south will double that required in scenario 1 since water for Agat-Santa Rita must pass through Yona on its way to the Cross Island pipeline. Trading water with the Navy is possible, with the Navy providing a gallon of water to Agat-Santa Rita for every gallon it receives from, for example, Barrigada.

Scenario 3: Groundwater Plus Ugum River Dam (Figure 2-3). In this scenario the Ugum River Dam will, as discussed in the Ugum River Interim Report (Honolulu District 1980), be constructed by the year 2000 and will yield 6246 gpm (9.0 mgd). This water will be supplemented by additional groundwater development in the northern lens, which can range from 0.72 to 18.38 mgd depending on use. This plan eliminates the need for connections with the Navy except for emergencies, and will protect groundwater from overdrafting and subsequent salinity problems. Since there will be a large central treatment plant and pumping station, operation should be considerably simpler than for the 100+ wells in scenario 2, and water quality should be excellent.

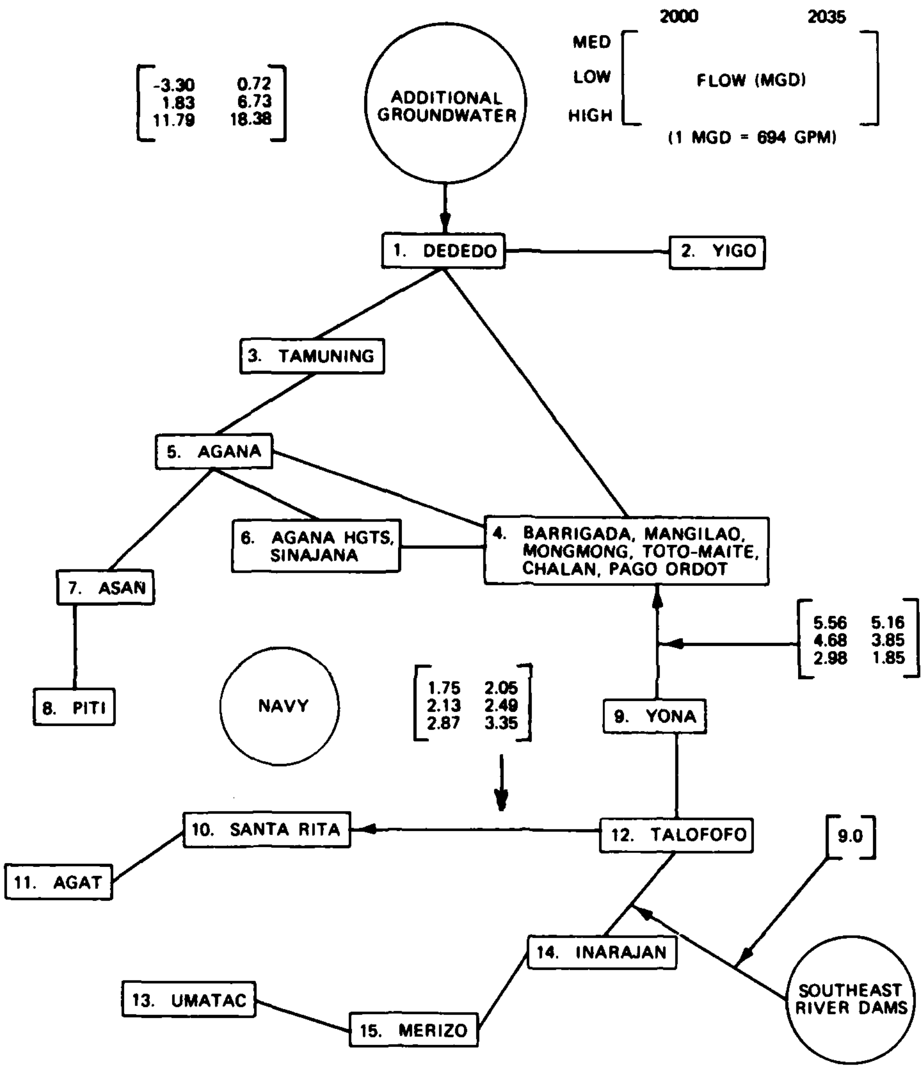


Figure 2-3. Scenario 3 - Ugum River + Groundwater

Scenario 4: Southern Surface Water Source Development Only (Figure 2-4). This scenario represents the case in which no additional groundwater development occurs and the water requirements are met by one or more reservoirs in the southern portion of the island. (Note that in Figure 2-4 this alternative is referred to as the Southeast River Dams, which consist of the Ugum and Inarajan Dams). In this scenario the stresses on the northern lens are greatly relieved and, as a result, water quality should improve. Instead of building separate chlorination

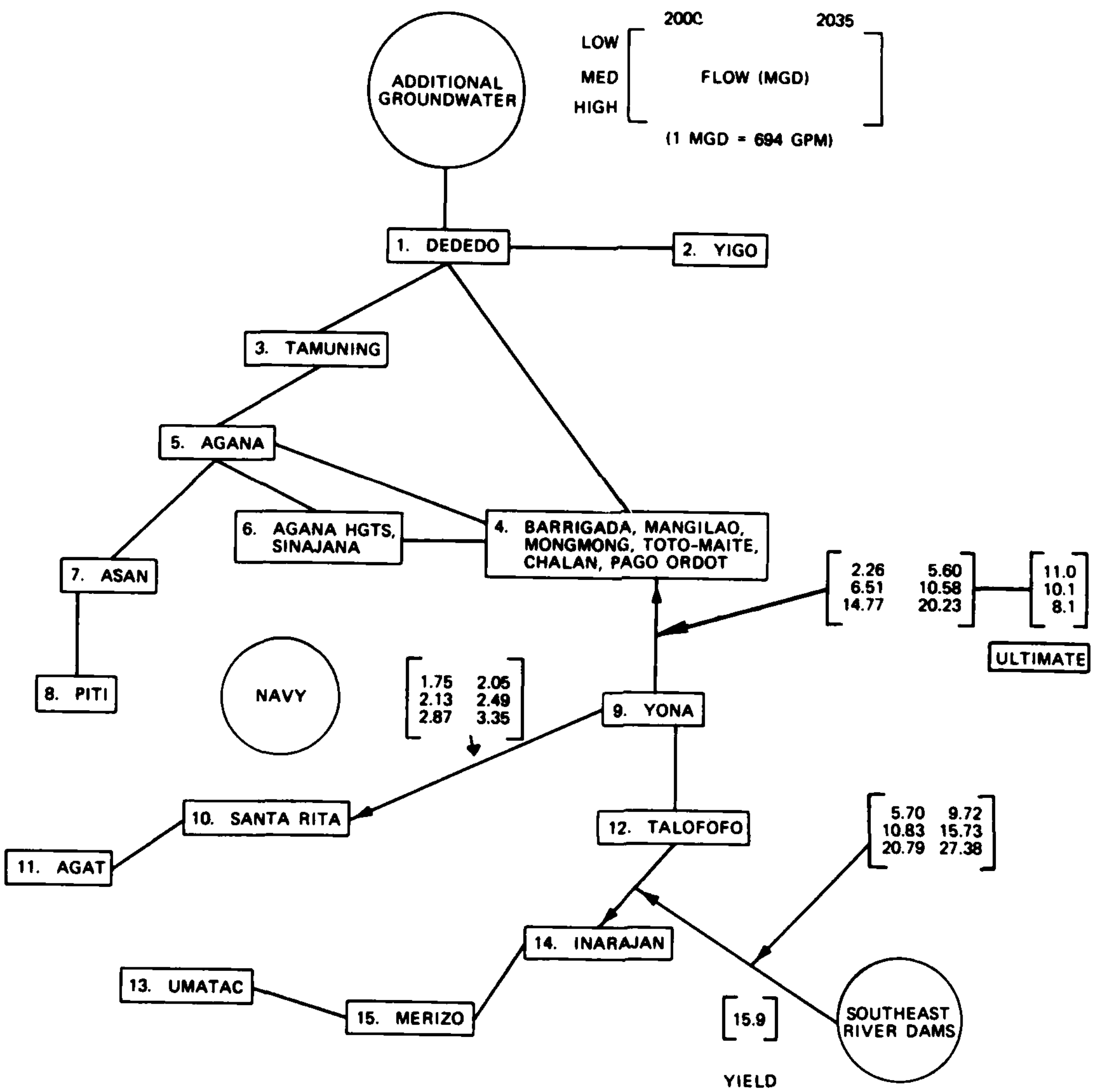


Figure 2-4. Scenario 4 - Southeast River Dams Only

facilities at each well, a centralized, modern, automated treatment plant can be built. The transmission cost to pump this water north to the high use areas in Tamuning and the large capital costs involved with dam construction will result in higher costs than some other alternatives. This alternative is most attractive if additional groundwater cannot be developed and connections with the Navy must be eliminated. If all island demands are met from the Southeast dams, the line from Yona to the north would carry from 5.60 to 20.23 mgd in 2035 assuming unlimited source capacity. Since total yield from the dams is 15.9 mgd, and the southern villages must be served first before pumping north, the actual ultimate flow that can be pumped north is given in the block labelled "ultimate." Note that in the low projection, there will be unused capacity even in 2035.

Scenario 5: Southern Surface Plus Navy (Figure 2-5). This scenario is similar to scenario 4 except that Navy connections would continue to be used for Asan-Piti-Nimitz Hill and Agat-Santa Rita. This would eliminate the need for a Cross Island road pipeline, and reduce the size (and possibly number) of the required reservoir(s). This plan also has negligible impact on the northern groundwater lens and would allow simple operation and good water quality. It will require a large pipeline connecting the reservoirs with the northern use areas. Under the high use projection, both the Ugum and Inarajan Dams must be built. Under the other projections only the Ugum Dam is required.

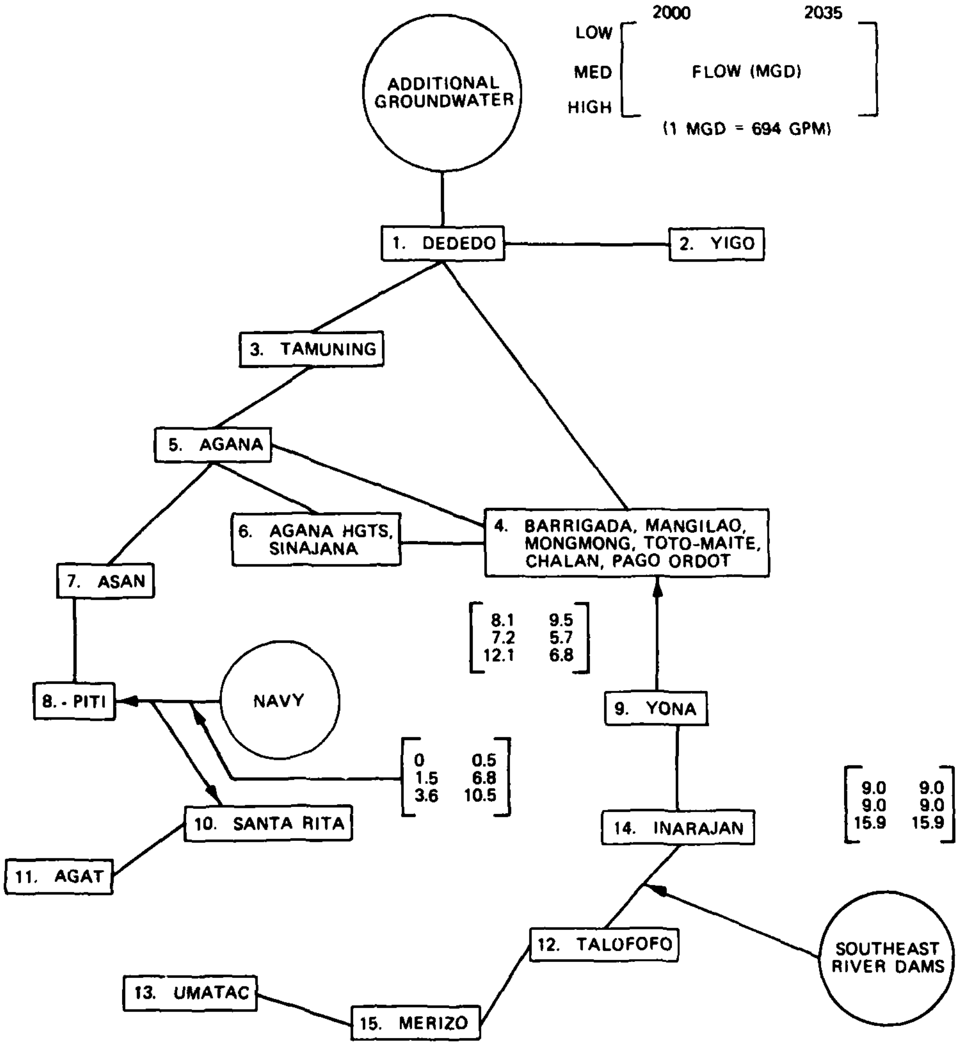


Figure 2-5. Scenario 5 - Southeast River Dams + Navy

3. Data Collection for Hydraulic Model

Most of the data used in this analysis were taken from the Master Plan. These data were supplemented by and cross checked with data from a variety of other sources including the GCS Stage 1 Report and the Ugum River Interim Study. A more detailed description of the sources of particular types of data is given below. Published data were supplemented by field observations and tests conducted by the Honolulu District and WES personnel with the assistance of PUAG personnel during August 1981.

Service Areas

The island has been divided into four "service areas" in the Master Plan, conforming to the Bureau of Planning's Land Use Plan. These service areas are:

A - Dededo, Yigo, and other northern areas;

B - South of Dededo to Piti in the west and Pago Bay in the south;

C - Agat-Santa Rita;

D - South portion of island from Pago Bay to Umatac.

It would be complicated and expensive to simulate the entire system at one time with the MAPS computer program, and it is not necessary to do this since some areas are separated from the others, or connected only through a booster pump or pressure-reducing valve. In addition, the boundaries between the service areas listed above are not convenient points at which to break off a hydraulic model. Therefore, for modeling purposes, it was necessary to divide the island into a different set of "subareas" related to the service areas as described below.

Subarea	Service Area In Master Plan
AB	A and B minus Harmon, Yigo, Mt. Santa Rosa, Barrigada Heights, Asan, Piti, and Nimitz Hill
C	C minus Sinfa Reservoir Area
D1	D north of Malojloj Pump Station
D2	D south of Malojloj Pressure-Reducing Valve

These subareas were simulated using the MAPS program. The areas not included were generally separate and so simple that it was best to use hand calculations.

Population/Water Use

Population data were taken from a document entitled "Revised Village Population Projections for the Year 2000" dated June 1977 and transmitted from Betty S. Guerrero, Bureau of Planning, to the Honolulu District on 31 March 1981. This document contains existing population and projected 2000 population broken down by village. The 2030 population was determined based on 3/4 percent growth per year for 1980-2000 and 1/4 percent growth per year from 2000-2035 as given in "Table 29" which was apparently taken from the Apra Harbor Survey Report and cited on Table All in the Ugum River Interim Study. This corresponds to a 17 percent growth from 2000 to 2030.

In developing water use from population data, the Master Plan used 80, 145, 100, and 105 gpcd for service areas A, B, C, and D, respectively (service areas as defined in Master Plan). The sum of water produced (15.11 mgd) and purchased (2.59 mgd) by the PUAG in 1977 is 17.70 mgd according to the Master Plan. This corresponds to 208.9 gpcd (17.7 mgd/84,701 people). In general, the ratio of water produced (plus purchased) to water used was 1.5, so, in the mathematical model runs, pressures and flows were simulated for the projected water use and twice the water use in order to bracket the possible pressures.

Water System Maps

The most important information required for modeling a water distribution system is a map of the distribution system. For this work, the skeletal system to be modeled was drawn on tracing paper overlaid on 1:24,000-scale U.S. Geological Survey (USGS) topographic maps. All of the elevations, pipe diameters and lengths, tanks, pressure-reducing valves, booster pumps, and wells were located on the maps.

There were several sources of data from which to develop maps of the water distribution system. The primary source was the "Existing Islandwide Water Facilities System Maps" prepared as part of the Master Plan. There were also two plan maps and one profile map in the Water

Facilities Master Plan, a plan and profile map in the Agat-Santa Rita and Yigo Sanitary Surveys, a set of blue line maps of the southern portion of the island, and a map from the Ugum River Interim Report. Data on elevations were taken from quad sheets and the system profile in the Master Plan. In some cases, the data from the various sources were inconsistent, so some judgment had to be made as to which source was more reliable. (Generally, the "Existing Islandwide Water Facilities System Maps" were used.)

The location of wells was taken from Figure 4-3 of the Master Plan and the capacity and head at the wells was taken from Appendix D of the Master Plan. Pressures and capacities of all of the booster pumping stations were not available in the Master Plan. These data were provided in a letter from the PUAG dated 6 June 1981. The upstream and downstream pressures at pressure-reducing valves were also provided in the same letter.

Additional Data Collection

In order to properly calibrate the water distribution model, it was necessary to know the pressures throughout the distribution system while also observing water elevations in tanks, and the pressures at pumps, wells, and pressure-reducing valves at roughly the same time. Virtually no pressure data could be found, except for some sketchy data in the Agat-Santa Rita and Yigo sanitary surveys, and it was felt that additional data collection was necessary to calibrate the model. Personnel from Honolulu District, PUAG, and WES performed pressure and flow tests and observed operation of the PUAG water distribution system during a field trip.

The primary purpose of the field testing on Guam was to collect sufficent data to enable WES to properly calibrate the network model of the PUAG water distribution system that WES has developed. Independent of the model, the data can be used to gain a quantitative understanding of the operation of the system and to predict fire flows from hydrants tested for insurance rating purposes.

Several types of data were collected. They include;

1. Static pressure at hydrants,

2. Pressure while nearby test hydrant was opened,
3. Flow from test hydrant,
4. Water levels in reservoirs,
5. Suction and discharge pressure at pumps,
6. Discharge pressure at wells,
7. Upstream and downstream pressure at pressure-reducing valves (PRV).

While much of the data could be collected by observing gages located on the tanks and pumps, gages for measuring the hydrant pressures and flows were needed at preselected points in the system. These gages were provided by WES and included a Pollard Hydrant Gage (P-670) with a 160-psi dial and a Pollard Hydrant Flow Gage (P-669) with a 1300-gpm dial. The tests were conducted by Mr. James Emerson, POD, Mr. Juan Soriano, PUAG, and Dr. Thomas Walski, WES, on 18-20 August 1981. The data collected are presented in the following sections.

Hydrant Tests

Table 3-1 contains data collected during the hydrant static and flow tests. For many hydrants only a static pressure reading was taken, while for others an adjacent hydrant was opened and a flow test was conducted as described in American Water Works Association (AWWA) Manual No. M17 (Installation, Field Testing and Maintenance of Fire Hydrants). Note that in previous Sanitary Surveys conducted for Agat-Santa Rita and Yigo areas, it appears that only one hydrant was used in conducting the flow test so that the pressure reported for the flowing condition is not the pressure during the flow test as defined in AWWA M17, but rather the velocity head at the mouth of the hydrant in pounds per square inch. Therefore, only the static pressures given in the Sanitary Surveys are correct.

The data contained in each column of Table 3-1 are described in greater detail below.

Column 1. The location is that of the hydrant at which the static pressure gage was located. The nearest hydrant to this hydrant is the one that was allowed to flow.

Column 2. The hydrants to be tested were selected partly based

Table 3-1

Results of Hydrant Tests

Location of Hydrant	Node No. in Model	Date of Test Aug 81	Elevation of Hydrant (ft)	Static Pressure (psi)	Static HGL (ft)	While Flowing Pressure (psi)	While Flowing HGL (ft)	Test Flow (gpm)	Predicted Flow @20 psi (gpm)
1. In Front of PUAG Bldg.	245*	18	160	43	259				
2. Marine Drive--In Front of McDonalds	268*	18	115	54	240				
3. Marine Drive--Across From Taco Bell	237	18	40	81	227				
4. Camp Watkins Rd--1/4 Mile From Marine Dr.	231*	18	25	84	219	75	198	790	2280
5. Route 4--1 Block From Route 1--Agana	203	18	10	95	229	90	218	1000	4320
6. Sinajana--Papato Lane--Just Off Route 4	211*	18	130	76	305				
7. West 10th St. & Route 1--End of Agana System	200	18	10	90	218	30	79	790	860
8. Piti Village--100 Yd. From Marine Dr.	275	18	10	88	213	82	199	820	3040
9. Old Agat--North End of System	314	18	10	54	135				
10. Hyundai--1/4 Mi. From Route 12	316	18	60	58	194	35	141	580	760
11. Agat--1/4 Mi. Below PRV	315*	18	60	62	203				
12. Route 2--Near Agat Cemetery	311	18	10	55	137	52	130	790	2980
13. At Connection With Navy (Fena) on Route 12	301*	18	260	40	352				

(Continued)

(Sheet 1 of 4)

Table 3-1 (Continued)

Location of Hydrant	Node No. in Model	Date of Test Aug 81	Elevation of Hydrant (ft)	Static Pressure (psi)	Static HGL (ft)	While Flowing Pressure (psi)	While Flowing HGL (ft)	Test Flow (gpm)	Predicted Flow @20 psi (gpm)
14. Route 2--Santa Ana Church	310	18	20	53	142	25	78	630	690
15. Umatac--By Magellen Mon.	402	19	5	44	103				
16. Umatac--In Front of Fire Station	405*	19	205	50	320				
17. Bile Bay--End of 8" Line from Merizo	408	19	10	33	86	18	51	440	410
18. Merizo--Route 4--100 Yd West of Road to Merizo School	409	19	10	35	90				
19. In Front of Merizo School	411*	19	250	30	319				
20. Route 4--Agfayan Bay Near Inarajan Church	420	19	20	115	285	46	126	790	940
21. Talofofo--Near C&F Mart	436	19	295	62	438	38	383	730	990
22. Entrance to Baza Gardens	449	19	300	65	450				
23. Route 4--Yona--Near Cruz Store	458	19	290	23	343	14	322	410	230
24. Route 4--50 Yd. North of Route 10	215	19	180	84	374				
25. Ordot--In Front of Washington Jr. High	212	19	125	100	356				

(Continued)

(Sheet 2 of 4)

Table 3-1 (Continued)

Location of Hydrant	Node No. in Model	Date of Test Aug 81	Elevation of Hydrant (ft)	Static Pressure (psi)	Static HGL (ft)	While Flowing Pressure (psi)	While Flowing HGL (ft)	Test Flow (gpm)	Predicted Flow @20 psi (gpm)
26. Dairy Rd. at Conga Road	279	19	110	135	422				
27. Mangilao--On Road to University 100 Yd. From Route 10	218*	19	220	70	382	65	370	470	1630
28. Camelia Lane--Latte Heights Between Mil Flores Rd. and Cadena del Amor Ln.	255*	19	410	35	491	23	463	240	270
29. Macheche Rd. at Chueto Rd.--Dededo	123	19	320	75	493				
30. Santa Monica Rd. Near Dededo Jr. High	115*	19	350	66	502				
31. W. Cebello Ct. Off Chalan Liguan--Liguan Terrace	122*	19	280	96	502	82	469	410	1020
32. In Front of 26 Calachuha St.--Barrigada Hgts.	262*	19	540	72	706	67	695	1010	3560
33. Route 10--In Front of Untalan Jr. High	266	19	200	85	396				
34. Route 10 at Leyan	224*	19	220	73	389				
35. In Front of 659 Chamacho Way--Barrigada	225*	19	200	81	387				
36. Duana St.--Mongmong-Toto-Maite	229	19	180	78	360	20	226	340	340
37. Paseo Antonio Near Dasco Ct.--Perez Acres	170*	20	430	95	649	65	580	710	1160

(Continued)

(Sheet 3 of 4)

Table 3-1 (Concluded)

Location of Hydrant	Node No. in Model	Date of Test Aug 81	Elevation of Hydrant (ft)	Static Pressure (psi)	Static HGL (ft)	While Flowing Pressure (psi)	While Flowing HGL (ft)	Test Flow (gpm)	Predicted Flow @20 psi (gpm)
38. Yigo Village--In Front of Church--200 Yd. North of Gayerno Rd.	161	20	460	78	640				
39. Agaga Ave.--Agafa Gumas	179	20	530	5	541				
40. Entrata St. and Apaca Ave.--Agafa Gumas	179	20	530	40	622				
41. Ysengsong Rd.--1/2 Mile North of Dededo	108	20	430	54	555	42	507	790	1390
42. Harmon Wastewater Treatment Plant	152	20	280	81	467				
43. Marine Drive at Tumon Loop Reservoir	249	20	190	90	398				
44. In Front of Guam Okura Hotel	248*	20	90	83	282	42	187	670	840
45. San Victores Rd. at Ypao Rd.	243	20	60	79	242				
46. Off San Victores Rd.--In Front of Houses Next to Guam Memorial Hospital	234*	20	130	50	245	15	164	240	220

(Sheet 4 of 4)

on their proximity to node points in the water distribution network model being developed by WES. The node number at which the hydrant is located is given in column 2. In some cases, the hydrant is a significant distance from the node. These node numbers are designated by an asterisk.

Column 3. The date on which the test was conducted is given in column 3. The number 18 indicates that it was conducted 18 August 1981.

Column 4. The elevation of the hydrant above mean sea level (msl) was obtained from USGS 1:24,000-scale topographic maps with 20-ft contour intervals. The data should only be considered accurate to ± 10 ft.

Column 5. The pressure (in pounds per square inch) recorded at the hydrant under normal flows is given in column 5. It is accurate to ± 5 psi.

Column 6. The elevation (in feet) of the hydraulic grade line (HGL) under normal flows is given in column 6. It is calculated using

$$HGL = E + 2.31\ P$$

where

HGL = height of hydraulic grade line, ft

E = elevation of hydrant, ft

P = pressure at hydrant, psi

Columns 7 and 8. Columns 7 and 8 contain the same information as given in columns 5 and 6, respectively, except that the entries are for the case in which the adjacent hydrant is flowing.

Column 9. Column 9 contains the flow from the adjacent hydrant rounded usually to the nearest 30 gpm.

Column 10. The predicted flow at 20 psi is the customary way of describing the flow that can be delivered through a pumper fire engine. It is determined from the following formula given by the National Board of Fire Underwriters:

$$Q_{20} = Q_T \left(\frac{P_S - 20}{P_S - P_T} \right)^{0.54}$$

where

Q_{20} = flow provided at 20 psi, gpm

Q_T = flow provided during test, gpm

P_S = static pressure reading, psi

P_T = pressure recorded during test, psi

Caution must be exercised in using some of the results in Table 3-1. For example, the accuracy of values for predicted fire flow at 20 psi depends on the relative size of $P_S - 20$ and $P_S - P_T$. If $P_S - 20$ is much greater than $P_S - P_T$ (e.g., a factor of 20), then the results will be less reliable than if P_T was approximately 20. This is due to the fact that opening the hydrant in these cases did not significantly change the pressure and, hence, did not closely simulate fire conditions. The results of test 5 (Route 4 Agana) will, therefore, not be as good an indicator as test 7 (End of Agana System).

Unusual results were found in running the hydrant test at some locations. These are described in detail below.

Location 12. Agat Cemetery--the flow at the hydrant varied from 440-1100 gpm during the test. The test was rerun and the flow stabilized near 790 gpm. The variation may have been due to the effect of the Agat pressure-reducing valve, or construction on a nearby water main. Results from this test were not used in calibration.

Location 28. During the test in the Latte Heights, the pressure did not return to the initial static pressure of 35 psi after the flow test but only to 28 psi. The value of 35 psi was used for calibration.

Locations 39 and 40. There was very little pressure in the Agafa Gumas area during the tests because the Agafa Gumas Tank was out of service. This, however, does not explain why the pressure in test 39 was almost nonexistent. It is very likely that there was a closed valve or blocked pipe near the hydrant. These values were also not used in the model calibration.

Reservoirs

The water elevation in every tank was checked immediately preceding or following the hydrant tests influenced by that tank. The results are shown in Table 3-2. In cases where the reservoir was remotely located or elevated, the water level reported that day by

Table 3-2

Water Elevation in Reservoirs

Location	Date Aug 81	Water Elevation ft	Node No. in Model
Observed			
Tumon Reservoir	18	36	240
Agana Heights Reservoir	18	38	206
Fena Clearwell	18	14	300
Umatac Tank	18-19	0	401
Merizo Reservoir	19	36	411
Windward Hills Large Reservoir	19	40	445
Chaot Reservoir	19	15	213
Mangilao Reservoir	19	40	220
Barrigada Reservoir	19	27	259
Yigo Reservoir	20	19	160
Reported by PUAG			
Piti Reservoir	18	37	276
Malojloj Reservoir	19	18	421
Barrigada Heights Reservoir	19	35	260
Yona Reservoir	19	14	462
Harmon Reservoir	20	12	150
Agafa Gumas Reservoir	20	0	100

PUAG was used. The Umatac Tank was empty due to a power outage in that part of the island, and the Agafa Gumas Tank was out of service.

Pumps and Wells

Discharge and suction head at most of the booster pumps and some of the wells are presented in Table 3-3. Numerous other wells were checked but no reading could be obtained since the faces on the pressure gages were not readable. The Yona Booster Pump Station was not included in Table 3-3 as it appeared that one of its gages was not reading correctly. While the pump was running, the difference between suction

Table 3-3

Pressure at Pumps and Wells

Location	Date Aug 81	Pressure Suction psi	Pressure Discharge psi	Node No. in Model
Agana Springs	19	-	45	270
Pigua	20	25	125	414
Malojloj	20	20	-	425*
Upper Brigade	20	-	70	452*
Lower Brigade	20	-	off	452*
Ylig Treatment Plant	20	-	235	454
Well A-7	20	-	105	214
Well A-18	20	-	120	222
Well A-2	20	-	134	214
Well A-14	20	-	78	222
Barrigada Heights	20	14	110	258*
Well D-16	20	-	82	116
Well D-18	20	-	90	116
Well M-14	20	-	105	122*
Well Y-3	21	-	118	170*
Well AG-1	21	-	70	124
Ysengsong	21	95	125	103
Well F-3	21	-	180	105
Well F-6	21	-	245	106
Well F-5	21	-	200	106
Well D-9	21	-	120	108

* Well or pump is a significant distance from the node.

and discharge pressure was 10 psi. This is inconsistent with the horsepower of the pump described in the Master Plan, and indicates that one of the gages was not working, or that the pump impeller was damaged.

At some of the pumps and wells, it was unclear whether the pressure was in pounds per square inch or feet because of the difficulty in reading the gage. Since most gages indicate pressure in pounds per square inch, in most cases it was concluded that the pressures were in pounds per square inch. This resulted in some inconsistencies between Table 3-3 of this report and Appendix D of the Master Plan. For example, the Master Plan reports pressure at well F-6 as 115 psi while the pressure gage read 245. These readings are only consistent if the 245 is the pressure in feet (i.e. 106 psi).

Pressure-Reducing Valves

Table 3-4 gives the pressures at the major pressure-reducing valves in the system. The area around the Agat pressure-reducing valve was so covered by vegetation that the valve could not be located.

Table 3-4

Pressure at Pressure-Reducing Valves

Location	Date Aug 81	Pressure Upstream psi	Pressure Downstream psi	Node No. in Model
Agat	18	Could not locate		325
Laelae Spring	18	-	80-85	404
Malojloj	19	-	100	427*
San Victores Road	20	65	60	250*

* Hydrant is a significant distance from the node.

4. Development and Calibration of Water Distribution System Model

This section contains a description of the steps used to develop and calibrate the water distribution system model. There were actually "models" for four separate subareas on the island (AB, C, D1, D2) as described in the previous section. These correspond to four separate data files for the MAPS computer program.

Procedure

Once the map of the distribution system was constructed, the layout of the system was coded in a form acceptable to the MAPS computer program as described in Appendix A. These data files were created and stored on the Boeing Computer Services (BCS) computer.

Next, water use was divided among the nodes. This information was stored in separate data files which were merged with the files describing the physical system at the time computer runs were made.

It must be remembered that the model is a "skeletal" model in that it does not include every pipe in the PUAG system, but only the major lines. Thus, most of the smaller neighborhood distribution lines have been omitted. Several parallel pipes may be represented by a single large pipe in the model. Similarly, withdrawals of water by users located in an area of several acres may be considered to occur together at a single node.

The model was considered calibrated when it was capable of predicting the elevation of the hydraulic grade line (i.e., pressure) at all nodes, for which calibration data were available under average flow and fire flow conditions. Noting that pressures are known to be approximately ±5 psi (12 ft) and elevations to ±10 ft, the model should be considered to be an accurate representation of the system if it predicts pressures to within 20 ft of those observed.

The first run of the program for a given area generally produced a very poor calibration. The first variables to be adjusted were the pressures at pumps and wells since the data associated with these appurtenances were often sketchy at best. Note that wells were generally not modeled separately but rather were grouped in "wellfields" which were assigned to nodes. The well data used for the program is given in Table 4-1. In service area AB, wellfield nodes have numbers in the 50's and are connected to the system by very short pipes.

Once the heads at tanks, pumps, and wells were established, the next parameters that required adjustment were the magnitudes and distribution of water use and hydraulic conductivities, as represented by the Hazen-Williams C-factor. In general, the flows were divided evenly among the nodes within a given part of a subvillage (e.g., Yona,

Table 4-1

Wellfield Pump Data

Wellfield Node	Wells	Total Capacity gpm	Head ft
501	A-1, 5, 6	701	190
502	A-2, 4, 7, 8	775	239
503	A-3, 11, 12	610	220
504	A-9, 10, 13	315	300
505	A-14, 18, 21	590	169
506	A-15	185	132
507	A-17	190	157
508	A-19	200	248
510	AG-1, 2	95	170
511	D-1, 2, 4, 5	1062	45
512	D-3	500	41
513	D-6, 7, 9, 10, 11	706	182
514	D-8, 12	337	189
515	D-13	94	174
516	D-14	165	138
517	D-15	158	228
518	F-3, 4	457	162
519	F-5, 6, 7	198	242
520	F-8	129	88
523	M-1, 2, 3, 4, 8, 9	987	139
524	M-5, 6, 7	540	122
525	M-12, 14	300	148
526	F-1	295	150

Asmisen, Baza Gardens, and Windward Hills are subvillages within the village of Yona).

A C-factor of 110 was used for all pipes. This was done since there was little need to further fine tune the model as it calibrated well with a single value for C. Since the model was of the skeletal type, the pipes in the model did not always correspond exactly to the existing distribution system. C-factor tests should be conducted on some of the major transmission lines in the PUAG system.

Pressure-reducing valves were modeled as a constant head node on the downstream side and a constant flow node on the upstream side, as described in Appendix A.

If pumps were not running during data collection for calibration (e.g., Lower Brigade), no flow was permitted during the calibration simulation. This was accomplished by "disconnecting" one end of the line on which the pump was located.

Results of Calibration

The results of the calibration runs are summarized in Table 4-2. Since it was difficult to determine the exact water use at the time the tests were run, the model was run for flow rates equal to the average water use and twice that amount. The pressures under both use rates are reported in Table 4-2. The pressure for average use is given as the first number in parentheses in the average flow column entitled "Predicted HGL" and the pressure at twice the average use is given as the second number in parentheses.

The predicted pressure at twice average flow is generally closer to the observed pressure since the tests were run during the daytime when water use was high, and the "average use" does not include unaccounted for water which may be carried by the distribution system. The detailed computer printouts for some runs are presented in Appendix C.

Each of the values in the predicted pressure under fire flow conditions column corresponds to a single run of the program at the given fire flow, while the remainder of the subarea is consuming water at twice the average flow rate.

There were a few nodes at which there were notable problems in the

Table 4-2

Results of Model Calibration

Location of Hydrant	Node No. in Model	Average Flow: Observed Pressure psi	Average Flow: Observed HGL ft	Average Flow: Predicted HGL ft	Fire Flow: Observed Pressure psi	Fire Flow: Observed HGL ft	Fire Flow: Predicted HGL ft	Test Flow gpm
1. In Front of PUAG Bldg.	245*	43	259					
2. Marine Drive--In Front of McDonalds	268*	54	240	(254,242)**				
3. Marine Drive--Across from Taco Bell	237	81	227	(236,229)				
4. Camp Watkins Rd--1/4 Mile from Marine Dr.	231*	84	219	(236,227)	75	198		790
5. Route 4--1 Block from Route 1--Agana	203	95	229	(235,223)	90	218	205	1000
6. Sinajana--Papato Lane--Just Off Route 4	211* 209	76	305	(345,337) (271,264)				
7. West 10th St. & Route 1--End of Agana System	200	90	218	(234,221)	30	79	16	790

(Continued)

* Hydrant is a significant distance from the node.
** (HGL low flow, HGL high flow.)

(Sheet 1 of 6)

Table 4-2 (Continued)

Location of Hydrant	Node No. in Model	Average Flow Observed Pressure psi	Average Flow Observed HGL ft	Average Flow Predicted HGL ft	Fire Flow Observed Pressure psi	Fire Flow Observed HGL ft	Fire Flow Predicted HGL ft	Test Flow gpm
8. Piti Village--100 Yd from Marine Dr.	275	88	213		82	199		820
9. Old Agat--North End of System	314	54	135 (266)	(285,270)				
10. Hyundai--1/4 Mi from Route 12	316	58	194 (283)	(285,274)	35	141	173	580
11. Agat--1/4 Mi Below PRV	315*	62	203 (218)	(204,203)				
12. Route 2--Near Agat Cemetery	311	55	137 (204)	(203,202)	52	130	178	790
13. At Connection with Navy (Fena) on Route 12	301*	40	352	(353,349)				
14. Route 2--Santa Ana Church	310	53	142 194	(203,201)	25	78	160	630
15. Umatac--By Magellen Mon.	402	44	103	(113,92)				
16. Umatac--In Front of Fire Station	405*	50	320	(327,312)				

(Continued)

* Hydrant is a significant distance from the node.

Table 4-2 (Continued)

Location of Hydrant	Node No. in Model	Average Flow Observed Pressure psi	Average Flow Observed HGL ft	Average Flow Predicted HGL ft	Fire Flow Observed Pressure psi	Fire Flow Observed HGL ft	Fire Flow Predicted HGL ft	Test Flow gpm
17. Bile Bay--End of 8" Line from Merizo	408	33	86	(91,91)	18	51	54	440
18. Merizo--Route 4--100 Yd West of Road to Merizo School	409	35	90	(91,90)				
19. In Front of Merizo School	411*	30	319	(319,319)				
20. Route 4--Agfayan Bay Near Inarajan Church	420	115	285	(299,287)	46	126	66	790
21. Talofofo--Near C&F Mart	436	62	438	(438,428)	38	483	336 388	730 500
22. Entrance to Baza Gardens	449	65	450	(448,443)				
23. Route 4--Yona--Near Cruz Store	458	23	343	(348,343)	14	322	332	410
24. Route 4--50 Yd North of Route 10	215	84	374	(405,380)				
25. Ordot--In Front of Washington Jr. High	212	100	356	(396,380)				

(Continued)

* Hydrant is a significant distance from the node.

Table 4-2 (Continued)

Location of Hydrant	Node No. in Model	Average Flow: Observed Pressure psi	Average Flow: Observed HGL ft	Average Flow: Predicted HGL ft	Fire Flow: Observed Pressure psi	Fire Flow: Observed HGL ft	Fire Flow: Predicted HGL ft	Test Flow gpm
26. Dairy Rd. at Conga Road	279	135	422	(413,392)				
27. Mangilao--On Road to University 100 Yd from Route 10	218*	70	382	(412,380)	65	370	380	470
28. Camelia Lane--Latte Heights Between Mil Flores Rd. and Cadena del Amor Ln.	255*	35	491	(500,502)	23	463		240
29. Macheche Rd. at Chueto Rd.--Dededo	123	75	493	(494,474)				
30. Santa Monica Rd. Near Dededo Jr. High	115*	66	502	(513,483)				
31. W. Cebello Ct. Off Chalan Liguan--Liguan Terrace	122*	96	502	(490,459)	82	469	441	410
32. In Front of 26 Calachuha St.--Barrigada Hgts.	262*	72	706		67	695		1010

(Continued)

* Hydrant is a significant distance from the node.

(Sheet 4 of 6)

Table 4-2 (Continued)

Location of Hydrant	Node No. in Model	Average Flow: Observed Pressure psi	Average Flow: Observed HGL ft	Average Flow: Predicted HGL ft	Fire Flow: Observed Pressure psi	Fire Flow: Observed HGL ft	Fire Flow: Predicted HGL ft	Test Flow gpm
33. Route 10--In Front of Untalan Jr. High	266	85	396	(396,384)				
34. Route 10 at Leyan	224*	73	389	(399,386)				
35. In Front of 659 Chamacho Way--Barrigada	225*	81	387	(397,381)				
36. Duana St.--Mongmong-Toto-Maite	229	78	360	(395,374)	20	226		340
37. Paseo Antonio Near Dasco Ct.--Perez Acres	170*	95	649		65	580		710
38. Yigo Village--In Front of Church--200 Yd North of Gayerno Rd.	161	78	640					
39. Agaga Ave.--Agafa Gumas	174	5	541					
40. Entrata St. and Apaca Ave.--Agafa Gumas	174	40	622	(643,630)				

(Continued)

* Hydrant is a significant distance from the node.

(Sheet 5 of 6)

Table 4-2 (Concluded)

Location of Hydrant	Node No. in Model	Average Flow: Observed Pressure psi	Average Flow: Observed HGL ft	Average Flow: Predicted HGL ft	Fire Flow: Observed Pressure psi	Fire Flow: Observed HGL ft	Fire Flow: Predicted HGL ft	Test Flow gpm
41. Ysengsong Rd.--1/2 Mile North of Dededo	108	54	555	(587,561)	42	527	532	790
42. Harmon Wastewater Treatment Plant	152	81	467					
43. Marine Drive at Tumon Loop Reservoir	249	90	398	(470,384)				
44. In Front of Guam Okura Hotel	248*	83	282	(295,294)	42	187		670
45. San Victores Rd. at Ypao Rd.	243	79	242	(260,248)				
46. Off San Victores Rd.--In Front of Houses Next to Guam Hospital	234*	50	245	(252,238)	15	164		240

* Hydrant is a significant distance from the node.

(Sheet 6 of 6)

calibration. These are discussed below.

The location of the pressure test conducted at Sinajana was a significant distance from either of the nearby nodes (nodes 209 and 211). Therefore, the predicted pressure at both nodes is given.

The predicted pressure at node 200 (south end of Agana) during fire flow is significantly lower than that observed. This could be corrected by slightly increasing the C-factor for some of the lines leading to node 200.

The data collected in the Agat-Santa Rita area during the August 1981 field trip were inconsistent with the pressure readings reported in the Agat-Santa Rita Sanitary Survey. It was decided that the data set that most closely reflected "typical" operations of the system should be used. During the August 1981 tests, the pressure was observed to fluctuate during tests, and there were inconsistencies in the data (e.g., HGL dropped by 66 ft in 2500 ft between node 315 (Juan Guererro Ave.) and 311 (near Agat Cemetery) in Agat). This indicates that there may have been some closed valves in Agat in order to accommodate nearby water main construction works. For this reason, the values for static pressure from the Agat-Santa Rita Sanitary Survey were used for calibration and are shown in parentheses below the observed pressures.

The fire flow pressures reported in Agat-Santa Rita Sanitary Survey cannot be used because the "pressure" reported was actually the velocity head at the flowing hydrant. In conducting a hydrant flow test, the "residual" hydrant (where pressure is measured) should not be the same as the "test" (flowing) hydrant (AWWA Manual 17). In the Agat-Santa Rita Study, the AWWA procedure was not used and the pressure was read at the flowing hydrant. This could result in significant head losses in the hydrant, especially if the hydrant valve is not completely open. Because of this problem, it was not possible to calibrate the pressure in Agat-Santa Rita for fire flow conditions.

In Hyundai, it was found that the pressure was controlled by Santa Rita Springs and not the Navy Mag Pumping Station source.

There are essentially two pressure zones in subarea C. They are separated by the Agat PRV. In order to simulate the two areas in a single model run, it was necessary to simulate the PRV connecting them with an "imaginary" pipe with very low flow. This imaginary pipe connecting node 300 and 326 must be included even though no such pipe actually exists. This was necessary since a PRV operates in an unsteady manner, but the model is a steady-state model.

In modeling the hydraulics of Umatac Village, the sources for the village (LaeLae Spring, Atlague Spring) were taken as a single node (404) and considered to produce an HGL of 130 ft.

The Merizo PRV was set to a pressure of 37 psi, although data from PUAG showed it had a downstream pressure of 30 psi. Similarly, the Malogloj-Inarajan PRV was set to 100 psi in the model (as observed in the field), although a letter from PUAG stated it was set at 80 psi and the Master Plan stated it was set at 25 psi.

The capacity of the booster pump at Umatac was set to 30 gpm at a head of 235 ft, although PUAG data showed it had a capacity of only 15 gpm. Data from PUAG also showed the Inarajan package pumping station to produce 160 psi, although this resulted in extremely high pressures near the Inarajan school (node 426). There were no data to confirm this pressure.

In Talofofo it was impossible to accurately calibrate the model for the fire flow condition. The most likely explanation was that the fire flow recorded as 730 gpm was actually 530 gpm. This is the flow required to give the correct pressure. Furthermore, there is a 530 mark on the pressure gage, but no 730; so the number may well have been recorded incorrectly.

In the Agafa Gumas and Ysengsong Road areas the predicted HGL is higher than the observed HGL. This is most likely due to combining several well pumps into a single wellfield node with a single pump curve. This approximation slightly underestimates the head losses between the well and the distribution mains. The calibration is considerably better for nodes nearer to tanks than wells.

Because subarea AB is so large, and the solution to a Hardy-Cross problem is not an exact solution, the pressure reported for nodes well away from the datum node will have a larger error than from nodes near the datum node. For the calibration runs, both the Tumon and Mangilao tanks were used as the datum on individual runs. Since the most critical nodes (i.e., most users) are in the Agana-Tumon area, the Tumon tank (node 248) was used as the datum for the runs shown in Table 4-2. Runs made using the Mangilao tank as datum were more accurate in the Mangilao area.

Summary of Calibration

The results of the calibration indicate that the model can correctly predict pressure and flow in the PUAG distribution system. While the model is adequately calibrated, there is margin for improvement by "fine tuning" the C-factors and assigning water users to nodes. Future users of the model are encouraged to perform this fine tuning, as well as to update the model to account for improvements to the system.

5. Predicted System Behavior Under Future Conditions

The purpose of developing the water distribution system model was not to simulate existing conditions, but rather to project the behavior of the system under many different conditions. Once the model was calibrated, it was run for different subareas for a variety of flows.

The most important runs were for average flow in the year 2000 and for peak flow in the year 2030, which corresponds to 4.5 times the average flow in 2000. Numerous other runs were made to investigate the existing system under alternative conditions in order to identify weak points in the system.

The results of these simulation runs are presented in the following sections. The hydraulics of areas of the island, which were not covered by the model, are also discussed briefly. Unless otherwise stated, the comments below refer to the existing system under current water use.

Subarea AB

Dededo. As long as the wells in the Dededo area are operating, pressures will be adequate in Dededo. If the wells are not pumping, the area is served primarily by the Barrigada Reservoir. The reservoir alone can meet average demands, but because of the distance from Dededo (approximately 2 miles), pressure will be very low during peak use or fire flow conditions.

Tumon-Tamuning. The Tumon-Tamuning area is one of the few areas with no sources. It receives its water primarily from the wells of the Dededo area. The pressure is controlled by the Tumon Reservoir and is adequate under normal conditions. Under high flow or fire flow, too much head loss occurs in the pipes to provide the required pressures. There is a valve between the Tumon Reservoir and Tamuning, which is described as normally closed (N.C.) in the Master Plan. If this valve is opened, the pressures in Tamuning during high flow period can be greatly improved. Replacing this valve with a pressure-reducing valve would serve this purpose well and would also serve to protect the system during low flow periods. The Tumon Loop Reservoir has not yet been connected to the system. When it is, it should improve the fire flow in the Tumon Bay area, since presently fire flow to this area must travel from Dededo or the Tumon Reservoir, and, either way, head losses are high.

Latte Heights. Latte Heights, which is located at 400 ft msl, is served, like Dededo, by the Barrigada Reservoir. It has adequate pressure during average and low flow periods and when the pumps at the M-series wells are operating. The proposed additional booster pump on the line from the Barrigada Reservoir should improve pressure in the Latte Heights area.

Mangilao-Barrigada-Chalan Pago-Ordot. The Mangilao-Barrigada-Chalan Pago-Ordot areas are served by the A-series wells. Pressure is further controlled by the Chaot and Mangilao Reservoirs. As long as the wells are pumping, pressure will be adequate. If the wells are shut off, pressure can be a problem at high flow in the Barrigada area since some of the nodes are several miles from the Mangilao Reservoir. One solution to this problem would be to connect Barrigada with Barrigada Heights by way of Security Road. A pressure-reducing-sustaining valve, set to open only during high flow periods, and approximately 2 miles of pipe would be required for this.

Mongmong-Toto-Maite. At present, Mongmong-Toto-Maite is served primarily by Navy sources. The proposed Barrigada Reservoir should result in adequate pressures in the area. A high priority should be placed on conducting the Sanitary Survey of Mongmong-Toto-Maite as recommended in the Master Plan.

Agana Heights-Sinajana. The Agana Heights-Sinajana area receives its water from the A-series wells. Pressure is controlled by the Agana Heights reservoir. The reservoir is not much higher than the Agana Heights community so the pressure will be low in that immediate area. During average flow, the pressure can be raised by wells and the Chaot Reservoir, but during high flow the pressure cannot be sustained because of the distance to that reservoir. Sinajana is lower and nearer the Chaot Reservoir, so it will have adequate pressure, even at high flow.

Agana. The Agana Area receives water from Agana Heights and Tamuning and also has an emergency connection to a Navy line. Because of the low elevation, the pressure is adequate during average conditions, but it is difficult to supply fire flows of about 1000 gpm at the east extremities of Agana where the system is essentially a dead-end line (6 in. and 8 in.). Since there are commercial buildings in the area, high flows for fire fighting are required. This situation should be corrected when the proposed 18-in. and 20-in. line along the coast is constructed.

Subarea C

Subarea C is at present isolated from the remainder of the PUAG distribution system. It receives water primarily from the Fena Water Treatment Plant, plus Santa Rita Springs and the Navy Mag Booster Pump. The pressures are generally adequate in the subarea during average conditions and the new line being installed along the coast should alleviate the problem of achieving high flows in Old Agat. The 2-in. section of pipe between the Navy Mag Booster Pump and Hyundai should be replaced by a larger line and a pressure-reducing valve. At present, the 2-in. line is preventing the area from receiving high flows from the Navy system that are needed under fire-fighting conditions.

Subareas D1 and D2

Service area D receives most of its water from local sources, although some water enters from service area B to the north. This area is divided into two subareas (D1 and D2) by the booster pump and pressure-reducing valve in Malojloj.

Yona. The areas downstream of the booster pump station generally have adequate pressure. However, in the hills to the west of Yona there is inadequate pressure for fire fighting. The proposed reservoir in the hills should correct this problem. The pumping station being constructed near the Pago River should raise pressure in the remainder of the area.

Baza Gardens. Baza Gardens has adequate pressure for both average and high flows since it is downstream of the Brigade Pump Station and is at a much lower elevation than the Windward Hills Reservoirs.

Windward Hills. The Windward Hills golf course, landing strip, and memorial park have adequate pressure for average conditions, but, because they are at an elevation comparable to that of the Windward Hill Reservoir, low pressures will exist at flows above 500 gpm. If the pipeline and pump station along Cross Island Road are installed, care must be exercised to ensure that adequate pressure can be maintained at the suction end of the pump. The pump station should be located at the intersection of Route 17 and 4A, and not farther up Route 17 as shown in the Master Plan.

Talofofo. The distribution system in Talofofo is fed from the Windward Hills Reservoirs. The main lines in Talofofo provide adequate pressure for average use and fire flows of 500 gpm.

Malojloj. Malojloj has adequate pressure because of the Malojloj Reservoir and Booster Pumping Station. The primary problem is that the Booster Pumping Station is located at an elevation of 250 ft, rather than being located in the Talofofo River valley. This means that very low pressures can develop at the suction end of the pump. This can result in cavitation and possible contamination if there are leaks in the pipe. The pumps should be moved to an elevation just above the Talofofo River floodplain.

Inarajan. Inarajan receives its water from the north by way of Malojloj. The pressure is regulated by a pressure-reducing valve on the 8-in. line from Malojloj. The area around Inarajan High School requires a separate booster pump station to provide flow to the higher elevations.

Merizo. Merizo takes its water from the Geus River, Siligen Spring, and the northern part of the island via Inarajan. The water is pumped from the Pigua Booster Pump Station to the Merizo Reservoir, which serves Merizo. The low-lying areas of Merizo receive water through a pressure-reducing valve. There is a problem in maintaining adequate pressure at the suction end of the Pigua booster pumps. This can be eliminated by installing a booster pump (possibly one from Pigua) between Inarajan and Merizo. When operating, it can serve the lower portion of Merizo and maintain positive suction pressure at Pigua. This will eliminate the wasteful practice of pumping water from the Pigua Booster Pump Station into a pressure-reducing valve.

Umatac. Umatac is served from Laelae Spring and La Sa Fua River. The distribution lines are barely adequate for high flow conditions and cannot provide fire flow. Major improvements in this area, as identified in the Master Plan, are required.

Other Areas

The following areas are either not connected to the other subareas, or are connected only at a single point, such as a booster pump. Therefore, it is easier to analyze them separately, rather than with large MAPS simulation runs. These areas are discussed individually below.

Yigo. Even though Yigo is considered part of service area A, it is virtually a separate system at present. The Yigo system provides adequate pressure at average and fire flows for users along Route 1. The pressures are somewhat lower in the area along Route 15 because of the higher elevations. The Anderson Elementary School is connected through a valve that is normally closed and receives flow from the Air Force, as does Mt. Santa Rosa. Fire demands cannot be met in this area because of the elevation. The 2-in. lines should be replaced by 6-in. lines and the area should be connected to the Yigo system through a new booster pump station. This area should be modeled using MAPS once the new construction is completed and calibration data obtained.

Harmon. The Harmon system is separate at present, but could be connected to the Dededo area near Wettengal Junction. The Harmon Tank is at too low an elevation and should be abandoned, raised, or replaced if Harmon is connected to Dededo.

Barrigada Heights. Barrigada Heights is connected to the Barrigada Reservoir through Barrigada Booster Pump Station. Because of its high elevation (reservoir at 705 ft) and large mains, there are no hydraulic problems in the area in the foreseeable future and Barrigada Height could provide backup fire flow to Barrigada and vicinity through a pressure-reducing-sustaining valve.

Asan-Piti-Nimitz Hill. Asan and Piti are served from Asan Spring and can be supplemented by a connection to the Navy. Adequate pressure exists in this area for average flow and fire-fighting conditions. Connecting this area to service area B would improve reliability and provide water to the Nimitz Hill area located above Asan-Piti, which is currently served by the Navy. Connecting Nimitz Hill, Nimitz Hill Estates, and other residential areas to the PUAG system will require construction of one or more booster pumping stations. The Master Plan shows two booster pumping stations along Spruance Drive. It may be less expensive to install one station with a pipeline from Asan, cross country to the reservoir location on Nimitz Hill, and a pressure-reducing-sustaining valve between Nimitz Hill Estates and Piti.

Sinifa-Talisay. Sinifa and Talisay are located on Cross Island Road above subarea C. This area receives water from the Navy through the Apra Heights Booster Pump Station and stores it in a reservoir at an elevation of 550 ft. There is very little development currently. Pressures are adequate for average flow conditions, but fire flows cannot be delivered because of the small size of the mains (2 in.). If areas C and D are connected, this area will be served by the line from Windward Hills to Santa Rosa. Under these conditions, it will be possible to provide fire protection and additional development can take place.

Review of Master Plan

The distribution system proposed in the Master Plan was reviewed and found to be an acceptable plan given that: (1) the PUAG should no longer rely on the military for supply and (2) all additional demands could be met from the northern groundwater lens. While some minor difficulties in the plan are pointed out in the preceding sections, the recommended improvements are generally hydraulically sound.

If the first assumption is invalid, and the Navy sources can be used indefinitely, there is little need for the large lines connecting Asan, Piti, Nimitz Hill, Agat, Santa Rosa, and Santa Rita to the remainder of the PUAG system. Elimination of these lines can result in significant savings in transmission and storage facility costs and will eliminate the need for some wells on the northern groundwater lens.

Since the Navy water is not taken from the groundwater lens, some of the stress on that aquifer will be relieved.

If the first assumption holds (i.e., PUAG is disconnected from the Navy) and the groundwater lens is not to be exploited, the Ugum River project or another project in southern Guam becomes attractive. This arrangement will require a significantly different distribution system with water flowing from south to north.

Future Use of Distribution Model

The results presented in this report show only a few of the cases that the water distribution model can simulate. If properly utilized by the PUAG or a contractor, this model can become a powerful management tool. For example, it can be used to:

1. Test the effect of installing new pipes, tanks, valves, or pumps;
2. Test the effect of shutting off several pumps or wells due to power failure or well contamination;
3. Test the effect of eliminating connections with the Navy.

The model users should construct separate data files (or card decks) representing the distribution system at present and various proposed systems for several time windows. In this way, the user can have an accurate understanding of the impact of each modification. It is also very easy to run the program for various water use rates or simulated fire needs.

With this model, the PUAG has been given substantially increased capability in managing the water distribution system. It is up to the PUAG to make maximum use of time capability to efficiently improve the system.

APPENDIX A: USER'S GUIDE

This appendix consists of the User's Guide for the MAPS Water Distribution Program (MAPDIST). It is Chapter 17 of Part 1 of the MAPS Manual (EM 1110-2-502) and, as such, the paragraph and figure numbers have the prefix "17."

* CHAPTER 17

WATER DISTRIBUTION SYSTEM ANALYSIS

17-1. Introduction. The MAPS Water Distribution System Analysis module calculates the velocity, flows, head losses, and pressures in each link and node of a water distribution system given the head at each tank, pressure at each pump, elevation at each node, diameter and length of each line, and water use. The program works for looped and branched networks and there is no need for the user to identify loops in the network. The program can be run as a stand-alone program or as part of MAPS. If run as part of MAPS, the user is limited to 350 nodes and a line of input is limited to 36 characters. Both methods are discussed in this chapter. The program does not automatically handle pressure reducing valves, but there are methods to account for their influence.

17-2. Input. Data for the distribution system analysis are read by the module from a data file. For the stand-alone program, this data file is built using the system editor. When the module is run as part of MAPS, the data file is built within the program using the commands given in paragraph 17-3. The MAPS keywords that are used for the water distribution program are listed in Table 17-1 and are described below.

a. Job. The JOB card provides the computer with the title of the job. It is printed at the top of every page of output.

b. Line. The format of the PIPE or LINE card used to describe every pipe to the program is given below.

Card Type	Node	Node	Diameter (inches)	Length (ft)	Optional
PIPE LINE	1084	2976	6.0	3756.0	Hazen Williams C if different from standard 120.

The order of data on the card is the node numbers at the ends of the pipe, the diameter of the pipe, and the length of the pipe. Optionally the Hazen Williams C may be specified if it is different from that specified on the COEF card (described later).

c. Node Elevations. Node numbers may be assigned in any order from 1 to 9999. Output of node data will be in the order of the node input

	Node Number	Elevation
ELEVATION	515	867.6

This card provides the ground elevation of the nodes of the system. Elevation is given in feet.

Table 17-1. Keywords for Water Distribution

```
JOB                          XXXXXX
LINE  FROM  XX.X  TO  XX.X  DIAMETER = XX.X IN LENGTH = XX.X FT  C=XX.X
PIPE  FROM  XX.X  TO  XX.X  DIAMETER = XX.X IN LENGTH = XX.X FT  C=XX.X
ELEVATION  OF  NODE  XX.X  IS  XX.X  FT
PUMP  AT  NODE  XX.X  PROVIDES  XX.X  PSI
TANK  AT  NODE  XX.X  IS  XX.X  FT  TO  WATER  LEVEL
OUTPUT  FROM  NODE  XX.X  IS  XX.X  GPM
INPUT  TO  NODE  XX.X  IS  XX.X  GPM
COEFFICIENT  C=XX.X
ACCURACY  XX.X  ITERATIONS  OR  XX.X  GPM
PRV  FROM  NODE  XX.X  TO  XX.X  SET  AT  XX.X  PSI
CHECK  VALVE  FROM  NODE  XX.X  TO  XX.X
BOOSTER  PUMP  FROM  NODE  XX.X  TO  XX.X  FOR  XX.X  GPM
LOOP  TABLES  PRINTED
RATIO  XX.X  OF  FLOW  TO  PREVIOUS  OUTPUT  FLOW
ERROR  OF  EACH  ITERATION  PRINTED
NO  ERROR  PRINTOUT
DATUM  NODE  XX.X
APUMP  NODE  XX.X  HEAD  XX.X  XX.X  XX.X  FT  FLOW  XX.X  XX.X  GPM
BPUMP  NODE  XX.X  HEAD  XX.X  FT  FLOW  XX.X  GPM
XBOOSTER  FROM  NODE  XX.X  TO  XX.X  HEAD  XX.X  FT  FLOW  XX.X  GPM
EXECUTE
END  OF  PROBLEM
```

d. Constant Head Nodes. PUMP and TANK cards specify constant head points. PUMP cards allow this specification in psi while TANK cards allow this specification in feet of head. Examples are:

	Node Number	Constant head in feet of water
TANK	3726	100

	Node Number	Constant head in psi
PUMP	3726	43.3

The two cards shown above would produce identical results. See paragraph 17-7 for a more detailed disc ssion of how the program considers pumps.

e. Input and Output. INPUT cards specify a point of supply of a constant amount of water at a variab.e pressure.

	Node Number	Input in gpm
INPUT	317	525

OUTPUT cards specify a constant output of water under variable pressure.

	Node Number	Demand in gpm
OUTPUT	715	535.0

f. Coefficient. The coefficient card enables the user to specify a value of Hazen Williams C, different from the default value of 120. The value is used for all pipes for which C is not given on the PIPE or LINE card. The format is

COEFFICIENT 110.

The above card specifies the Hazen Williams C to be used is 110 if not specified optionally on the PIPE or LINE card.

g. Execute. The EXECUTE card tells the program that data input is complete. This card says that the system has been completely described and that the analysis of the system may proceed. The data cards may be presented to the computer in any order, with the exception of the EXEC card, which must be the last card of the data deck before a run starts.

h. Convergence Criteria. The network problem is solved using the Hardy-Cross method. The flows in each loop are corrected by ΔQ at each iteration where

$$\Delta Q = \frac{\sum h\, Q^{1.85}}{1.85 \sum h\, Q^{0.85}} \qquad (17\text{-}1)$$

where

Q = flow, gpm

h = friction factor

(See documentation for more details on solution method.) The program stops when the maximum number of iterations (NOITER) is reached or the largest value of ΔQ is less than a critical tolerance (ACCU). The default values for NOITER and ACCU are 50. and 0.1 gpm. The iterations cease when either of these limits is reached. The user can change the default values by using the ACCURACY card

	Number of Iterations	Accuracy (gpm)
ACCURACY	100.	0.01

The above line decreases the error tolerance to 0.01 gpm and the maximum number of iterations to 100. Increasing the number of iterations or decreasing the tolerance increases the accuracy of the solution and the run cost. Decreasing the number of iterations or increasing the tolerance has the opposite effect.

i. Terminating Run. Once the solution is output, the user can change the inputs and outputs for the network using the INPUT and OUTPUT cards as before and rerun the program using the EXEC command. To stop the program, the user must enter END. The program will also stop when it reaches an "end-of-file" from the input file.

j. Valves. The user can specify the existence of a check valve or pressure reducing valve (PRV) by giving the nodes (in direction of flow) between which the valves are located. In the case of the pressure reducing valve; the user must also specify the pressure (in psi) to be maintained on the downstream end of the PRV. Examples are

	Nodes
CHECK	101 102

permits flows only from 101 to 102, nodes and

	Nodes	Pressure psi
PRV	200 300	50

permits flow from 200 to 300 only and pressure at the 200 beginning end of line cannot exceed 50 psi. Valves are discussed in more detail in paragraph 17-6.

k. Pumps. Pumps which pump into the system (as opposed to in-line booster pumps) can be represented not only using the INPUT or PUMP cards, which model the pump as a constant flow or constant head node, but also by the APUMP or BPUMP card, which simulate the fact that a pump operates at a point on a pump head curve. In the case of the APUMP card, three points from the pump curve are used to represent the pump, while for BPUMP, only one point is used. Given the pump curve in Figure 17-1, the APUMP and BPUMP cards at node 20 are

	Node	Heads (ft)	Flow (gpm)
APUMP	20	250 212.5 100	100 200
BPUMP	20	200	115.5

When an APUMP or BPUMP card is used, there must only be one pipe from the node at which the pump is located. More details on pumps are given in paragraph 17-7. Note that on the APUMP card, the first head is the head when flow is zero. A node with an APUMP or BPUMP must be connected to the network through one and only one line.

l. Booster Pumps. In-line booster pumps can be simulated in two ways. Either a BOOSTER card can be used which forces a given flow to pass between two nodes with the head calculated by the program, or a XBOOSTER card can be used which forces the flow and head at a booster pump to fall on the pump head curve. Unlike the LINE or PIPE cards, the order in which the from and to nodes are specified on the booster cards is critical. Examples are

	From Node	To Node	Flow (gpm)	
BOOSTER	10	11	200	
			Head (ft)	Flow (gpm)
XBOOSTER	105	106	150	300

See paragraph 17-7 for additional information. For the BOOSTER card, node 10 and 11 cannot be connected by a line card and nodes 10 and 11 must not be a constant head or INPUT or OUTPUT nodes. The elevation of node 10 and 11 must be the same. For an XBOOSTER card, node 105 and 106 must be connected by a line.

m. Datum. The program selects the constant head node with the highest hydraulic grade line elevation to be the datum node from which the loop tables are established. In some cases the user may wish to select another, more centrally located, node as the datum. In this case the user would select a TANK or PUMP node and call it the datum

	Node	Head
TANK	115	50
DATUM	115	

17-3. Rerunning Program. With the earlier version of the program, it was possible to run the program several times using a single data file, and changing the input and output flows between runs. Now it is possible in MAPDIST to change virtually every parameter as long as the network remains the same (i.e. lines not removed, node elevations not changed, booster pumps not changed). In addition to enabling the user to make several runs with a single data file, these changes reduce the number of iterations required for the solution to converge since the program uses the previous solution as a starting point for the reruns. To rerun the program, the user merely inserts cards

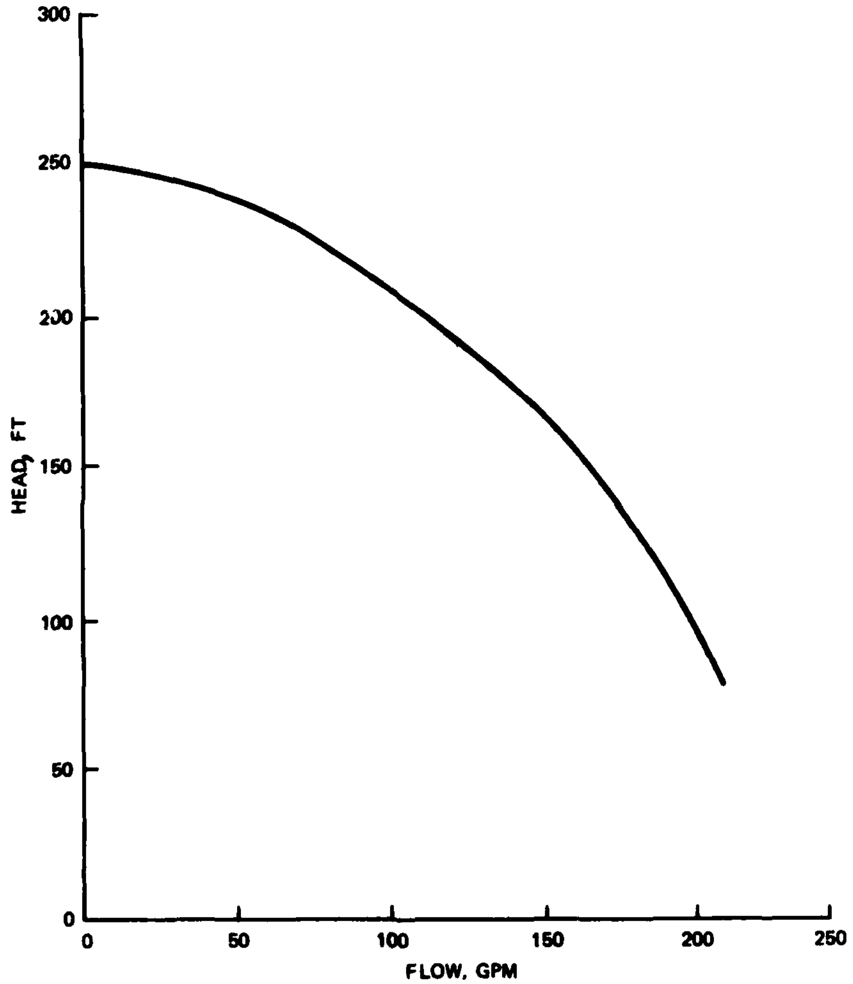


Figure 17-1. Typical Pump Head Characteristic Curve

to be changed after the EXECUTE card of the first run. The data for the rerun must be followed by an EXECUTE card. There is no limit to the number of reruns that can be made. A typical data file is shown below.

```
(Data for the first run)
     :
EXECUTE
(Data changed for first rerun)
     :
EXECUTE
(Data changed for second rerun)
     :
EXECUTE
(Data changed for the n-th rerun)
     :
EXECUTE
END
```

This type of file setup is shown in example problem 1 (page 17-22).

a. LINE or PIPE Card. A pipe cannot be added or deleted from the network, but the diameter, length, or Hazen-Williams coefficient (C) can be changed. This allows the user to try several different pipe sizes. While the user cannot remove a line for a rerun, it is possible to virtually eliminate the flow from the line by setting the diameter or C coefficient to a small value (e.g. diameter=0.1 or C=1).

b. PUMP or TANK. The pressure provided by a pump or the elevation of a tank can be changed for a rerun. Pumps or tanks cannot be added or deleted, but by setting the head to zero, the same effect can be achieved.

c. INPUT or OUTPUT. The input and output flows to and from a node can be changed on a rerun. This is especially helpful in simulating fire flows or the effect of future development on flows and pressure.

d. RATIO. The RATIO card can be used to adjust the water use at all output nodes in a network. It is useful for simulating the effect of conservation or the heads during peak use or low use times without having to enter data for each output node. For example, to reduce water use by 20% due to conservation (i.e. 0.8 of the original flow), the user would enter

```
RATIO 0.8
```

To simulate a peak use period in which flow is 2 times the average flows input (except for node 105 in which the flow is 150 gpm), the user would enter

```
RATIO 2.
OUTPUT 105 150
```

The location of the RATIO card in the input is important as any OUTPUT cards after the RATIO card will not be changed. For example, if the order of the two cards above is reversed, the output at node 105 would be 300 gpm (i.e. 2x150).

e. COEFFICIENT. It is possible to change the Hazen-Williams C coefficient for a rerun. This makes it easy to perform a sensitivity analysis on the effect of C. Changing C for a rerun using the COEFFICIENT card will not override the C values specified on LINE or PIPE cards.

f. PRV and CHECK. The setting of a pressure reducing valve can be changed for a rerun. While the PRV cannot be removed, the same effect can be achieved by changing the pressure setting to a large number. Similarly check valves can be added but cannot be deleted.

g. ACCU. The convergence criteria on the ACCURACY card can be changed. Both the maximum number of iterations and maximum ΔQ should be specified. If the max ΔQ is omitted the program will run the maximum number of iterations. With the ACCU card, the user can look at the initial solution, stop the program after 1, 10, r 20 iterations and then allow the program to run to completion to check the speed with which the solution converges.

h. ERROR. It is possible to switch the printing of the largest loop correction factor on or off by using the ERROR or NO ERROR card in a rerun.

i. Pump Curves. It is possible to change the coefficients of the pump head curves for a rerun. In the case of an in-line booster (XBOOST) it is even possible to add a booster pump, provided that the line on which it is added already is part of the network.

j. JOB. The JOB card can be used to change the title in a rerun.

k. Other Cards. ELEVATION, DATUM, and BOOSTER cards cannot be changed for reruns. Similarly LOOP TABLES cannot be printed for reruns as they would be the same as for the initial run.

17-4. Building Data File. The water distribution program reads its data from a file. The stand-alone version, MAPDIST, reads data from a file built using the computer system editor (CMEDIT in the case of BCS). In the case of the version contained in MAPS, the data file can be built using MAPS. If the user wishes to build the system data file using MAPS, he can enter the distribution analysis portion of the program by entering

DISTRIBUTION

in response to an 'INPUT MAPS COMMAND' prompt. The program responds with the prompt

READ, EDIT, RUN OR END?

a. Building File. To build a data file, the user would enter READ and receive the prompt

ENTER DISTRIBUTION DATA AND END WITH FILE

The user then builds a data file using the keywords given in Table 17-2. When he has completed building the file, he enters FILE and again receives the prompt

READ, EDIT, RUN OR END?

b. Running Program. To run the program at this point, the user enters RUN and the output as given in paragraph 17-5 is produced. Following the run, the user is again prompted

READ, EDIT, RUN OR END?

If the user wishes to return to the MAPS system level, he should enter END.

c. Editing File. If the user wishes to change the data file, he should enter EDIT, to which he receives the prompt

LIST, REPLACE, DELETE, ADD OR FILE?

These keywords are given in Table 17-2. List $XX.X_1$ TO $XX.X_2$ prints all the

Table 17-2. MAPS Editor Keywords

LIST LINES $XX.X_1$ TO $XX.X_2$
REPLACE LINE XX.X
DELETE LINE XX.X
ADD LINE XX.X
FILE

lines from $XX.X_1$ to $XX.X_2$. If neither argument is given, the entire file is printed. If one argument is given, all lines from that line to the end are printed. If the user enters REPLACE XX.X, the line immediately following the REPLACE command is placed in place of line XX.X. For example, if line 31 is ELEV 41 123, the user can change the elevation from 123 to 133 by entering

REPLACE 31

ELEV 41 133

The DELETE command deletes the line from the file and decreases the line number of lines after the deleted line by one. For example, if the file contained

41: OUTP 41 100

42: EXEC

43: END

and the user entered DELE 41, the file would contain

41: EXEC

42: END

The ADD command adds a line at the desired location. For example, if the file contained

29: TANK 2 115

30: TANK 3 120

and the user entered

ADD 30

TANK 4 150

the file would contain

29: TANK 2 115

30: TANK 4 150

31: TANK 3 120

The FILE command returns control to the distribution program.

17-5. Output. There are several types of tables printed by the program depending on the option specified. The line table and node table will be printed for all runs that go to completion. Each type of table is described below.

a. Line Table. Two types of tables are produced by the distribution system module. The first is the pipe summary, which gives

(1) direction of flow (from and to nodes),

(2) diameter, in.,

(3) length, ft,

(4) C coefficient,

(5) slope of energy grade line, ft/ft,

(6) head loss, ft,

(7) flow, gpm,

(8) velocity, ft/sec.

b. Node Table. The second table is the node summary, which gives

(1) node number,

(2) elevation of junction, ft,

(3) pressure, psi,

(4) elevation of hydraulic grade line, ft,

(5) net flow into/out of system at node, gpm,

(6) type of node (i.e., constant head, input, output).

Note that pumps requested by APUMP and BPUMP are called "CONSTANT HEAD" nodes in 6.

c. Loop Tables. The loop table output is divided into two parts. The first contains one row for each pipe. It contains the internal line number assigned to the pipe (I), the user's external node numbers of the pipe (KFM, KTO), and the internal node numbers (NFM, NTO) corresponding to the external node number. If there is a booster pump station assigned to the line, there are two additional columns: the first gives the row in the XB matrix containing the pump head characteristic curve coefficients for the pump while the second contains a + or -1 depending on if the flow is from KFM to KTO (+1) or the opposite (-1) direction. The second section of the loop tables contain, the loop number, the number of pipes in the loop (NPPLO), and the difference in head between the constant head node on the loop and the datum, followed by a list of the pipes in the loop.

d. Error Listing. The table titled "LOOP ERROR" gives the largest value of the correction factor, DELO, for the current iteration and the number of the loop to which the value applies. This output is helpful in determining how the program is converging.

e. Valves and Pumps. There are several special warning flags given when flow is in the wrong direction at valves. These are described in the section on flags. When pumps or valves are operating properly the following types of output are printed. If there are no valves or pumps of a given type, the entire section is skipped.

(1) Check Valves. The from and to nodes of each check valve are printed.

(2) PRV. The from and to nodes and the pressure at the downstream end of the PRV are printed.

(3) Booster Pump. For booster pumps at which only the head is specified (BOOST card), the table titled "BOOSTER PUMPS" is printed, giving the suction and discharge nodes, the head calculated by the program, and the flow entered by the user. Where the pump head curve is given (XBOOST card), the suction and discharge nodes are given, plus the three coefficients of the

pump curve (a, b, c), and the head produced by the pump. The pump curve coefficients are

$$H = a Q^2 + bQ + c \qquad (17-2)$$

where H = head, ft
Q = flow, gpm

(4) Pumps. For pumps, pumping into the system, only the node at which the pump is located and the pump curve coefficients (APUMP and BPUMP) are printed as the flow and head at the pump can be read from the node table. The coefficients are in the same order as for booster pumps above.

f. Run Statistics. At the end of the above tables, the program prints the node number of the datum node, the value of DELO (the largest loop correction factor) and the total number of iterations.

g. Warning Flags. The program provides warning flags to the user to indicate a condition in the program that must be corrected before a successful run can be made. The flags and the user's response are given in Table 17-3.

17-6. Valves. The program does not automatically control pressure and flow at check valves and PRVs, but it does provide sufficient information so that the user can manually correct the program for the effect of the valves.

a. Direction of Flow in Pressure Reducing Valves (PRV) and Check Valves. The program can recognize check valves and PRV's and test to determine: 1. if the flow is in the correct direction in the line, and 2. for PRV's if the PRV will be regulating pressure downstream. Since both types of valves have the effect of permitting flow in only one direction, they essentially remove the line from the network if the pressure gradient in the line is in the wrong direction. Since the program cannot remove a pipe from the network within a given run, it is necessary for the user to remove the pipe and rerun the network if the flow is in the wrong direction as the program will merely issue the warning "CHECK (or PRV) VALVE AT __ TO __ CLOSED--FLOW IN WRONG DIRECTION--REMOVE AND RERUN." In inputting data for valves, the nodes are entered in the direction in which flow can occur. In the case of the PRV, the valve is assumed to be located at the "from" node while it makes no difference for the check valve.

b. Pressure Regulation at PRV's. The pressure setting (i.e. the pressure maintained at the downstream end of a pressure reducing valve in psi) is the third value on a PRV card. If the pressure at the upstream node exceeds this pressure, the valve will be reducing the pressure in the pipe; therefore, the flow through the line and the pressures downstream will be reduced. When this occurs, the program prints "PRV AT __ WILL REDUCE PRESSURE; PRESSURE DOWNSTREAM OF PRV MUST BE CORRECTED." When this occurs, the user should check the pressure at the node. If it is close to the pressure setting the PRV will probably not have much effect on the system and the results are accurate. If the pressure is much higher than the setting, the PRV should be replaced by two nodes, a constant head tank or a pump in the downstream direction and a constant output node on the upstream end. The head for the constant head node

Table 17-3. Flags for Distribution Module

Flag	User Response
CAN ONLY USE RATIO ON RERUNS	Ratio card cannot be used on initial run. OUTPUT cards must be used.
CANT FIND BOOSTER xx_1 xx_2 PUMP IGNORED	LINE or PIPE card for line from xx_1 to xx_2 must preceed XBOOSTER card.
CANT FIND BOOSTER xx_1 xx_2 TO CHANGE	To rerun with XBOOSTER pump, line from xx_1 to xx_2 must be in original data set.
CANT FIND DATUM IN NODE TABLE	Node specified on DATUM card must have an ELEV and PUMP or TANK card in data file.
CANT FIND PIPE FROM PUMP xx	There is no pipe connecting pump at node xx to network. There should be one and only one pipe connected to APUMP or BPUMP pumps.
CANT FIND yyyy xx IN LOOP TABLE	Program was unable to locate a tank or pump to change the elevation for a rerun. Check node number on tank or pump to insure it agrees with original node number.
CANT FIND yyyy xx TO CHANGE	Program could not find node to change for rerun. Check node numbers to insure node agrees with original.
CANT FIND yyyy xx_1 xx_2 TO CHANGE	Program could not locate line xx_1 to xx_2 to change its values. Remember that the order of the nodes on this card is important. Try changing order.
CANT TRACE FLOW TO ORIGIN	Program cannot balance inputs and outputs for initial solution. Check to be sure input and output nodes are connected to system.
CHECK VALVE PRV AT xx_1 TO xx_2 CLOSED FLOW IN WRONG DIRECTION REMOVE AND RERUN	Valve is preventing flow in direction of decreasing hydraulic grade line. This has effect of removing pipe from network since flow cannot go backwards through valve. Pressures near valve are incorrect. Remove line xx_1 to xx_2 and rerun to determine effect of closed valve.

(continued)

Table 17-3 (continued)

Flag	User Response
DIAMETER CANNOT BE ZERO ON LINE xx_1 xx_2	Use a positive number for the third entry on a line card.
ERROR IN LOOP TABLE SETUP xx	Check data. Call program developers. xx is loop causing problems.
JUNCTION xx ON yyyy CARD NOT DEFINED BY LINE CARD	If a pump, tank, etc., is specified at a node, that node must also be specified on at least one PIPE or LINE card, and ELEVATION must be given.
LENGTH CANNOT BE ZERO ON LINE xx_1 xx_2	Use a positive number for the fourth entry on a line card.
MUST HAVE AT LEAST ONE CONSTANT HEAD NODE	There must be at least one PUMP or TANK node to serve as a datum. APUMP and BPUMP nodes cannot be datum nodes.
NODE xx NOT CONNECTED TO NETWORK a b c	There is a line not connected to the datum node except possibly through a booster pump station. Connect node xx to the system. a is the internal node number, b is the number of nodes, and c is the position in the node table of the node being addressed when the problem occurred.
NOT CONVERGING xx	The correction factor for iteration xx is larger than for iteration xx-1. If this occurs many times in a run check MAXERR of output to insure convergence has occurred or turn on convergence printout with an ERROR card to determine loop causing problem.
ONE PUMP OR TANK MUST BE SPECIFIED	There must be at least one constant head node (pump or tank) in the system to act as a datum.
PRV AT xx WILL REDUCE PRESSURE PRESSURE DOWNSTREAM OF PRV MUST BE CORRECTED	See paragraph 17-6a for discussion.
TOO MANY LOOPS REMOVE PIPES	Limits exceeded on variable NPPLO or DIFF. Increase limits in dimension statement or remove enough pipes to allow program to fit. Presently MAXN = 350.

(continued)

Table 17-3 (concluded)

Flag	User Response
TOO MANY PIPES IN LOOPS REMOVE PIPES	Limits exceeded on variable LPPI or LPSGN. Increase limits in dimension statement or remove enough pipes to allow program to fit. Presently MAXLP = 899.
TOO MANY yyyy CARDS LAST CARD IGNORED	Limits on dimension statement for yyyy card has been exceeded. Reduce number of yyyy cards or increase limit.
yyyy IS AN INVALID INPUT CARD TYPE	Look up correct keyword in Table 17-1.
yyyy NOT ALLOWED IN NEW FLOW RERUN CARD IGNORED	A yyyy card cannot be specified on a rerun. Change must be made on a new run.

(concluded)

is the pressure setting of the valve while the output flow can be estimated from

$$Q\ (est) = \frac{Q\ (\text{through valve first run}) * \text{Pressure setting}}{\text{Pressure at valve (first run)}} \qquad (17\text{-}3)$$

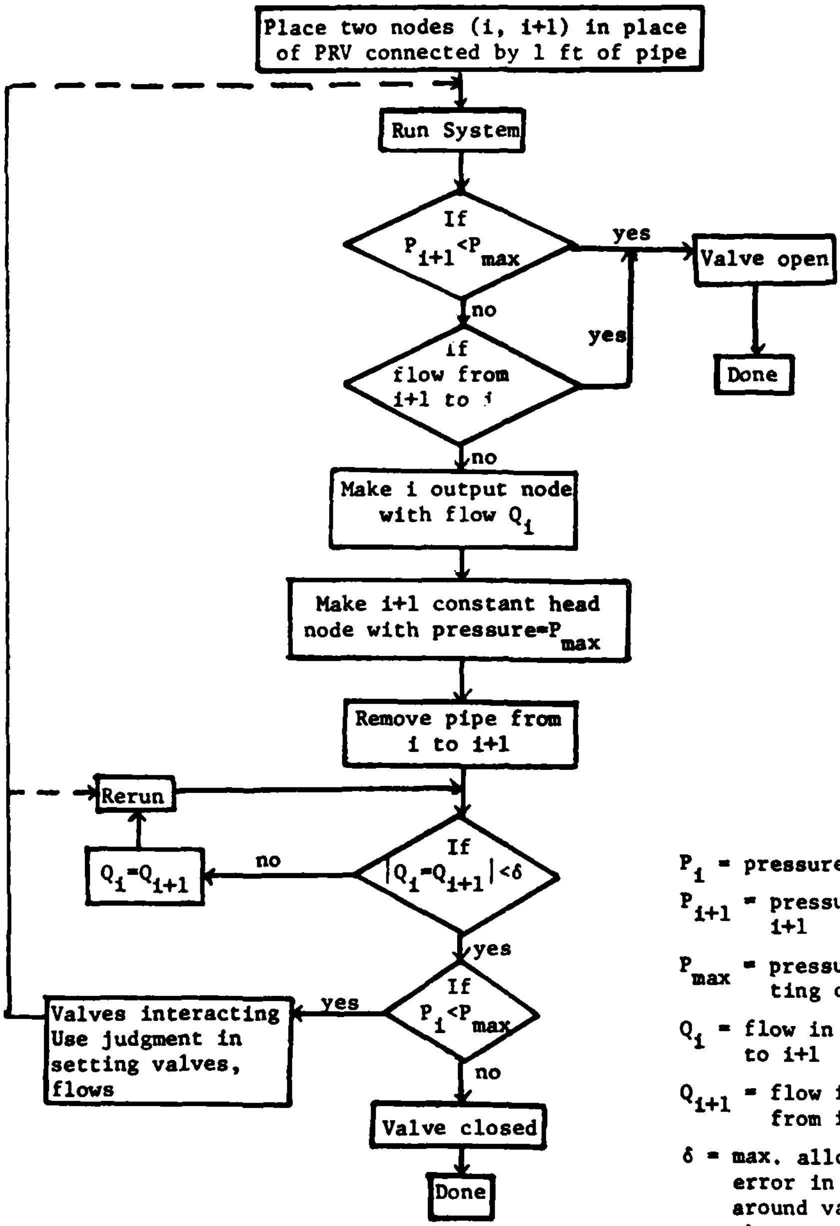
The network can then be rerun until the output flow from the constant head node equals the output from the constant output node. This procedure is shown schematically in Figure 17-2.

17-7. Special Consideration for Pumps. Pumps in a water distribution system can perform a wide variety of functions. They may be operated to maintain a constant head or flow, or be allowed to find their own operating points along a pump head curve. Similarly pumps may withdraw water from tanks, wells, or pressure pipes. Pump head curves may be available in some cases while in others only the head provided by the pump or the capacity of a pump (or pump station) may be known. Because of the variability in the function, operation and data availability for pumps, there are seven different keywords which can be used to represent pumps. Each keyword was discussed individually in Paragraph 17-2 and the relationship between the keywords is shown in Table 17-4.

a. Location. In modeling the behavior of a pump, it is necessary to know if the suction end of the pump is connected (1) to another portion of the system or (2) to a point outside of the distribution system. In the first case, the pump is called an "In-Line Booster" pump and the head at the suction end of the pump depends on the flows in the remainder of the system. In the second case, the pump is said to be pumping "Into the System" and the elevation specified on the node card is taken as the height of the hydraulic grade line at the suction inlet. The node elevation in these cases may not always be the elevation of the pump but rather may be the elevation of water in a tank. (See subparagraphs d and e).

b. Operating Mode. Figure 17-3 shows the three ways which the program can represent pumps. Knowing the characteristics of a given pump, and the manner in which it is operated, the user can select the correct keyword based on the discussion contained in the following paragraphs. From a computational standpoint (i.e., amount of computer time used), the constant head representation is most efficient while the pump curve representation is the least. In many cases though, it is impossible to simply specify the flow from a pump, as the flow will vary depending on the head near the pump.

c. Multiple Pumps at Pump Station. Most pumping stations do not consist of a single pump but rather a number of pumps connected in parallel. In most cases enough pumps are operated at anytime to insure that each pump is discharging at a flow near its maximum efficiency. Such operation produces a relatively constant head at most flows so the pump station can be modeled as a constant head node (PUMP or TANK card). If the head drops significantly, at higher flows, the station should be represented by a cumulative pump curve for all operating pumps (APUMP, BPUMP, XBOOST cards). For example, if there are four pumps each rated at 200 ft for 100 gpm, a single pump at node 50 would be described on a BPUMP card as BPUMP 50 200 100. If the four pumps are



P_i = pressure at i

P_{i+1} = pressure at i+1

P_{max} = pressure setting on valve

Q_i = flow in line i to i+1

Q_{i+1} = flow in line from i+1

δ = max. allowable error in flow around valve when shut

Figure 17-2. Procedure for pressure reducing valve operating in parallel (remembering that for parallel pumps, flows are added), the BPUMP card would be BPUMP 50 200 400.

d. Pumping from Tank. In specifying a pump taking suction from a tank, clearwell or pressure pipe, not part of the system being modeled, the user must be careful to insure that the total head (elevation of hydraulic grade line) at the discharge end of the pump is correct. (If a constant flow pump is specified, this is not a problem). For example, if a pump at node 10, located at elevation 400 ft, takes suction from a buried clearwell with water surface at 390 ft and produces 200 ft of head at 300 gpm (HGL at 590 ft), the following statements would be correct

```
     ELEV    10   400              ELEV   10   390
     TANK    10   190              TANK   10   200

     ELEV    10   400              ELEV   10   390
     BPUMP   10   190    300       PUMP   10   86.6

but  ELEV    10   400
     BPUMP   10   200
```

would be incorrect since the result is a HGL elevation of 600 ft.

Table 17-4 .
Guide for Selecting
Pump Keywords

Operating Mode \ Location	Into System	In-Line Booster
Constant Flow	INPUT (gpm)	BOOST (gpm)
Constant Head	PUMP (psi) TANK (ft)	-
Pump Curve	APUMP (ft, gpm) BPUMP (ft, gpm)	XBOOST (ft, gpm)

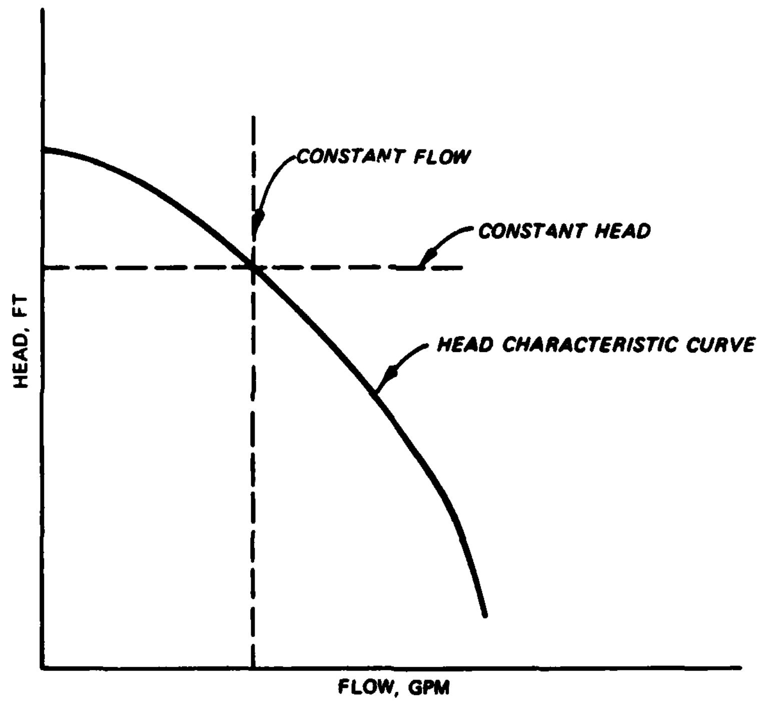


Figure 17-3. Alternative Representation of Pumps

e. Pumping from Wells. In modeling the head produced at a well the user should enter the actual pump elevation on the ELEV card and the head (above that elevation) on the APUMP or BPUMP card. Fluctuations in the groundwater table can be accounted for by changing the head at the pump. Where several wells are located together in a wellfield, it is often desirable to consider the well pumps as one pump station at a single node. For example, given data for the three pumps below

	Elevation (ft)	Head (ft)	Flow (gpm)
1.	402	200	100
2.	395	200	100
3.	420	180	100

The wellfield at node 20 can be represented as

```
ELEV    20   400
BPUMP   20   200   300
```

It is generally not desirable to use PUMP or TANK cards for well pumps as flow from well pumps is fairly constant but flows tend to vary widely at nodes represented by PUMP or TANK cards.

17-8. Example Problems. The following example problems illustrate some of the functions of the water distribution program. For both examples the MAPDIST (stand-alone) version of the program is used.

a. Example Problem 1. The network for this example is shown in Figure 17-4. In this example average flows are simulated first. Following this, the program is rerun with a fire flow of 500 gpm (in addition to the 75 gpm average flow) at node 8625. Note that the pressure is maintained between 40 and 70 psi for average conditions but that during the fire, pressures drop to as low as -26.8 psi. Usually it is desirable to maintain a pressure of at least 20 psi during fire flow conditions.

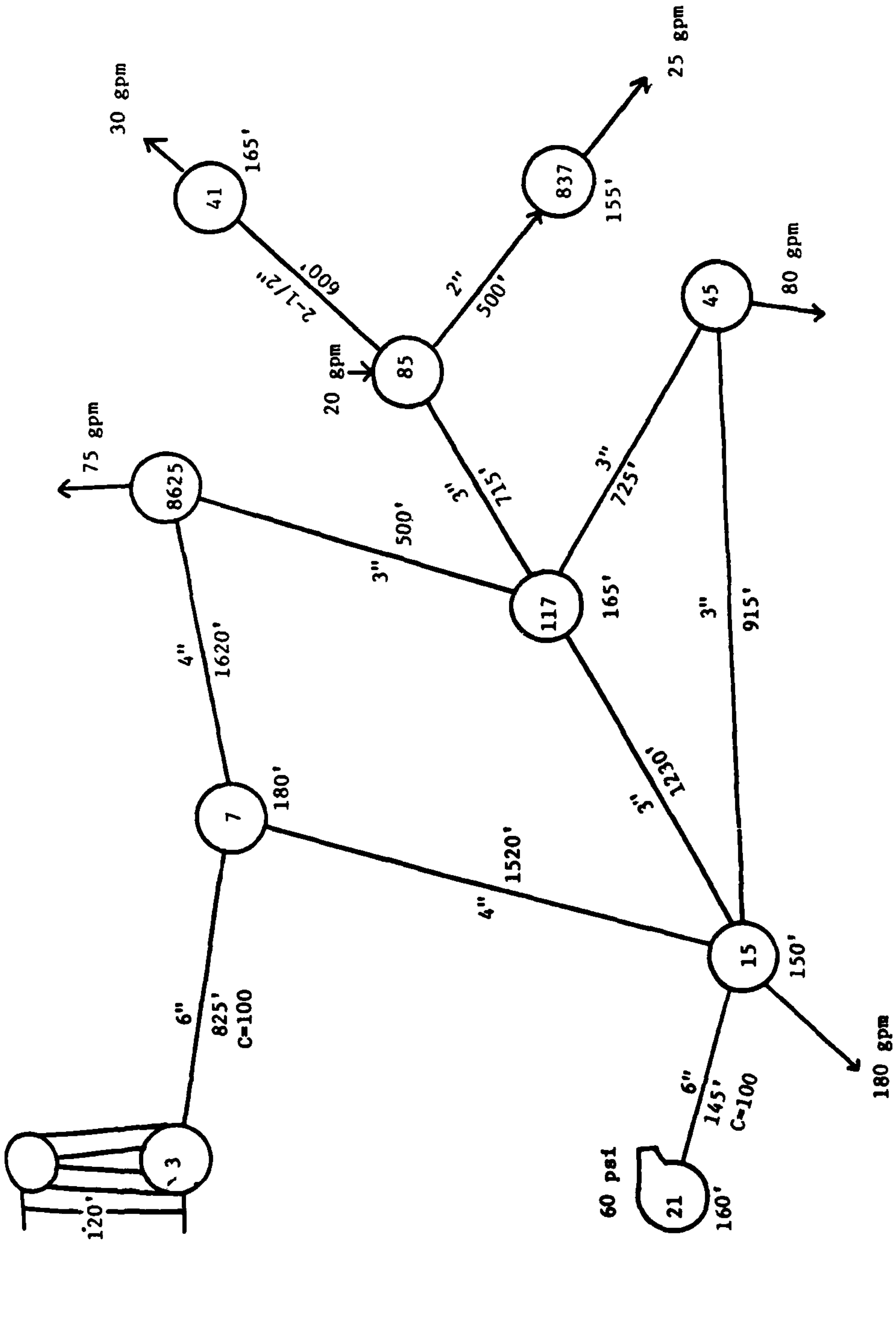


Figure 17-4. Input for example problem

```
  LIST,F=EX1A
JOB  FIREFLOW EXAMPLE PROBLEM
LINE    3    7    6  825 100
LINE 8625    7    4 1600
LINE    7   15    4 1500
LINE   15  117    3 1230
LINE   15   21    6  415 100
LINE   15   45    3  915
LINE 8625  117    3  500
LINE  117   85    3  715
LINE  117   45    3  725
LINE   85   41  2.5  600
LINE   85  837    2  500
ELEV    3  170
ELEV    7  160
ELEV   21  160
ELEV   15  150
ELEV  117  165
ELEV 8625  180
ELEV   45  160
ELEV   85  170
ELEV   41  165
ELEV  837  155
TANK    3  120
PUMP   21   60
OUTP 8625   75
OUTP   41   30
OUTP  837   25
OUTP   45   80
OUTP   15  180
INPUT  85   20
EXEC
OUTP 8625  575
EXEC
END
```

```
MAPLIST VERSION 31JUL81
MAP NODES   352

****************************************************

FIREFLOW EXAMPLE PROBLEM                    PAGE  1

                                          HEAD     HEAD
FROM    TO   DIA   LENGTH    C      LOSS/FT   LOSS      FLOW  VELOCITY
            (IN)   (FT)                        (FT)     (GPM)    (FPS)
   3     7   6.0    825.0 120.0   .00032     .26      40.6       .46
   7  8625   4.0   1620.0 120.0   .00374    3.29      70.8      2.24
  15     7   4.0   1520.0 120.0   .00154    2.35      35.3      1.00
  15   117   3.0   1230.0 120.0   .00954   11.73      49.3      2.24
  21    15   6.0    415.0 120.0   .01537    6.38     329.4      3.74
  15    45   3.0    915.0 120.0   .01413   12.93      60.9      2.77
8625   117   3.0    500.0 120.0   .00013     .06       4.8       .22
 117    85   3.0    715.0 120.0   .00507    3.62      35.2      1.59
 117    45   3.0    725.0 120.0   .00165    1.20      13.1       .57
  85    41   2.5    600.0 120.0   .00926    5.55      30.0      1.96
  85   837   2.0    500.0 120.0   .01959    9.79      25.0      2.55


FIREFLOW EXAMPLE PROBLEM                    PAGE  2

                                            NET FLOW
JUNCTION   ELEVATION     HGL   PRESSURE   INPUT OUTPUT
              (FT)       (FT)   (PSI)     (GPM)  (GPM)
     3        170.0     290.0    52.0      40.6          CONSTANT HEAD
     7        180.0     289.7    47.5
    21        160.0     299.0    62.0     329.4          CONSTANT HEAD
    15        150.0     292.1    61.5            200.0   OUTPUT
   117        165.0     280.4    49.9
  8625        180.0     280.4    43.1             75.0   OUTPUT
    45        160.0     279.2    51.6             60.0   OUTPUT
    85        170.0     276.7    46.2      20.0          INPUT
    41        165.0     271.2    46.0             30.0   OUTPUT
   837        155.0     266.9    48.5             25.0   OUTPUT
NODE    21 IS DATUM
    7 ITERATIONS REQUIRED
MAXERR=     .009
NOT CONVERGING    6
```

FIREFLOW EXAMPLE PROBLEM PAGE 3

FROM	TO	DIA (IN)	LENGTH (FT)	C	HEAD LOSS/FT	HEAD LOSS (FT)	FLOW (GPM)	VELOCITY (FPS)
3	7	6.0	825.0	100.0	.21452	11.98	319.5	3.63
7	8625	4.0	1020.0	120.0	.15672	159.89	371.6	9.49
15	7	4.0	1520.0	120.0	.00260	3.95	52.1	1.33
15	117	3.0	1230.0	120.0	.07979	98.15	155.3	7.05
21	15	6.0	415.0	100.0	.03973	16.49	550.5	6.25
15	45	3.0	915.0	120.0	.08737	79.95	163.1	7.41
117	8625	3.0	500.0	120.0	.13147	65.74	203.4	9.24
117	85	3.0	715.0	120.0	.00507	3.62	31.2	1.59
45	117	3.0	725.0	120.0	.02510	18.22	83.1	3.78
85	41	2.5	600.0	120.0	.00926	5.55	30.0	1.96
85	837	2.0	500.0	120.0	.01959	9.79	25.0	2.55

FIREFLOW EXAMPLE PROBLEM PAGE 4

JUNCTION	ELEVATION (FT)	HGL (FT)	PRESSURE (PSI)	NET FLOW INPUT (GPM)	NET FLOW OUTPUT (GPM)	
3	170.0	290.0	52.0	319.5		CONSTANT HEAD
7	180.0	278.2	42.4			
21	160.0	298.5	60.0	550.5		CONSTANT HEAD
15	150.0	282.0	57.1		150.0	OUTPUT
117	165.0	183.6	8.2			
8625	180.0	118.1	-26.8		575.0	OUTPUT
45	160.0	202.2	19.2		20.0	OUTPUT
85	170.0	190.2	4.4	20.0		INPUT
41	165.0	174.7	4.2		30.0	OUTPUT
837	155.0	170.4	6.7		25.0	OUTPUT

NODE 21 IS DATUM
14 ITERATIONS REQUIRED
MAXERR= .058

b. Example Problem 2a. Given the distribution system shown in Figure 17-5 consisting of a source (202), a tank (201), a high service area (300-303), a low service area (101-304), and a PRV (103-102), simulate the flows and pressures at a time when the tank is full and all flow is being provided by source 202. The pressures should be between 20 and 50 psi. The data file is given below followed by the output (including node table and convergence check).

```
  LIST,F=EX2
JOB  EXAMPLE W/PRV & CHECK VALUE
ELEV 100 100
ELEV 101 100
ELEV 102 100
ELEV 103 100
ELEV 200 200
ELEV 201 200
ELEV 202 200
ELEV 300 160
ELEV 301 160
ELEV 302 160
ELEV 303 160
ELEV 304 100
LINE 100 102 6   30
LINE 100 101 4  300
LINE 102 103 4    1
LINE 103 200 6 2400
LINE 101 304 4 1500
LINE 200 201 8  300
LINE 200 202 8  300
LINE 200 300 8 1500
LINE 300 301 6  300
LINE 301 303 6  300
LINE 300 302 6  300
LINE 302 303 6  300
LINE 303 304 4 3000
CHEC 202 200
PRV  103 102 50
OUTP 300 100
OUTP 301 100
OUTP 302 100
OUTP 303 100
OUTP 304  50
OUTP 101  50
TANK 201  50
INPU 202 500
LOOP TABLE
ERROR PRINT
EXEC
END
```

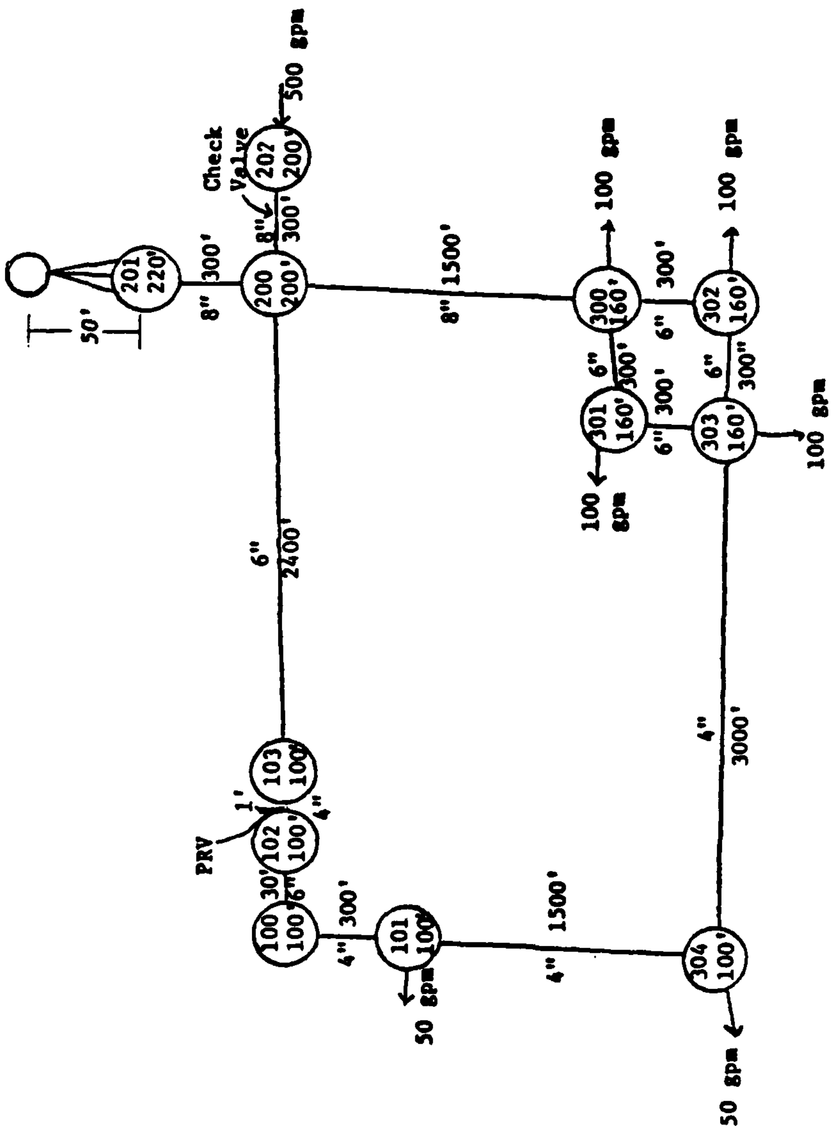


Figure 17-5. Example Problem 2a

```
**************************************************

     MAPLIST VERSION 3110001
     MAX NODES   350

**************************************************



LOOP TABLES
LINE  NFN  PTC  NFM  NTC  IBF
   1  100  102    1    2
   2  100  101    1    3
   3  102  103    2    4
   4  103  200    4    5
   5  101  304    3    6
   6  200  201    5    7
   7  200  202    5    8
   8  200  300    5    9
   9  300  301    9   10
  10  301  303   10   11
  11  300  302    9   12
  12  303  302   11   12
  13  304  303    6   11

 LOOP     1           NPFLC=    4             DIFF=        0.
 12  11   9  10

 LOOP     2           NPFLC=    9             DIFF=        0.
  5   2   1   3   4   8   9  10  13
LOOP ERROR
   1      77.3007
   1      21.4223
   1       1.5125
   1        .054[illegible]
```

EXAMPLE W/PRV & CHECK VALVE PAGE 1

FROM	TO	DIA (IN)	LENGTH (FT)	C	HEAD LOSS/FT	HEAD LOSS (FT)	FLOW (GPM)	VELOCITY (FPS)
[illegible]	[illegible]	[illegible]	3[illegible]	12[illegible]	.[illegible]	.22	91.3	1.24
100	101	4.0	300.0	120.0	.00736	2.21	91.3	2.33
103	102	4.0	1.0	120.0	.00736	.01	91.3	2.33
200	103	6.0	2400.0	120.0	.00102	2.45	91.3	1.04
101	304	4.0	1500.0	120.0	.00170	2.55	41.3	1.06
2[illegible]0	201	[illegible]	520.0	120.0	0.00000	0.00	0.0	0.00
202	200	8.0	300.0	120.0	.00585	1.75	500.0	3.19
200	300	8.0	1500.0	120.0	.00423	6.34	408.7	2.61
300	301	6.0	300.0	120.0	.00270	.81	154.3	1.75
301	303	6.0	300.0	120.0	.00039	.12	54.3	.62
300	302	6.0	300.0	120.0	.00270	.81	154.3	1.75
302	303	6.0	300.0	120.0	.00039	.12	54.3	.62
303	304	4.0	2000.0	120.0	.00009	.20	6.7	.22

EXAMPLE W/PRV & CHECK VALVE PAGE 2

JUNCTION	ELEVATION (FT)	HGL (FT)	PRESSURE (PSI)	NET FLOW INPUT (GPM)	NET FLOW OUTPUT (GPM)	
100	100.0	247.5	63.9			
101	100.0	245.3	62.9		50.0	OUTPUT
102	100.0	247.5	63.9			
103	100.0	247.5	63.9			
200	200.0	250.0	21.7			
201	200.0	250.0	21.7	0.0		CONSTANT HEAD
202	200.0	251.9	22.4	500.0		INPUT
300	160.0	244.0	36.4		100.0	OUTPUT
301	160.0	243.2	36.2		100.0	OUTPUT
302	160.0	243.2	36.0		100.0	OUTPUT
303	160.0	243.0	36.0		100.0	OUTPUT
304	100.0	242.8	61.8		50.0	OUTPUT

PRV AT 103 WILL REDUCE PRESSURE
PRESSURE DOWNSTREAM OF PRV MUST BE CORRECTED
****

CHECK VALVES
FROM TO
202 200

PRV'S
FROM TO PRESSURE(PSI)
103 102 5[illegible].0
[illegible]
4 ITERATIONS REQUIRED
MAXERR= .054

c. Example 2b. The output indicates that the pressures are adequate through the system but the values for pressure downstream of node 102 should be reduced by the PRV. To simulate this condition the PRV is replaced by a constant head node at 102 and a constant output at 103. By trial-and-error it is found that when the pressure is 15 psi at node 102, the flow to node 103 should be approximately 10 gpm. The input and output for the run are shown below.

```
         F=EX3
JOB  EXAMPLE W/PRV ACTING AS CONSTANT HEAD
ELEV 100 100
ELEV 101 100
ELEV 102 100
ELEV 103 100
ELEV 200 200
ELEV 201 200
ELEV 202 200
ELEV 300 160
ELEV 301 160
ELEV 302 160
ELEV 303 160
ELEV 304 100
LINE 100 102 6   30
LINE 100 101 4  300
LINE 103 200 6 2400
LINE 101 304 4 1500
LINE 200 201 8  300
LINE 200 202 8  300
LINE 200 300 8 1500
LINE 300 301 6  300
LINE 301 303 6  300
LINE 300 302 6  300
LINE 302 303 6  300
LINE 303 304 4 3000
CHEC 202 200
OUTP 300 100
OUTP 301 100
OUTP 302 100
OUTP 303 100
OUTP 304  50
OUTP 101  50
TANK 201  50
INPU 202 500
OUTP 103 10
PUMP 102 50
EXEC
END
 EOI ENCOUNTERED.
C>
```

d. Example 2c. Next, suppose that the source at node 202 is to be abandoned, and replaced by a 90 ft high tank at node 100 (e.g., at a new treatment plant) as is shown in Figure 17-6. The higher elevations near node 200 will be served by a booster pump which can produce 200 gpm at 100 ft of head.

```
JOB EXAMPLE WITH NEW SOURCE AND BOOSTER
DATUM 100
ELEV 100 100
ELEV 101 100
ELEV 102 100
ELEV 103 100
ELEV 200 200
ELEV 201 200
ELEV 202 200
ELEV 300 160
ELEV 301 160
ELEV 302 160
ELEV 303 160
ELEV 304 100
LINE 100 102 6   50
LINE 100 101 4  300
LINE 102 103 4    1
LINE 103 200 6 2400
LINE 101 304 4 1500
LINE 200 201 8  300
LINE 200 202 8  300
LINE 200 300 8 1500
LINE 300 301 6  300
LINE 301 303 6  300
LINE 300 302 6  300
LINE 302 303 6  300
LINE 303 304 4 3000
OUTP 300 100
OUTP 301 100
OUTP 302 100
OUTP 303 100
OUTP 304  50
OUTP 101  50
TANK 201  50
TANK 100  90
XBOO 102 103 100 200
EXEC
END
```

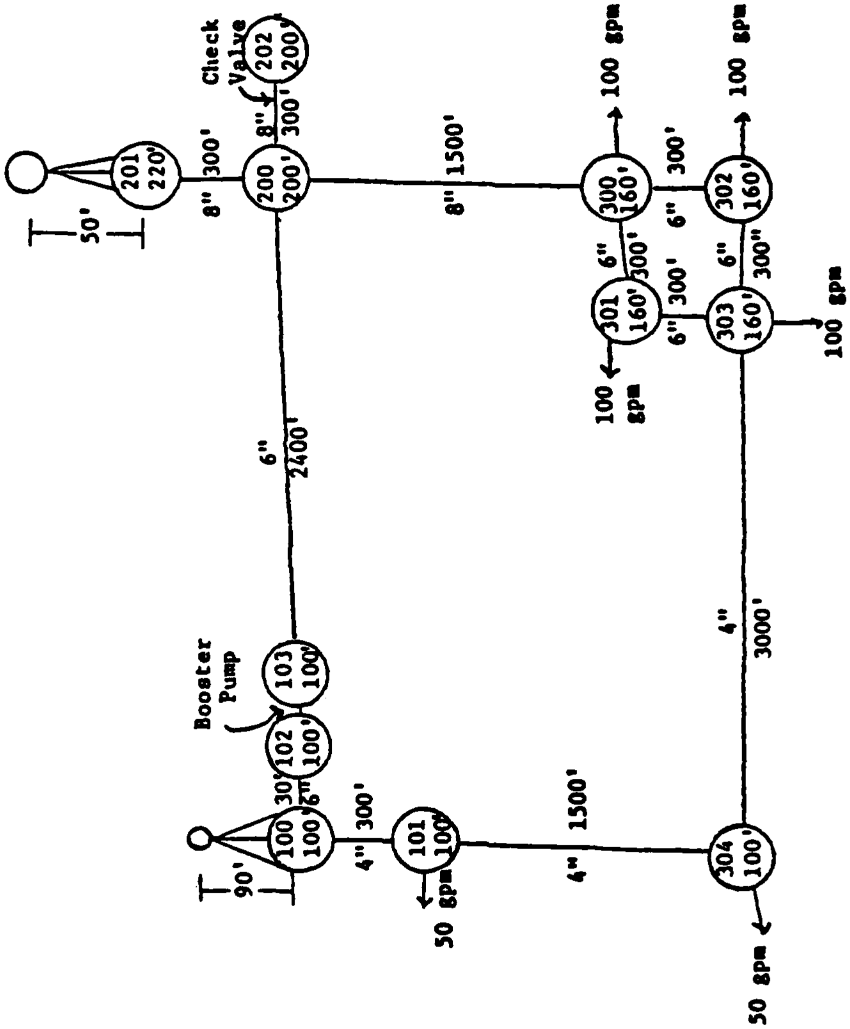


Figure 17-6. Example Problem 2c

1***

MAPLIST VERSION 011081
MAX NODES 352

****

1 EXAMPLE W/PRV ACTING AS CONSTANT HEAD PAGE 1

FROM	TO	DIA (IN)	LENGTH (FT)	C	HEAD LOSS/FT	HEAD LOSS (FT)	FLOW (GPM)	VELOCITY (FPS)
102	100	6.0	30.0	120.0	.00001	.00	8.8	.10
100	101	4.0	300.0	120.0	.00016	.05	8.8	.22
200	103	6.0	2400.0	120.0	.00002	.04	10.0	.11
304	101	4.0	1500.0	120.0	.00169	2.53	41.2	1.05
201	200	8.0	300.0	120.0	.00000	.00	1.2	.01
202	200	8.0	300.0	120.0	.00585	1.75	500.0	3.19
200	300	8.0	1500.0	120.0	.00566	8.49	491.2	3.14
300	301	6.0	300.0	120.0	.00418	1.25	195.6	2.22
301	303	6.0	300.0	120.0	.00111	.33	95.6	1.09
300	302	6.0	300.0	120.0	.00418	1.25	195.6	2.22
302	303	6.0	300.0	120.0	.00111	.33	95.6	1.09
303	304	4.0	3000.0	120.0	.00735	22.04	91.2	2.33

1 EXAMPLE W/PRV ACTING AS CONSTANT HEAD PAGE 2

JUNCTION	ELEVATION (FT)	HGL (FT)	PRESSURE (PSI)	NET FLOW INPUT (GPM)	NET FLOW OUTPUT (GPM)	
100	100.0	215.4	50.0			
101	100.0	215.4	49.9		50.0	OUTPUT
102	100.0	215.4	50.0	8.8		CONSTANT HEAD
103	100.0	250.0	64.9		10.0	OUTPUT
200	200.0	250.0	21.6			
201	200.0	250.0	21.7	1.2		CONSTANT HEAD
202	200.0	251.8	22.4	500.0		INPUT
300	160.0	241.5	35.3		100.0	OUTPUT
301	160.0	240.3	34.8		100.0	OUTPUT
302	160.0	240.3	34.8		100.0	OUTPUT
303	160.0	239.9	34.6		100.0	OUTPUT
304	100.0	217.9	51.0		50.0	OUTPUT

NODE 201 IS DATUM
4 ITERATIONS REQUIRED
MAXERR= .265

LIST,F=OUTS
1***

MAPLIST VERSION 31DEC81
MAX NODES 350

****
1EXAMPLE WITH NEW SOURCE AND BOOSTER PAGE 1

FROM	TO	DIA (IN)	LENGTH (FT)	C	HEAD LOSS/FT	HEAD LOSS (FT)	FLOW (GPM)	VELOCITY (FPS)
100	102	6.0	30.0	120.0	.02775	.23	273.1	3.10
101	120	4.0	300.0	120.0	.00075	.22	20.6	.65
102	103	4.0	1.0	120.0	.05584	.06	273.1	6.98
103	220	6.0	2400.0	120.0	.00775	18.60	273.1	3.12
304	101	4.0	1500.0	120.0	.00531	7.97	76.6	1.96
201	200	8.0	300.0	120.0	.00166	.50	253.5	1.62
200	202	8.0	300.0	120.0	0.00000	0.00	0.0	0.00
200	300	8.0	1500.0	120.0	.00643	9.65	526.6	3.36
300	301	6.0	300.0	120.0	.00646	1.94	247.5	2.81
301	303	6.0	300.0	120.0	.00248	.74	147.5	1.67
300	302	6.0	300.0	120.0	.00355	1.06	179.0	2.03
302	303	6.0	300.0	120.0	.00078	.23	79.0	.92
303	304	4.0	3000.0	120.0	.01346	40.39	120.6	3.23

1EXAMPLE WITH NEW SOURCE AND BOOSTER PAGE 2

JUNCTION	ELEVATION (FT)	HGL (FT)	PRESSURE (PSI)	NET FLOW INPUT (GPM)	NET FLOW OUTPUT (GPM)	
100	100.0	190.0	39.0	246.5		CONSTANT HEAD
101	100.0	190.2	39.1		52.0	OUTPUT
102	100.0	189.8	38.9			
103	100.0	268.1	72.8			
200	200.0	249.5	21.4			
201	200.0	250.0	21.6	253.5		CONSTANT HEAD
202	200.0	249.5	21.4			
300	160.0	239.9	34.6		100.0	OUTPUT
301	160.0	239.3	34.3		100.0	OUTPUT
302	160.0	238.8	34.1		100.0	OUTPUT
303	160.0	238.0	34.0		100.0	OUTPUT
304	120.0	198.2	42.5		50.0	OUTPUT

BOOSTER CURVE COEFFICIENTS HEAD
102 103 -.025E-05 0. .125E+03 78.39
NODE 100 IS DATUM
50 ITERATIONS REQUIRED
MAX ... 1.

APPENDIX B: DOCUMENTATION

This appendix consists of the Documentation for the MAPS Water Distribution Program (MAPDIST). It is Chapter 17 of Part 2 of the Maps Manual and, as such, the paragraph and figure numbers have the prefix "17."

* CHAPTER 17

WATER DISTRIBUTION SYSTEM ANALYSIS

17-1. Introduction. The water distribution system analysis module calculates the pressure, flows, and head loss in a looped or branched water distribution system using the Hardy-Cross Method. The module can be run as part of the MAPS program or as a stand-alone program called MAPDIST. Paragraph 17-2 describes input to the program, paragraph 17-3 describes the overall solution algorithms and paragraph 17-4 describes the method used by the program in setting up internal tables for the solution algorithm. Paragraphs 17-5 and 17-6 present methods on how valves and pumps are considered by the program. Paragraph 17-7 contains a description of the program's capability to rerun a system with modified data, and paragraph 17-9 lists the subroutines used by the program. The modifications made to the program since the original MAPS manual (EM 1110-2-502) was published were made only to the MAPDIST version of the program. The version contained in the MAPS program is the original (Nov 80) version.

17-2. Input Required.

Elevation of each node, ft
Length of each line, ft
Diameter of each line, in.
Hazen-William C for each line (default = 120)
Water elevation (above node elevation) for each tank, ft
Pressure at each pump, psi
Constant flow input or output at variable pressures, gpm
Number of iterations (default = 50)
Accuracy of iterative solution, gpm (default = 0.1)
PRV setting, psi
Check valve location
Level of detail of printouts
Pump characteristic curve (if using this type of pump)

To protect the user from errors caused by exceeding the limits of a dimension statement, every line of the user's input is tested against the maximum number of nodes, lines, tanks, etc. to insure that the limits are not exceeded. If they are exceeded, the input is not accepted and a warning is printed.

17-3. Solution Method. The program reads data from the input device until it encounters an EXEC card. At this time it identifies and stores the loops, establishes internal junction numbers, and assigns initial flows to the system. It balances the system using the Hardy-Cross method until the convergence criteria is met (DELQ(max) < DELQ(allowable)) or the maximum number of iterations is reached. It prints the output and stops if it receives an END command or an end-of-file from the input device, or continues to the next problem. The user can rerun the system for new flows once output has been printed by entering the data to be changed, and an EXEC command to begin the execution. The flowchart of the program is given in Figure 17-1. The Hardy-Cross method for balancing flows is based on the principle that, under steady conditions, the head loss around any loop is zero and the flow into a node is equal to flow

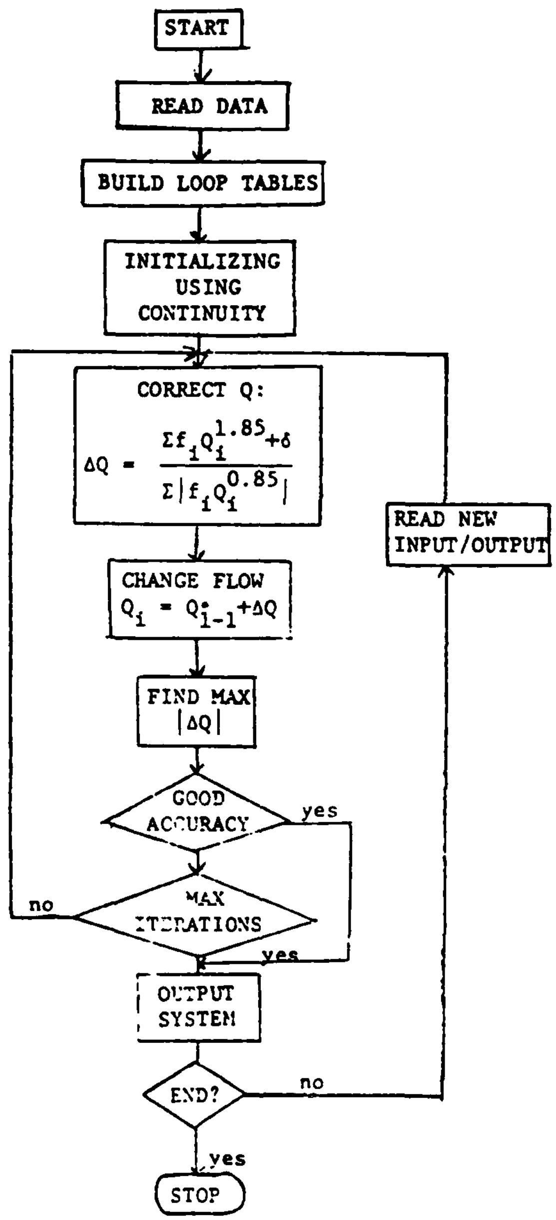


Figure 17-1. Flowchart for distribution program

out of that node. The initial flow assignments are made to meet the requirement of zero flow gained or lost in each node. The flows are then altered to comply with the head loss constraint using one of three formulas.

$$\text{DELQ} = \frac{\sum f_i \; Q_i 1.85 + \text{DIFF} - \text{HB}}{1.85 \sum f_i \; Q_i^{0.85} - \text{DB}} \qquad (17\text{-}1)$$

where

DELQ = correction to flow, gpm

f_i = friction factor for i-th line

Q_i = flow in i-th line, gpm

DIFF = correction for loop with tank or pump

HB = head provided by j-th booster pump, ft

= XB (j,1) * Q_i^2 + XB (j,2) * Q_i + XB (j,3)

DB = slope of head capacity curve for j-th booster pump, ft/gpm

= 2 * XB (j,1) * Q_i + XB (j,2)

Equation (17-1) is appropriate for all loops except those which have a pump acting as a water source (not an in-line booster) and a pump head curve is given for the pump. In that case DELQ is given by

$$\text{DELQ} = \frac{\text{LPUMP} * \text{B4} * (\text{HP2} - \text{HP})}{\text{DQ} - \text{SUMZ}} \qquad (17\text{-}2)$$

where

LPUMP = indicator of direction of flow in line

B4 = indicator of direction of pumping

HP2 = head produced by j-th pump at flow QP, ft

= A (j,1) * QP^2 + A (j,2) * QP + A (j,3)

HP = head required from pump to balance loop, ft

DQ = slope of head characteristic curve for pump j, ft/gpm
2 * A (j,1) * QP + A (j,2)

QP = flow through pump at last iteration

SUMZ = $1.85 \sum f_i \; Q_i^{\;0.85}$

In some special cases involving pumps in which a pump curve is given, the program also checks to insure that 1. flow is passing through the pumps in the correct direction, and 2. if head required by the line from the pump exceeds the peak head that can be exerted by the pump, the flow will be zero. In each case DELQ is set so that the flow in the line in the following iteration will be zero (i.e. DELQ = -QP).

The flow for the k-th iteration in the i-th line is corrected using

$$Q_{ik} = Q_{ik-1} + DELQ \qquad (17\text{-}3)$$

where k refers to the iteration number.

The flows are altered in such a way that the property of zero net change in flow at every node is maintained. The friction factors in each pipe are calculated using the Hazen-Williams equation

$$h_i = f_i \, Q_i^{1.85} \qquad (17\text{-}4)$$

where h_i = head loss in i-th pipe, ft

$$f_i = \frac{10.43 \, L_i}{C^{1.85} \, D_i^{4.87}}$$

L = length of i-th pipe, ft

C = Hazen-Williams coefficient

D = diameter of i-th pipe, in.

17-4. Establishing Loops. Another difficult problem in applying the Hardy-Cross method is that of automatically converting the user's description of the system into a table of loops (LPPI) for use by the program. The steps involved with this procedure are shown in Figure 17-2. The steps in this figure correspond to the box labelled BUILD LOOP TABLES in Figure 17-1. Definitions of variables used in the program are given in Table 17-1*. The program first renumbers the nodes for internal use and identifies the tank or pump with the greatest hydraulic head as the datum unless the user specifies another constant head node as the datum. The program builds a tree starting from the datum. It identifies loops by finding the same node in two locations in the tree and tracing the lines between the nodes.

a. Loops With Constant Head Nodes. For constant head nodes other than the datum, the difference in head (DIFF) between the two nodes must be added into the total head loss in these loops. It is calculated as

$$DIFF = REFHD - ELEV - HEAD \qquad (17\text{-}5)$$

where

REFHD = head at datum, ft

= $ELEV_d + HEAD_d$ for datum

ELEV = elevation at other constant head node, ft

$$HEAD = \begin{cases} \text{head at other constant head node, ft} \\ \text{0 if representing pump with pump curve} \end{cases}$$

* Located at end of Chapter.

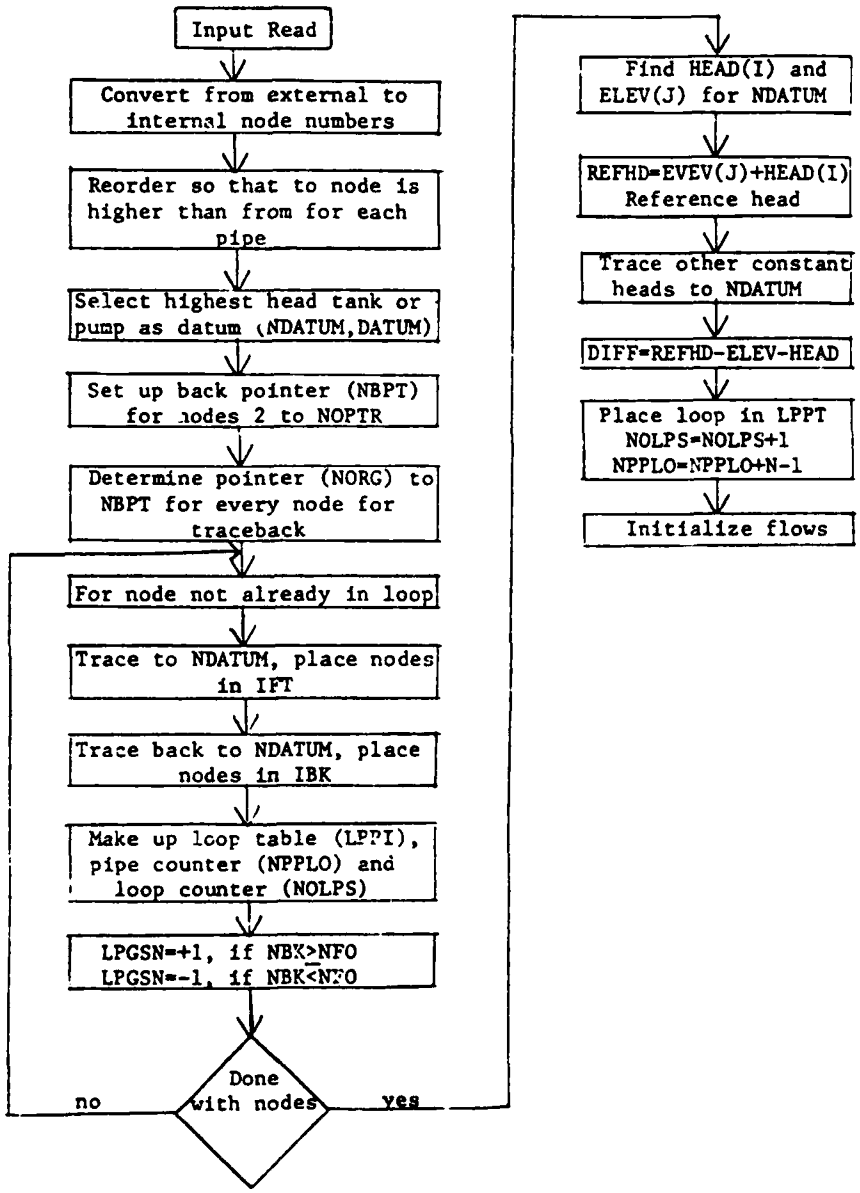
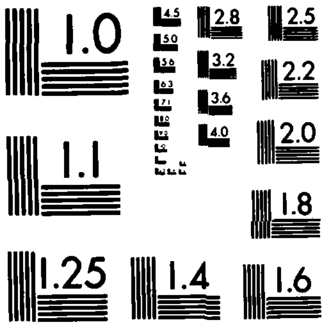


Figure 17-2. Flowchart for building loop tables

AD-A122 614 WATER SUPPLY ANALYSIS FOR THE GUAM COMPREHENSIVE STUDY 2/3
(U) ARMY ENGINEER WATERWAYS EXPERIMENT STATION
VICKSBURG MS ENVIRONMENTAL LAB T M WALSKI OCT 82
UNCLASSIFIED WES/MP/EL-82-5 F/G 13/2 NL



MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

b. Loop Tables. To illustrate the building of a loop table, tables for the example problem shown in Figure 17-3a are built in a step-by-step procedure. The data input is shown in Table 17-2. The user-supplied nodes are converted into internal nodes shown in Figure 17-3b. The internal pipe and node tables (Tables 17-3 and 17-4) are constructed for reference. The tree structure shown in Figure 17-4 is built using the pointer in Table 17-5. The program then traces the loops through the tree to build the ITBL array for each loop. These ITBL arrays are strung together to form LPPI, the loop table used by the program. The numbers stored in LPPI are not the beginning and ending nodes of the line, but the location of the line in Table 17-3. LPPI and ITBL are shown in Table 17-6.

c. Initial Solution. An initial starting solution is required for the Hardy-Cross solution. This solution is obtained by tracing the inputs and outputs back to the datum keeping track of the signs. The steps required to initialize the flows are shown in Figure 17-5, and correspond to the box labelled INITIALIZE USING CONTINUITY in Figure 17-1.

17-5. Valves. Some special tests are required in the program to determine if check valves and pressure reducing valves are being modeled properly.

a. Check Valves. The "from" and "to" external node numbers for the I-th check valve are stored in ICHK (I,1) and ICHK (I,2) respectively. Once the network has been solved, these valves are compared with the direction of flow in the arrays IS1 and IS2. If the direction is reversed a warning message is printed. A check valve does not affect the output flows and pressures.

b. Pressure Reducing Valves. The "from" and "to" external node numbers of the I-th pressure reducing valve are stored in IPV (I,1) and IPV (I,2) respectively. The pressure setting of the valve in psi is stored in PRV (I). After the line data is printed, the direction of flow in the PRV is checked the same way as for the check valve. After the node data is printed, the pressure is checked against the pressure at the "from" node. If the pressure at the node (XS) exceeds PRV, a warning is printed. A pressure reducing valve does not affect the flows and pressures printed.

17-6. Pumps. There are two types of situations in which pumps can be used: 1. pumping into system, and 2. in-line booster pumps. Pumps can be represented in MAPS as 1. a constant head node, 2. a constant flow node, or 3. a pump head characteristic curve. These three ways are shown graphically in Figure 17-6. Each of these cases is discussed in one of the following subparagraphs. Note that it is not possible to specify a constant head for an in-line booster pump.

a. Constant Head into System (TANK or PUMP Card). In this case the pump merely maintains a constant pressure at the pump node (much like a tank). No check is made to insure that water is actually flowing out of the pump. This corresponds to the horizontal line in Figure 17-6.

b. Constant Inflow to System (INPUT Card). In this case the pump forces a constant flow into the system at whatever pressure is required. This corresponds to the vertical line in Figure 17-6.

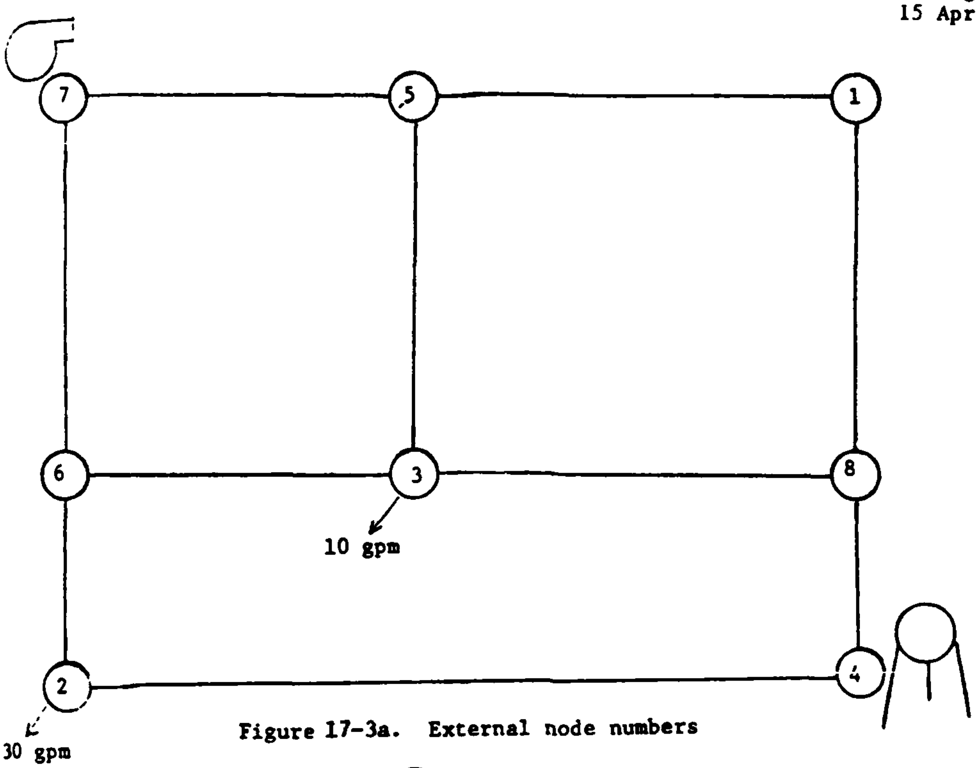


Figure 17-3a. External node numbers

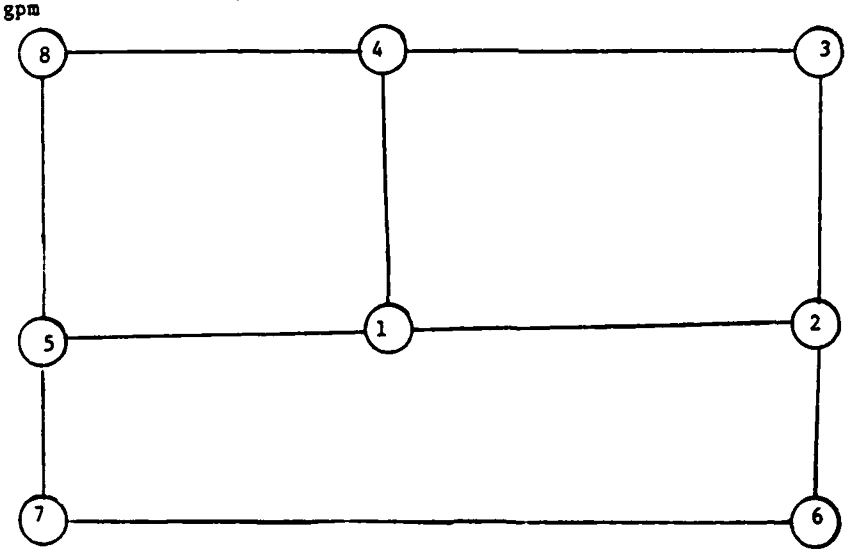


Figure 17-3b. Internal node numbers

Table 17-2. Input for System Shown in Figure 17-4

JOB	EXAMPLE	OF	LOOP	TABLES
LINE	8	3	6	100
LINE	5	1	6	100
LINE	8	1	6	50
LINE	5	3	6	50
LINE	6	3	6	100
LINE	8	4	6	50
LINE	4	2	6	200
LINE	6	2	6	50
LINE	7	6	6	50
LINE	7	5	6	100
ELEV	1	100		
ELEV	2	100		
ELEV	3	100		
ELEV	4	100		
ELEV	5	100		
ELEV	6	100		
ELEV	7	100		
ELEV	8	100		
PUMP	7	50		
TANK	4	115		
OUTPUT	2	30		
OUTPUT	3	10		
EXEC				
END				

Table 17-3. Internal Pipe Table

Line	KTO	KFM	NTO	NFM
1	8	3	2	1
2	5	1	4	3
3	8	1	2	3
4	5	3	4	1
5	6	3	5	1
6	8	4	2	6
7	4	2	6	7
8	6	2	5	7
9	7	6	8	5
10	7	5	8	4

Table 17-4. Internal Node Table

Internal Node I	External Node KJNOC(I)	NORG
1	3	4
2	8	8
3	1	6
4	5	3
5	6	2
6	4	9
7	2	5
8	7	1

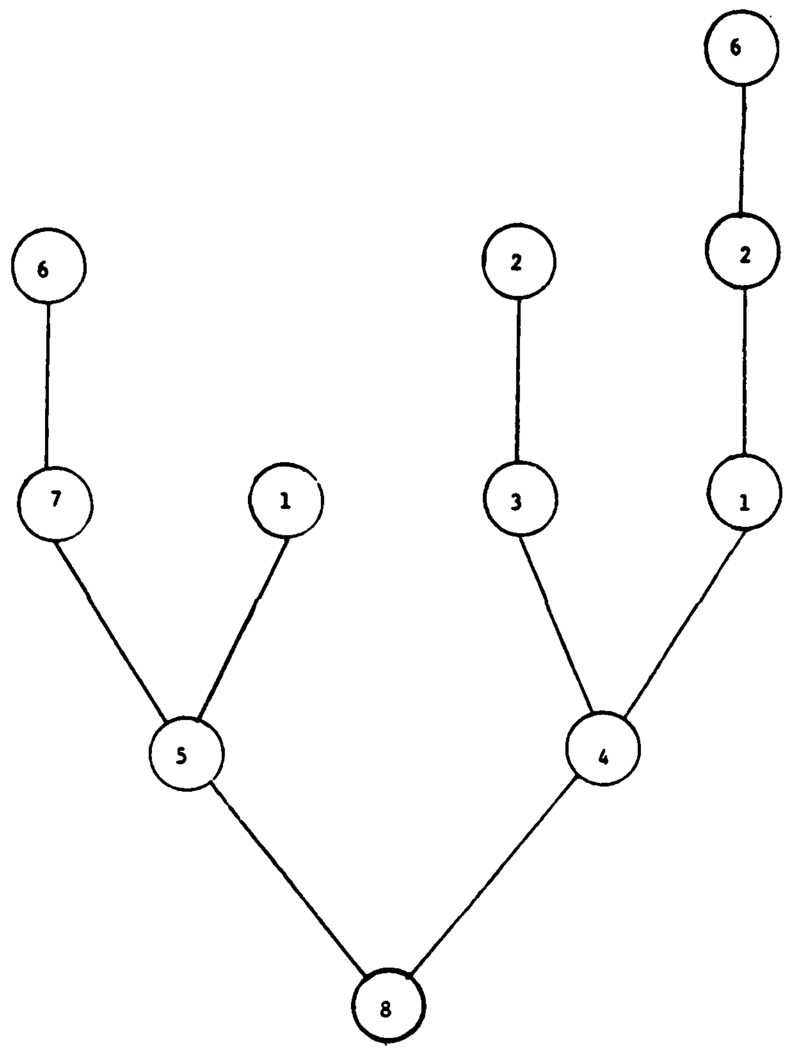


Figure 17-4. Tree structure used to build LPPI from NBPT
(Node 8 = NDATUM)

Table 17-5. Pointer Table

	JCT	NBPT
1	8	---
2	5	8
3	4	8
4	1	5
5	7	5
6	3	4
7	1	4
8	2	1
9	6	7
10	2	3
11	6	2

c. Pump Curve into System (APUMP and BPUMP Card). In this case, the pump characteristic curve is represented by a parabola with the equation

$$H = a\,Q^2 + b\,Q + c \qquad (17\text{-}6)$$

where

H = head produced by pumps, ft

Q = flow produced by pumps, gpm

a,b,c = coefficients

With the APUMP card, three points on the pump head curve are required, including the intercept with the vertical axis (0,H1). Letting the other points be called (Q2,H2) and (Q3,H3), the subroutine PARA calculates a, b, and c as follows

$$\begin{aligned} c &= H1 \\ a &= \left(\frac{(H3-c)}{Q3} - \frac{(H2-c)}{Q2}\right) \Big/ (Q3 - Q2) \qquad (17\text{-}7) \\ b &= \frac{(H3-c)}{Q3} - a * Q3 \end{aligned}$$

When BPUMP is used, only one point on the pump head characteristic curve is given and the assumptions are made that 1. the intercept with the vertical axis is at a head 25 percent greater than the given head, and 2. the derivative of the curve is 0 at that point. Therefore, given a single point (Q1,H1)

$$\begin{aligned} c &= 1.25 * H1 \\ b &= 0 \qquad (17\text{-}8) \\ a &= -.25{*}H1/Q1^2 \end{aligned}$$

Table 17-6. Loop Table (LPPI)
and
Loop Building Tables (ITBL)

ITBL		LPPI	LPSGN
1	NPPLO(1) = 4	4	+1
4		10	+1
8		9	-1
5		5	-1
1			

ITBL		LPPI	LPSGN
2	NPPLO(2) = 4	3	+1
3		2	+1
4		4	-1
1		1	+1
2			

ITBL		LPPI	LPSGN
6	NPPLO(3) = 7	6	-1
2		1	-1
1		4	+1
4		10	+1
8		9	-1
5		8	+1
7		7	-1
6			

NOLPS = 3

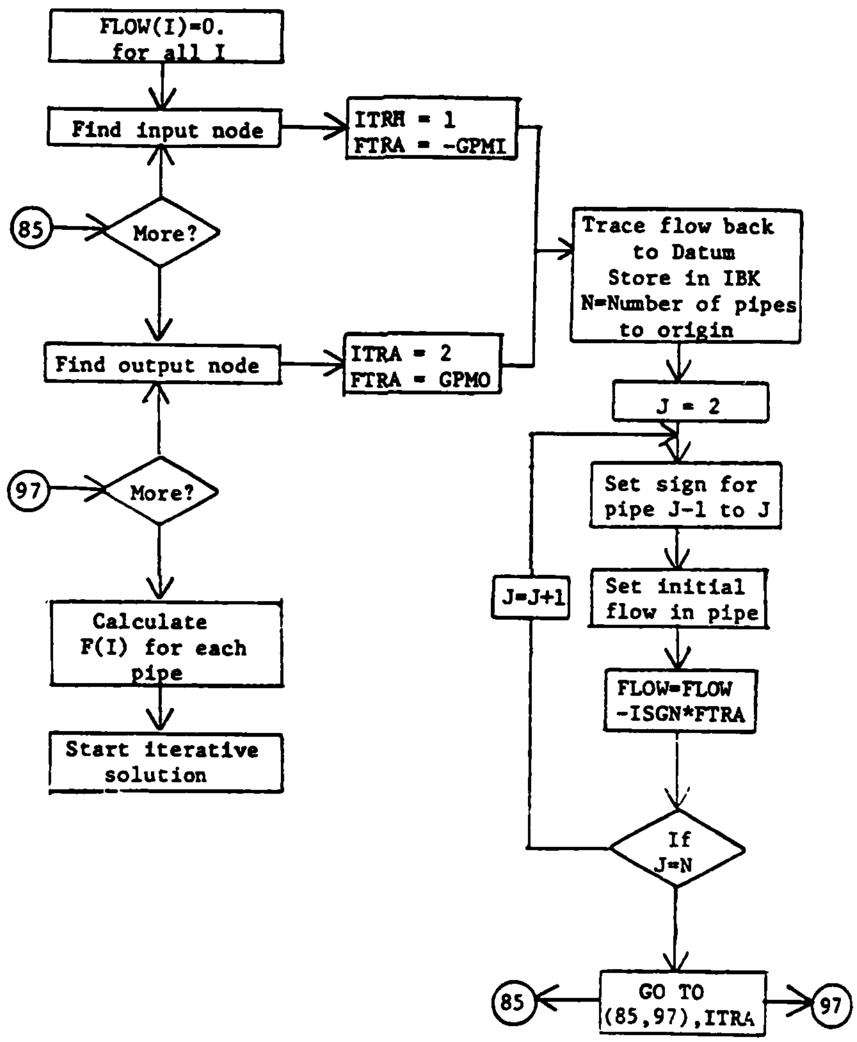


Figure 17-5. Flowchart for initializing flows

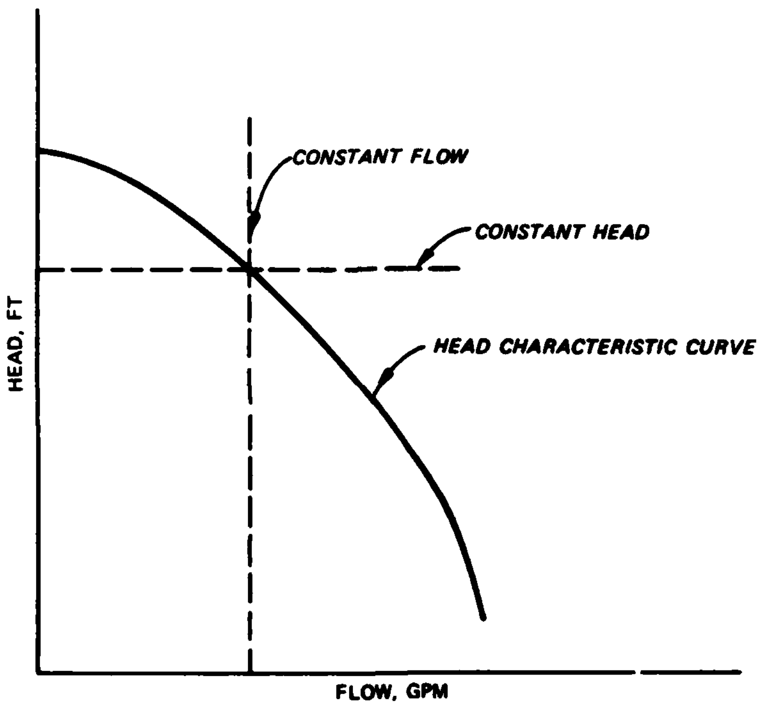


Figure 17-6. Alternative Representation of Pumps

d. Constant Flow Booster Pump (BOOSTER). A booster pump is represented by two nodes corresponding to the suction and discharge ends of the pump to deliver constant flow Q. In the program, the suction end of the pump is replaced by an output node with flow Q and the discharge end is replaced by an input node with flow Q. This is the reason that the booster pump cannot also be a constant head, input, or output node. Furthermore, since the suction and discharge end of the pump are not connected directly by a LINE, there must exist some other path to the datum from each end of the pump, else the program will not run. The head provided by the pump is calculated from the pressure at the discharge (p_2) and suction (p_1) end of the pump using

$$\text{Head} = (P_2 - P_1)/0.433 \qquad (17\text{-}9)$$

The head is not forced to fall on a pump head curve.

e. Booster Pump with Pump Curve (XBOOSTER). In this case the pump is represented by a pump head characteristic curve similar to that described in paragraph 17-6c for BPUMP except that the coefficients are stored in the array XB. The location of the coefficients in XB are given for the I-th line in IBP (I,1) while the direction in which the pump is pumping in the I-th line is given by a +1 or -1 in IBP (I,2).

17-7. Rerun Capabilities. Formerly the network could only be rerun with different INPUT or OUTPUT values. Presently reruns can be made for new PIPE, LINE, TANK, PUMP, COEF, PRV, ACCURACY data, and pump curve coefficients (APUMP, BPUMP, XBOOST). In each case the location of the node or line in the array within the program is located and the value is changed. If the node or line cannot be found, a warning is printed and the new values are ignored, except for CHECK and PRV, in which case a new valve is added. Output flows are modified using the value input on the RATIO card according to the formula

$$\text{GPMO (I)} = \text{RAT} * \text{GPMO (I)} \qquad (17\text{-}10)$$

where

GPMO = output for node I

RAT = value on ratio card

The above calculation is carried out only for output nodes that do not correspond to booster pumps (i.e. KJNO(JCTO)$\neq$IBOOS(I,1)). Once the values of GPMO are changed, the flows are traced back to the datum as was done for input and output nodes except that ITRA=JTRA=5.

17-8. Calculating Output. Once the iterative solution has terminated, the flows in each line are known but the user needs more output than merely these flows and an echo of the input data. These other quantities, such as head loss in each pipe, velocity, and pressure, are calculated once the iterative solution is complete. The methods used to determine these outputs are given below.

a. Head Loss. The head loss in each pipe is calculated as

$$HLOSS(I) = F(I)*GLOW^{1.85} \quad (17\text{-}11)$$

where

HLOSS(I) = head loss in I-th pipe, ft

F(I) = head loss constant (eq. 17-5)

GLOW = flow, gpm

The value printed as head loss is

$$H = |HLOSS(I)| \quad (17\text{-}12)$$

and the head loss per foot (HPF), given by

$$HPF = H/REACH(I) \quad (17\text{-}13)$$

where

REACH(I) = length of J-th pipe, ft

b. Velocity. The velocity is calculated as

$$VELP = \frac{GLOW*144}{448.8*DIA^{2}*0.785} \quad (17\text{-}14)$$

where

VELP = velocity, ft/sec

DIA = diameter, in.

c. Pressure. The value printed as pressure is the difference between the reference head and the elevation of the node minus the head loss between the datum node and the node.

$$PRESS = (REFHD-FOSS-ELEV)*0.433 \quad (17\text{-}15)$$

where

FOSS = $\sum HLOSS_k$ for all pipes k between reference head and node

ELEV = elevation at node, ft

REFHD = system reference head, ft

The height of the hydraulic grade line is given by

$$HGL = REFHD-FOSS \quad (17\text{-}16)$$

where

HGL = height of hydraulic grade line, ft

d. Flow. The flow into or out of a node is that specified by the user on the INPUT or OUTPUT card for those nodes. For constant head nodes, the values of the flow are the sum of the flows of all of the pipes coming into the constant head node

$$\text{SLOW} = \sum_{j} \text{FLOW}_j \qquad (17\text{-}17)$$

for all pipes, j, coming into the constant head node.

17-9. Routines Used. There are two MAPS water distribution programs. Stand-alone program MAPDIST is a separate program. Because MAPDIST is not tied to the MAPS data base system, the number of nodes considered by MAPDIST can be increased rather easily. At present the limit is set to 350 nodes. Subroutine MWATER is a MAPS subroutine called by subroutine DISTRI which also calls the data base editing subroutines DEDIT and DREAD. It is limited to systems with 350 nodes and 350 pipes. Both programs use the subroutine SCAN to read data. The DEDIT and DREAD subroutines are identical to the REDIT and RREAD subroutines used by the report generator module. The reader is referred to Chapter 21 for a description of these routines. The stand-alone program also calls a subroutine PARA which fits a parabolic system head curve to three points on the curve as given in an APUMP card.

Table 17-1. Definition of Variables for Water Distribution Module

Variable	Definition	Units
A(I,J)	Coefficients in the equation for pump head curve for pump I. If flow at pump I is QP, head produced is $HP2 = A(I,1)*QP^2 + A(I,2)*QP + A(I,3)$	---
ACCU	Accuracy for solution procedure; to stop the maximum DELQ must be less than ACCU (default = 0.1)	gpm
BHEAD	Head provided by booster pump	ft
BOOST	Flow through booster pump	gpm
B1, B2, B3, B4	Indicators of where flow is in positive or negative direction (+1. or -1.)	---
C(I)	Hazen-Williams C for I-th pipe	---
COEF	Constant Hazen-Williams C for all pipes if C(I) not specified	---
CUSE	$\begin{cases} C(I), & \text{if } C(I)>0 \\ COEF, & \text{if } C(I)\leq 0 \end{cases}$	---
DATUM	HEAD+ELEV for highest tank	ft
DB	Slope of booster pump head curve	ft/gpm
DELQ	Loop correction factor	gpm
DFCHK	Difference between peak hydraulic grade elevation and datum elevation. Warning is printed if DFCHK is negative.	ft
DIA(I)	Diameter of I-th pipe	in.
DIFF(I)	Difference in elevation between reference head and head at tank or pump for I-th loop	ft
DQ	Slope of pump head curve $= \begin{cases} 2*A(I,J)*QP + A(I,2) & \text{if} > 0 \\ 0 & \text{if} \leq 0 \end{cases}$	ft/gpm
DREF	Difference in head between original and rerun when constant head node is changed for rerun	ft
ELEV(I)	Elevation of I-th node	ft
ERR	Value of largest DELQ in iteration	gpm
ERRL	Value of ERR for previous iteration	gpm
F(I)	Friction constant for I-th pipe $= \dfrac{10.43*REACH(I)}{CUSE^{1.85} DIA(I)^{4.87}}$	---

(continued)

Table 17-1. (continued)

Variable	Definition	Units
FLOW(I)	Flow in I-th pipe	gpm
FOSS	Total head loss from reference head	ft
FTRA	Flow to output or from input node	gpm
G	Flow in pipe corrected for direction	gpm
GLOW	Flow in pipe corrected for direction	gpm
GPMI(I)	Flow into I-th input node	gpm
GPMIT	Input on input card for rerun	gpm
GPMO(I)	Flow out of I-th output node	gpm
GPMOT	Output on output card for rerun	gpm
H	Head loss in pipe $\|F*G^{1.85}\|$	ft
HB	Head provided by booster pump	ft
HEAD(I)	Head at I-th constant head pump or tank	ft
HIGH	Highest head encountered in finding datum	ft
HLOSS(I)	Head loss in I-th pipe (can be positive or negative)	ft
HP	Head required at pump	ft
HPF	Head loss per foot H/(REACH)	ft/ft
HP2	Head produced by pump at flow from previous iteration	ft
H1, H2	Head at suction and discharge end of booster pump	psi
I	Counter on loops	---
IB	Indicator on direction of flow in line from pump	---
IBK	Array containing number of nodes coming after IBK(1)	---
IBOOS(I,J)	Node number of suction (J=1) and discharge (J=2) ends of I-th booster pump	---
IBP(I,J)	Location in booster table of coefficients of I-th booster pump curve for J=1. Indicator of direction of flow in pump for J=2.	---
IBUF	Characters in columns 5 through 80 on input card	---
ICHK(I,J)	"From" (J=1) and "to" (J=2) node of I-th booster pump	---
ID	First four characters of input card	---

(continued)

Table 17-1. (continued)

Variable	Definition	Units
IDIFF(I,J)	Indicator on loop with pump = {Location in pump table of pump on loop, J=1 Location in elevation table of pump, J=2 Location in pipe table of pipe from pump, J=3} = 0 if no pump curve pump on I-th loop	---
IER	{0, do not print ERR 1, print ERR for each iteration}	---
IFT	Array containing numbers of nodes coming before IFT(1)	---
ILINE	Counter on number of lines printed	---
IP	Indicator on heading for pump curve coefficients, = 1, if heading already printed	---
IPAGE	Counter on number of pages printed	---
IPUMP	Line number of line from pump	---
IPV(I,J)	"From" (J=1) and "to" (J=2) node of I-th PRV	---
IREF	Placeholder on JCT in building loops	---
IS1(I), IS2(I)	External node number for I-th node or line in output	---
ISGN	Index on direction of flow (+1, -1)	---
IT	Counter on output nodes for ratio rerun	---
ITBL	Array containing node numbers of node in loop	---
ITLE	Title of run	
ITRA	1, if node is input node 2, if node is output node 3, if new input is zero 4, if new output is zero	---
J	Counter on loops	
JBP(I,J)	Beginning and ending node number for line with I-th booster pump	---
JCT(J)	Internal node number (e.g., if JCT(5)=7, the back pointer to node 7 is NBPT(5) and NORG(7)=5)	---
JCTE	Node number for elevation card	---
JCTI	Node number for input nodes	---
JCTIT	Node number for input nodes (rerun)	---

(continued)

Table 17-1. (continued)

Variable	Definition	Units
JCTO	Node number for output nodes	---
JCTOT	Node number for output node (rerun)	---
JCTT	Node number for tank or pump node	---
JER	Number of loop with max DELQ	---
JREF	Placeholder for JCT in building loops	---
JTRA	Index on tracing outputs to origin 4, output node 5, ratio	---
K	Counter on loops	---
KFM	External "from" node on pipe	---
KJNO	External junction number	---
KK	Counter on loops	---
KTO	External "to" node on pipe card	---
L	Counter on loops	---
LIST	Alphanumeric keywords recognized by program	---
LL	1, if IREF not input, output, tank, or pump 2, if IREF is input 3, if IREF is output 4, if IREF is tank or pump	---
LOOPT	0, no print 1, print loop tables	--- ---
LPPI	Array containing loops in order in which they are processed	---
LPSGN(I)	Direction of flow in I-th pipe	---
LPUMP	Direction of flow in Line I from pump 1, if LPSGN(I) > 0 -1, if LPSGN(I) ≤ 0	---
M	Counter on loops	---
MARK	0, if JCT is not already identified as to or from node 1, if JCT identified already	---
MAXLI	Number of lines per page of output (default = 50)	---

(continued)

Table 17-1. (continued)

Variable	Definition	Units
MAXN	Maximum number of nodes and pipes Currently = 350	---
MM	Counter on loops	---
N	Counter on loops	---
NBK	Placeholder used in building ITBL	---
NBOOS	Number of booster pumps	---
NBPT(J)	Node flowing into node at J-th location in JCT (e.g., if JCT(3)=4 and NBPT(3)=8, then node 4 receives flow from node 8 and NORG(4)=3)	---
NCHK	Number of check valves	---
NDATUM	Internal number of datum node	---
NFM	Internal "from" node number	---
NFO	Placeholder used in building ITBL	---
NN	Counter on loops	---
NOELE	Number of nodes for which elevation specified	---
NOIN	Number of input nodes	---
NOITER	Maximum number of iterations	---
NOJNC	Number of internal nodes	---
NOLIN	Number of pipes	---
NOLPS	Number of loops	---
NOOUT	Number of output nodes	---
NOPTR	Number of internal nodes with pointers	---
NORG(L)	Location in junction and back pointer table of node (1) (e.g., NORG(5)=2 means JCT(2)=5 and node coming to 5 is NBPT(2))	
NOTNK	Number of tanks and pumps	---
NPPLO	Number of pipes in I-th loop. Used in identifying loops in LPPI	---
NPRV	Number of PRV's	---
NTO	Internal "to" node number	---
OHEAD	Head for pump or tank before rerun	ft
PRESS	Dynamic pressure (REFHD-FOSS-ELEV)*0.433	psi

(continued)

Table 17-1. (concluded)

Variable	Definition	Units
PRV (I)	Pressure setting for I-th PRV	psi
QB	Flow through booster pump	gpm
QM	Flow at pump at maximum head	gpm
QP	Flow at pump from previous iteration	gpm
RAT	Ratio of output for current run to previous run	---
REACH(I)	Length of I-th pipe	ft
REFHD	Elevation of hydraulic grade line at datum node	ft
SLOW	Net flow into or out of node	gpm
STATIC	Static pressure (REFHD-ELEV)*0.433	psi
SUMH	Sum of head loss in loop Σ F(I) $G^{1.85}$	ft
SUMZ	Σ 1.85 F(I) $G^{0.85}$	---
THD	Total head at pump or tank before rerun	ft
VALUE	Array of values returned from SCAN subroutine	---
VELP(I)	Velocity in I-th pipe	ft/sec
XB(I,J)	Coefficients in pump head curve equation for booster pump I HB = XB(I,1) $*QB^2$ **2 + XB(I,2) *QB + XB(I,3)	---
XS(I)	Pressure at I-th node	psi
Z	1.85*F(I)*$G^{0.85}$	---

APPENDIX C: CALIBRATION OUTPUT

This appendix contains the printout from the calibration runs of the MAPS Water Distribution Program for the PUAG system. These printouts generally agree with the results as summarized in Table 4-2 of the main text and the data files prepared on tape for the PUAG (although there may be some minor differences). These printouts can be used to check the output of the model when it is run on a new system.

FROM	TO	DIA (IN)	LENGTH (FT)	HEAD C	HEAD LOSS/FT	LOSS (FT)	FLOW (GPM)	VELOCITY (FPS)
124	101	6.0	4500.0	110.0	.0003[illegible]	[illegible]	43.7	.57
102	103	6.0	6100.0	110.0	.00325	19.84	156.5	1.78
103	105	6.0	1300.0	110.0	.00217	2.33	[illegible]	1.43
106	105	12.0	4000.0	110.0	.00084	3.68	487.8	1.38
106	107	8.0	1400.0	110.0	.00005	.05	[illegible]	.18
105	108	12.0	5400.0	110.0	.00111	6.01	543.4	1.54
109	108	12.0	2000.0	110.0	.00002	.04	62.7	.18
108	110	12.0	2300.0	110.0	.00124	2.84	577.5	1.63
110	111	8.0	2800.0	110.0	.02337	65.42	969.7	6.19
102	106	8.0	5600.0	110.0	.00333	18.61	347.2	2.16
111	114	12.0	800.0	110.0	.00129	1.03	59[illegible].2	1.67
111	112	8.0	3200.0	110.0	.02916	93.32	1095.6	6.99
112	113	6.0	500.0	110.0	.08342	41.71	1929.4	[illegible].32
117	112	6.0	7000.0	110.0	.00965	67.52	291.7	3.29
113	253	14.0	11000.0	110.0	.00663	72.97	2141.2	4.47
111	115	8.0	4500.0	110.0	.00104	4.67	1[illegible].0	1.15
125	115	12.0	3700.0	110.0	.00034	7.11	407.2	1.33
115	116	12.0	1400.0	110.0	.00137	1.92	6[illegible].1	1.77
116	117	8.0	1000.0	110.0	.01623	16.23	779.6	5.08
117	119	12.0	1100.0	110.0	.00087	.95	473.1	1.3[illegible]
123	119	12.0	3400.0	110.0	.00046	1.55	335.6	.95
119	122	12.0	1800.0	110.0	.00112	2.02	546.1	1.55
122	121	8.0	1200.0	110.0	.00088	1.05	16[illegible].3	1.05
121	120	6.0	2700.0	110.0	.00013	.34	37.4	.37
119	120	6.0	4200.0	110.0	.00081	3.11	157.7	1.01
114	125	12.0	1600.0	110.0	.00097	1.56	505.[illegible]	1.44
120	252	12.0	1600.0	110.0	.00092	1.47	49[illegible]	1.39
122	126	12.0	2400.0	110.0	.00055	1.33	[illegible]	1.09
253	245	14.0	3100.0	110.0	.00646	20.04	211[illegible]	[illegible]
245	241	14.0	4500.0	110.0	.00629	28.32	2091.2	4.34
241	268	14.0	3000.0	110.0	.00207	6.22	[illegible]	[illegible]
268	240	8.0	2300.0	110.0	.00805	18.52	544.5	3.4[illegible]
268	239	14.0	2000.0	110.0	.00078	1.57	677.[illegible]	1.41
23[illegible]	243	12.0	4420.0	110.0	.00010	.46	151.3	.43
[illegible]	237	14.0	2000.0	110.0	.00012	.23	239.9	.50
236	231	[illegible].0	2000.0	110.0	.00008	.22	275.6	.44
232	231	12.0	8600.0	110.0	.00044	3.75	327.7	.93
[illegible]	235	10.0	3100.0	110.0	.00061	1.90	244.0	1.00
[illegible]	236	10.0	500.0	110.0	.00330	1.65	605.1	2.47
[illegible]	235	8.0	2500.0	110.0	.00564	14.11	449.7	2.87
239	238	12.0	400.0	110.0	.00128	.51	[illegible]	1.6[illegible]
238	242	8.0	[illegible]	110.0	.00[illegible]29	[illegible]	4[illegible].3	.31
234	242	12.0	1500.0	110.0	.00[illegible]01	[illegible]	40.3	.11
234	233	12.0	1600.0	110.0	.00170	2.71	630.7	1.94
233	232	[illegible].0	2400.0	110.0	.00387	9.30	669.3	2.70
243	234	12.0	3500.0	110.0	[illegible]	7.58	715.4	2.12
241	243	12.0	1600.0	110.0	.00170	2.72	683.9	1.94
24[illegible]	244	4.0	1600.0	110.0	.01146	18.33	126.3	2.72
247	246	6.0	900.0	110.0	.00227	2.04	129.7	1.46
248	247	8.0	1600.0	110.0	.00075	1.20	151.1	.97

FROM	TO	DIA (IN)	HEAD LENGTH (FT)	C	HEAD LOSS/FT	LOSS (FT)	FLOW (GPM)	VELOCITY (FPS)
257	12[illegible]	12.0	5500.0	110.0	.00097	5.33	504.7	1.43
257	258	12.0	2000.0	110.0	.00046	.92	377.[illegible]	.96
258	259	12.0	30.0	110.0	.00022	.21	228.0	.65
255	257	8.0	45[illegible]4.0	110.0	.00032	1.43	[illegible]5.1	.61
25[illegible]	255	8.0	1200.0	110.0	.02671	[illegible].05	1041.9	6.65
25[illegible]	249	8.0	400.0	110.0	.00004	.02	30.0	.19
252	25[illegible]	8.0	2800.0	110.0	.00588	16.49	460.0	2.94
255	254	8.0	2100.0	110.0	.01892	39.73	864.8	5.52
25[illegible]	221	12.0	22500.0	110.0	.00263	59.10	864.8	2.45
221	219	8.0	2100.0	110.0	.00576	12.09	154.7	[illegible].99
219	220	8.0	500.0	110.0	.00253	1.31	297.3	1.90
21[illegible]	218	8.0	6000.0	110.0	.00027	1.63	87.2	.56
222	218	12.0	4200.0	110.0	.00074	3.13	177.6	1.21
221	222	12.0	2100.0	110.0	.00047	.98	344.1	[illegible]
222	224	12.0	2700.0	110.0	.00061	1.65	392.6	1.11
224	225	10.0	4800.0	110.0	.00033	1.59	174.5	.71
225	226	8.0	1800.0	110.0	.00067	1.21	142.3	.91
226	227	8.0	1600.0	110.0	.00013	.21	59.0	.38
226	228	8.0	2000.0	110.0	.00003	.05	24.0	.16
228	229	8.0	5500.0	110.0	.00013	.72	59.0	.3[illegible]
232	228	8.0	4700.0	110.0	.00031	1.46	97.7	.62
206	232	8.0	920.0	110.0	.00103	.93	173.5	1.15
224	266	12.0	4000.0	110.0	.00011	.46	130.0	.45
230	225	6.0	1600.0	110.0	.00012	.20	26.[illegible]	.30
218	216	12.0	6800.0	110.0	.00109	7.44	538.7	1.53
216	217	12.0	4800.0	110.0	.00011	.54	158.3	.45
216	215	[illegible].0	630.0	110.0	.00010	.07	47.9	.31
21[illegible]	265	6.0	5300.0	110.0	.000[illegible]9	.40	22.0	.25
206	207	12.0	1000.0	110.0	.00002	.02	57.0	.17
214	265	6.0	1600.0	110.0	.00110	1.64	[illegible]	1.01
214	215	8.0	5200.0	110.0	.00026	1.36	[illegible]	[illegible]
213	214	6.0	4500.0	110.0	.00150	6.77	103.2	1.17
214	211	8.0	7300.0	110.0	.00828	60.46	553.3	3.53
21[illegible]	210	12.0	8500.0	110.0	.00147	12.47	631.3	1.79
206	205	12.0	3200.0	110.0	[illegible]	[illegible]	19[illegible]	.54
206	207	12.0	800.0	110.0	.00003	.02	[illegible]	[illegible]
208	206	12.0	1600.0	110.0	.00069	1.10	418.[illegible]	1.19
209	208	10.0	3400.0	110.0	.01015	34.52	1111.5	4.54
211	209	8.0	22[illegible]	110.0	.03401	74.83	1107.4	7.06
205	203	18.0	1700.0	110.0	.00001	.02	117.0	.15
203	204	12.0	1000.0	110.0	.00001	.01	[illegible]2.0	.15
231	204	16.0	7600.0	110.0	.00025	1.8[illegible]	315.7	.52
203	202	8.0	5400.0	110.0	.00009	.49	45.3	.31
202	201	6.0	1000.0	110.0	.00017	.17	32.2	.37
201	200	8.0	1200.0	110.0	.00001	.01	16.1	.10
171	118	12.0	1798.0	110.0	[illegible]	.14	131.4	.37
123	171	12.0	2.0	110.0	.00022	.00	225.0	.64
101	172	8.0	6298.0	110.0	.00418	26.30	382.1	2.44
172	102	8.0	2.0	110.0	.00363	.01	354.3	2.26
124	174	8.0	4498.0	110.0	.00014	.64	23.9	.33

FROM	TO	DIA (IN)	LENGTH (FT)	C	HEAD LOSS/FT	HEAD LOSS (FT)	FLOW (GPM)	VELOCITY (FPS)
174	100	6.0	2.2	110.0	0.00000	0.00	0.0	0.00
279	222	8.0	8938.0	110.0	.00023	2.06	79.4	.51
223	279	8.0	2.0	110.0	.00037	.00	123.0	.66
[illegible]	290	8.0	3000.0	110.0	0.00000	0.00	0.0	0.00
216	212	12.0	8700.0	110.0	.00103	8.96	521.3	1.48
212	210	12.0	79[illegible].0	110.0	.00003	.23	75.9	.22
281	248	12.0	850.0	110.0	.00013	.11	173.5	.49
250	282	12.0	850.0	110.0	.00063	.54	40[illegible].0	1.14
283	243	6.0	100.0	110.0	.0010[illegible]	.10	83.9	.95
244	284	6.0	2900.0	110.0	.00103	2.98	83.9	.95
284	285	6.0	130.0	10.0	.06671	8.67	83.9	.95
510	124	6.0	1.0	110.0	.02085	.02	75.6	.86
511	112	6.0	1.0	110.0	.03370	.03	554.0	[illegible].29
512	114	6.0	1.0	110.0	.00136	.00	97.7	1.11
513	111	6.0	1.0	110.0	.10027	.10	998.8	11.34
514	116	6.0	1.0	110.0	.01792	.02	393.7	4.47
515	109	6.0	1.0	110.0	.00125	.00	93.3	1.06
516	123	6.0	1.0	110.0	.00048	.00	55.9	.63
517	116	6.0	1.0	110.0	.00456	.00	187.9	2.13
501	211	6.0	1.0	110.0	.04326	.04	634.1	7.20
502	214	6.0	1.0	110.0	.04212	.04	625.0	7.10
503	212	6.0	1.0	110.0	.00447	.00	185.9	2.11
504	223	6.0	1.0	110.0	.00542	.01	206.2	2.34
505	222	6.0	1.0	110.0	.01930	.02	410.6	4.66
506	266	6.0	1.0	110.0	.00093	.00	79.4	.90
507	218	6.0	1.0	110.0	.00004	.00	13.9	.16
508	216	6.0	1.0	110.0	.00460	.00	188.8	2.14
526	102	6.0	1.0	110.0	.00264	.00	140.0	1.59
518	101	6.0	1.0	110.0	.01332	.01	335.4	3.81
519	106	6.0	1.0	110.0	.00551	.01	208.1	2.36
103	520	6.0	1.0	110.0	0.00000	0.00	0.0	0.00
523	256	6.0	1.0	110.0	.10841	.11	1041.9	11.83
524	257	6.0	1.0	110.0	.25865	.06	747.4	8.48
122	525	6.0	1.0	110.0	0.00000	0.00	0.0	0.00

JUNCTION	ELEVATION (FT)	HGL (FT)	PRESSURE (PSI)	NET FLOW INPUT (GPM)	NET FLOW OUTPUT (GPM)	
100	593.0	643.1	21.7			
121	460.0	642.1	78.9			
102	435.0	615.8	78.3			
103	430.0	596.0	71.9		50.6	OUTPUT
105	430.0	593.2	70.6		52.3	OUTPUT
106	360.0	597.0	102.6		52.3	OUTPUT
107	390.0	597.0	89.8		27.8	OUTPUT
108	410.0	587.2	76.7		30.6	OUTPUT
109	412.0	587.2	76.7		30.6	OUTPUT
110	410.0	584.0	75.5			
111	380.0	518.9	60.1		237.2	OUTPUT
112	372.0	425.6	21.1			
113	366.0	383.9	6.9	212.8		CONSTANT HEAD
114	375.0	517.9	61.9			
115	310.0	513.2	88.0		28.9	OUTPUT
116	300.0	509.3	90.6			
117	300.0	493.1	83.6		39.2	OUTPUT
118	300.0	493.5	83.8		131.4	OUTPUT
119	285.0	492.1	89.7		[illegible]	OUTPUT
120	280.0	488.7	90.4		138.9	OUTPUT
121	290.0	489.1	86.2		120.9	OUTPUT
122	282.0	490.1	88.6			
123	320.0	497.7	75.2			
124	455.0	643.7	81.7			
125	355.0	516.3	69.8		38.9	OUTPUT
171	320.0	493.7	75.2		30.6	OUTPUT
172	435.0	615.8	78.3		27.8	OUTPUT
174	540.0	643.1	44.8		28.9	OUTPUT
200	5.0	233.9	99.1		16.1	OUTPUT
201	5.0	233.9	99.1		16.1	OUTPUT
202	5.0	234.1	99.2		16.1	OUTPUT
203	5.0	234.6	99.4		16.1	OUTPUT
204	5.0	234.6	99.4		583.3	OUTPUT
205	15.0	234.6	95.1		75.0	OUTPUT
206	170.0	235.1	28.2		111.0	OUTPUT
207	150.0	235.1	36.9		73.9	OUTPUT
208	196.0	236.2	17.4		692.0	CONSTANT HEAD
209	200.0	270.8	30.6		75.9	OUTPUT
210	30.0	395.6	158.3		75.9	OUTPUT
211	145.0	345.6	86.9			
212	125.0	395.8	117.3			
213	349.0	383.3	14.9		631.3	CONSTANT HEAD
214	145.0	406.1	113.0			
215	145.0	404.7	112.4		111.4	OUTPUT
216	150.0	404.8	110.3			
217	8.0	404.2	171.6		[illegible]	OUTPUT
218	210.0	412.2	87.6			
219	320.0	389.5	30.8		79.1	OUTPUT
220	341.0	388.2	20.4		237.3	CONSTANT HEAD
221	225.0	401.6	76.5		79.1	OUTPUT

JUNCTION	ELEVATION (FT)	HGL (FT)	PRESSURE (PSI)	NET FLOW INPUT (GPM)	NET FLOW OUTPUT (GPM)	
222	205.0	400.6	84.7			
223	70.0	412.8	148.4			
224	235.0	395.0	71.0		59.0	OUTPUT
225	220.0	397.4	76.8		59.0	OUTPUT
226	220.0	396.2	76.3		59.0	OUTPUT
227	200.0	396.0	84.9		59.0	OUTPUT
228	220.0	396.1	76.3		59.0	OUTPUT
229	220.0	395.4	76.0		59.0	OUTPUT
230	220.0	397.6	76.9		59.0	OUTPUT
231	5.0	236.5	122.2		88.6	OUTPUT
232	60.0	240.2	78.0		88.6	OUTPUT
233	130.0	249.5	51.7		22.4	OUTPUT
234	112.0	252.2	61.6		22.4	OUTPUT
235	75.0	238.3	70.7		88.6	OUTPUT
236	55.0	236.7	78.7		88.6	OUTPUT
237	93.0	236.5	62.1		88.6	OUTPUT
238	111.0	252.4	61.2		88.6	OUTPUT
239	93.0	253.0	69.3		88.6	OUTPUT
240	196.0	236.0	17.3		636.2	CONSTANT HEAD
241	110.0	262.5	66.0		88.6	OUTPUT
242	120.0	252.2	65.9		88.6	OUTPUT
243	75.0	259.8	80.0		22.4	OUTPUT
244	20.0	271.6	108.9		22.4	OUTPUT
245	162.0	290.8	55.8		32.0	OUTPUT
246	20.0	289.9	116.9		22.4	OUTPUT
247	20.0	291.9	117.7		22.4	OUTPUT
248	95.0	293.1	85.8		22.4	OUTPUT
249	187.0	470.8	122.9		32.0	OUTPUT
250	205.0	470.8	115.1		30.0	OUTPUT
252	223.0	467.3	114.4		30.0	OUTPUT
253	200.0	310.9	48.0		30.0	OUTPUT
254	425.0	460.7	15.5			
255	400.0	500.3	43.5		22.0	OUTPUT
256	395.0	532.5	59.5			
257	405.0	499.0	42.7			
258	460.0	498.1	16.5		109.8	OUTPUT
259	458.0	498.1	17.4		228.0	CONSTANT HEAD
265	125.0	424.2	129.6		111.4	OUTPUT
266	235.0	398.5	70.8			
267	235.0	392.5	70.8		59.0	OUTPUT
268	112.0	254.5	61.7		88.6	OUTPUT
279	70.0	412.2	148.4		23.6	OUTPUT
281	155.0	293.2	59.9	173.5		CONSTANT HEAD
282	155.0	470.2	136.5		400.0	OUTPUT
283	50.0	259.9	90.9			
284	50.0	[illegible]	[illegible]			
290	65.0	404.2	146.9			
310	455.0	643.7	81.7	75.6		CONSTANT HEAD
311	370.0	425.6	24.1	554.0		CONSTANT HEAD
312	375.0	517.9	61.3	97.7		CONSTANT HEAD

JUNCTION	ELEVATION (FT)	HGL (FT)	PRESSURE (PSI)	NET FLOW INPUT (GPM)	NET FLOW OUTPUT (GPM)	
513	380.0	519.0	60.2	998.8		CONSTANT HEAD
514	410.0	584.3	75.5	393.7		CONSTANT HEAD
515	410.0	587.2	76.7	93.3		CONSTANT HEAD
516	322.0	493.7	75.2	55.9		CONSTANT HEAD
517	300.0	509.3	90.6	167.9		CONSTANT HEAD
501	145.0	345.6	86.9	634.1		CONSTANT HEAD
502	145.0	406.1	113.1	625.0		CONSTANT HEAD
503	125.0	395.8	117.3	186.9		CONSTANT HEAD
504	70.0	412.8	148.4	206.2		CONSTANT HEAD
505	205.0	400.7	84.7	410.6		CONSTANT HEAD
506	235.0	398.5	70.8	79.4		CONSTANT HEAD
507	210.0	412.2	87.6	13.9		CONSTANT HEAD
508	150.0	404.8	110.3	168.8		CONSTANT HEAD
526	435.0	615.8	78.3	142.0		CONSTANT HEAD
518	460.0	642.2	78.9	335.4		CONSTANT HEAD
519	360.0	597.0	102.6	208.1		CONSTANT HEAD
520	430.0	596.0	71.9	0.0		CONSTANT HEAD
523	395.0	532.6	59.6	1041.9		CONSTANT HEAD
524	425.0	499.1	40.7	747.4		CONSTANT HEAD
525	290.0	490.1	86.6	0.0		CONSTANT HEAD

PUMP CURVE COEFFICIENTS

510	-.471E-02	0.	.213E+03
511	-.997E-05	0.	.563E+02
512	-.410E-04	0.	.513E+02
513	-.913E-04	0.	.228E+03
514	-.416E-03	0.	.236E+03
515	-.492E-02	0.	.218E+03
516	-.127E-02	0.	.173E+03
517	-.228E-02	0.	.285E+03
501	-.942E-04	0.	.238E+03
502	-.995E-04	0.	.299E+03
503	-.148E-03	0.	.275E+03
504	-.756E-03	0.	.375E+03
505	-.121E-03	0.	.211E+03
506	-.964E-03	0.	.165E+03
507	-.109E-02	0.	.196E+03
508	-.155E-02	0.	.31[illegible]E+03
526	-.428E-03	0.	.188E+03
518	-.194E-03	0.	.203E+03
519	-.154E-02	0.	.305E+03
520	-.132E-02	0.	.110E+03
523	-.357E-04	0.	.174E+03
524	-.105E-03	0.	.153E+03
525	-.411E-03	0.	.185E+03

NODE 240 IS DATUM
55 ITERATIONS REQUIRED
MAXERR= 12.930

1INPUT GUAM AREA C PAGE 1

FROM	TO	DIA (IN)	LENGTH (FT)	C	HEAD LOSS/FT	HEAD LOSS (FT)	FLOW (GPM)	VELOCITY (FPS)
316	317	2.0	600.0	110.0	.03067	18.40	29.2	2.98
315	316	8.0	2750.0	110.0	.00004	.10	29.2	.19
315	313	8.0	2000.0	110.0	.00082	1.64	158.5	1.01
313	314	6.0	1500.0	110.0	.00024	.36	38.0	.43
303	315	8.0	4500.0	110.0	.00185	8.32	246.0	1.57
321	301	12.0	900.0	110.0	.00030	.27	268.0	.76
300	321	12.0	3500.0	110.0	.00038	1.31	302.0	.86
301	320	12.0	1000.0	110.0	.00023	.23	234.0	.66
312	304	8.0	1850.0	110.0	.00001	.02	15.5	.10
320	325	12.0	1100.0	110.0	.00017	.19	200.0	.57
326	304	12.0	2000.0	110.0	.00009	.17	135.4	.38
313	312	3.0	2800.0	110.0	.02905	81.35	82.5	3.74
312	311	6.0	1950.0	110.0	.00014	.27	28.4	.32
305	311	2.0	1000.0	110.0	.00004	.04	.8	.08
304	305	12.0	3500.0	110.0	.00006	.21	112.5	.32
308	311	8.0	1200.0	110.0	.00000	.01	9.3	.06
306	308	12.0	300.0	110.0	.00002	.01	61.4	.17
305	306	12.0	1000.0	110.0	.00003	.03	73.2	.21
306	307	6.0	1500.0	110.0	.00001	.01	5.9	.07
308	309	12.0	7750.0	110.0	.00001	.09	46.2	.13
309	310	8.0	1250.0	110.0	0.00000	0.00	0.0	0.00
300	326	1.0	100.0	1.0	1.51154	151.15	.4	.14

1INPUT GUAM AREA C PAGE 2

JUNCTION	ELEVATION (FT)	HGL (FT)	PRESSURE (PSI)	NET FLOW INPUT (GPM)	OUTPUT (GPM)	
300	350.0	355.0	2.2	302.4		CONSTANT HEAD
301	250.0	353.4	44.8		34.0	OUTPUT
303	290.0	295.0	2.2	246.0		CONSTANT HEAD
304	125.0	203.7	34.1		38.5	OUTPUT
305	30.0	203.5	75.1		38.5	OUTPUT
306	15.0	203.4	81.6		5.9	OUTPUT
307	40.0	203.4	70.8		5.9	OUTPUT
308	10.0	203.4	83.8		5.9	OUTPUT
309	10.0	203.3	83.7		46.2	OUTPUT
310	10.0	203.3	83.7			
311	10.0	203.4	83.8		38.5	OUTPUT
312	10.0	203.7	83.9		38.5	OUTPUT
313	10.0	285.0	119.1		38.0	OUTPUT
314	10.0	284.7	118.9		38.0	OUTPUT
315	50.0	286.7	102.5		58.3	OUTPUT
316	110.0	286.6	76.5			
317	150.0	268.2	51.2		29.2	OUTPUT
320	230.0	353.2	53.3		34.0	OUTPUT
321	240.0	353.7	49.2		34.0	OUTPUT
325	100.0	353.0	109.5		200.0	OUTPUT
326	100.0	203.8	45.0	135.1		CONSTANT HEAD

NODE 326 IS DATUM
13 ITERATIONS REQUIRED
MAXERR= .001

FROM	TO	DIA (IN)	LENGTH (FT)	C	HEAD LOSS/FT	HEAD LOSS (FT)	FLOW (GPM)	VELOCITY (FPS)
466	467	12.0	5000.0	110.0	.00017	.87	230.0	.57
433	466	12.0	5030.0	110.0	.00019	.94	227.6	.55
434	433	2.0	3000.0	110.0	.02946	88.38	[illegible].6	2.92
435	434	4.0	1300.0	110.0	.00[illegible]	4.39	40.4	1.03
436	435	6.0	1250.0	110.0	.00[illegible]	.53	[illegible]	.59
455	433	12.0	5420.0	110.0	.00017	.93	155.5	.55
436	455	2.0	3800.0	110.0	.02439	92.67	[illegible]	2.54
437	436	6.0	200.0	110.0	.00146	.29	121.0	1.1[illegible]
437	439	8.0	1500.0	110.0	.00013	.20	59.[illegible]	.38
436	438	6.0	500.0	110.0	.00022	.12	37.6	.43
439	440	6.0	300.0	110.0	.00037	.11	[illegible]	.54
440	441	6.0	200.0	110.0	.00007	.01	[illegible]	.13
443	442	4.0	800.0	110.0	.00071	.57	20.6	.62
442	443	2.0	1000.0	110.0	.00574	5.74	11.5	1.21
444	437	8.0	6300.0	110.0	.00084	5.29	163.6	1.03
445	444	12.0	945.0	110.0	.00002	.02	56.9	.16
446	444	8.0	3700.0	110.0	.00037	1.38	123.7	.65
446	447	6.0	3400.0	110.0	.00001	.03	13.9	.09
446	448	6.0	2200.0	110.0	.00004	.07	13.9	.16
449	446	8.0	3200.0	110.0	.00070	2.24	143.4	.93
449	450	8.0	3600.0	110.0	.00003	.12	27.3	.17
468	449	8.0	3700.0	110.0	.00126	4.66	28[illegible].0	1.28
469	456	3.0	6027.0	110.0	0.00000	0.00	0.0	0.00
456	455	12.0	13460.0	110.0	.00016	2.19	192.2	.5[illegible]
452	451	12.0	50.0	110.0	.00017	.01	200.0	.57
453	452	6.0	4700.0	110.0	.00146	6.8[illegible]	121.6	1.15
454	453	6.0	3100.0	110.0	.01083	33.58	320.0	3.61
455	457	6.0	3100.0	110.0	.00432	13.40	182.5	2.07
458	457	12.0	5000.0	110.0	.00000	.02	25.6	.07
457	456	12.0	3600.0	110.0	.00016	.59	192.2	.55
459	462	4.0	1300.0	110.0	.00225	2.67	42.0	1.07
459	461	2.0	200.0	110.0	.01579	3.16	[illegible].4	[illegible].18
462	459	8.0	2000.0	110.0	.00038	.76	[illegible]	.67
456	462	6.0	1100.0	110.0	.00044	.48	[illegible]	.72
465	458	12.0	9400.0	110.0	.00014	1.36	15[illegible]	.51

JUNCTION	ELEVATION (FT)	HGL (FT)	PRESSURE (PSI)	NET FLOW INPUT (GPM)	NET FLOW OUTPUT (GPM)	
433	20.0	345.1	140.8		13.5	OUTPUT
434	240.0	433.5	83.8		11.8	OUTPUT
435	300.0	437.9	59.7		11.8	OUTPUT
436	300.0	438.4	59.9		11.8	OUTPUT
437	302.0	438.7	62.1			
438	300.0	432.3	57.9		11.8	OUTPUT
439	300.0	438.5	60.0		11.8	OUTPUT
440	300.0	438.4	59.9		11.8	OUTPUT
441	300.0	438.4	59.9		11.8	OUTPUT
442	300.0	437.8	59.7		11.8	OUTPUT
443	300.0	432.1	57.2		11.8	OUTPUT
444	350.0	444.0	40.7			
445	404.0	444.0	17.3	56.9		CONSTANT HEAD
446	350.0	445.4	41.3		13.9	OUTPUT
447	366.0	445.3	34.4		13.9	OUTPUT
448	360.0	445.3	36.9		13.9	OUTPUT
449	300.0	447.6	63.9		27.3	OUTPUT
450	240.0	447.5	89.8		27.3	OUTPUT
451	325.0	354.9	13.0			IN BOOSTER
452	335.0	354.9	8.6	98.4		CONSTANT HEAD
453	40.0	361.8	139.3		15.9	OUTPUT
454	33.0	395.4	150.9	300.0		INPUT
455	40.0	345.6	132.3		19.5	OUTPUT
456	60.0	347.8	124.6			
457	210.0	349.4	59.9		15.9	OUTPUT
458	272.0	349.4	34.6		42.0	OUTPUT
459	200.0	385.0	80.1		42.0	OUTPUT
460	160.0	382.3	96.3		42.0	OUTPUT
461	290.0	381.9	39.8		22.4	OUTPUT
462	362.0	385.8	11.2		9.3	CONSTANT HEAD
465	9.5	349.6	147.3	189.3		CONSTANT HEAD
466	100.0	344.2	125.7		7.0	OUTPUT
467	360.0	343.3	-7.2		200.0	OUTPUT
468	335.0	452.3	50.8			OUT BOOSTER
469	60.0	347.8	124.6			

BOOSTER PUMPS

FROM	TO	PRESSURE(FT)	FLOW(GPM)
451	468	87.4	200.0

BOOSTER CURVE COEFFICIENTS HEAD

458	462	-.715E-02	0.	.129E+03	37.84

NODE 445 IS DATUM
17 ITERATIONS REQUIRED
MAXERR= .855

1 YONA-TALOFOFO (2X AVE FLOW) PAGE 1

FROM	TO	DIA (IN)	LENGTH (FT)	C	HEAD LOSS/FT	HEAD LOSS (FT)	FLOW (GPM)	VELOCITY (FPS)
466	467	12.0	5000.0	110.0	.00063	3.15	400.0	1.14
433	466	12.0	5200.0	110.0	.00068	3.38	415.2	1.18
434	433	2.0	3000.0	110.0	.03379	101.38	30.8	3.14
435	434	4.0	2300.0	110.0	.00331	7.62	54.4	1.39
436	435	6.0	1250.0	110.0	.00090	1.12	79.0	.89
455	433	12.0	5400.0	110.0	.00070	3.78	423.4	1.20
438	455	2.0	3800.0	110.0	.02643	100.44	26.9	2.75
437	436	6.0	200.0	110.0	.00308	.62	152.1	1.73
437	439	8.0	1500.0	110.0	.00047	.71	118.0	.75
436	438	6.0	500.0	110.0	.00040	.20	50.5	.57
439	440	6.0	300.0	110.0	.00128	.38	94.4	1.07
440	441	6.0	200.0	110.0	.00010	.02	23.6	.27
440	442	4.0	800.0	110.0	.00255	2.04	47.2	1.21
442	443	2.0	1000.0	110.0	.02269	22.68	23.6	2.41
444	437	8.0	6300.0	110.0	.00220	13.85	270.1	1.73
445	444	12.0	945.0	110.0	.00029	.27	262.7	.75
446	444	8.0	3700.0	110.0	.00000	.01	7.4	.05
446	447	8.0	3400.0	110.0	.00003	.11	27.8	.18
446	448	6.0	2000.0	110.0	.00013	.27	27.8	.32
449	446	8.0	3200.0	110.0	.00029	.94	92.6	.59
449	450	8.0	3800.0	110.0	.00011	.43	54.6	.35
468	449	8.0	3700.0	110.0	.00126	4.60	200.0	1.28
469	456	8.0	6027.0	110.0	0.00000	0.00	0.0	0.00
456	455	12.0	13460.0	110.0	.00074	9.94	435.5	1.24
452	451	12.0	50.0	110.0	.00017	.01	200.0	.57
453	452	6.0	4700.0	110.0	.00061	2.86	65.2	.72
454	453	6.0	3100.0	110.0	.01083	33.59	300.0	3.41
453	457	6.0	3100.0	110.0	.00536	16.60	205.0	2.33
458	457	12.0	5000.0	110.0	.00029	1.44	262.5	.74
457	456	12.0	3600.0	110.0	.00074	2.66	435.5	1.24
459	460	4.0	1300.0	110.0	.00741	9.63	84.2	2.15
459	461	2.0	200.0	110.0	.05684	11.37	40.8	4.17
462	459	8.0	2000.0	110.0	.00137	2.73	208.8	1.33
458	462	8.0	1100.0	110.0	.00042	.46	110.2	.70
465	458	12.0	9400.0	110.0	.00081	7.57	456.5	1.30

JUNCTION	ELEVATION (FT)	HGL (FT)	PRESSURE (PSI)	NET FLOW INPUT (GPM)	NET FLOW OUTPUT (GPM)	
433	20.0	319.1	129.5		39.0	OUTPUT
434	240.0	420.5	78.2		23.6	OUTPUT
435	300.0	428.1	55.5		23.6	OUTPUT
436	300.0	429.3	56.0		23.6	OUTPUT
437	300.0	429.9	56.2			
438	300.0	429.1	55.9		23.6	OUTPUT
439	300.0	429.2	55.9		23.6	OUTPUT
440	300.0	428.8	55.8		23.6	OUTPUT
441	300.0	428.8	55.8		23.6	OUTPUT
442	300.0	426.7	54.9		23.6	OUTPUT
443	300.0	406.1	45.9		23.6	OUTPUT
444	350.0	443.7	40.6			
445	404.0	444.0	17.3	262.7		CONSTANT HEAD
446	350.0	443.7	40.6		27.8	OUTPUT
447	366.0	443.6	33.6		27.8	OUTPUT
448	360.0	443.5	36.1		27.8	OUTPUT
449	300.0	444.7	62.6		54.6	OUTPUT
450	240.0	444.2	88.4		54.6	OUTPUT
451	325.0	355.0	13.0			IN BOOSTER
452	335.0	355.0	8.6	136.8		CONSTANT HEAD
453	40.0	357.8	137.6		31.8	OUTPUT
454	33.0	391.4	155.2	300.0		INPUT
455	40.0	328.6	125.0		39.0	OUTPUT
456	60.0	338.6	120.6			
457	210.0	341.2	56.8		31.8	OUTPUT
458	270.0	342.7	31.5		54.0	OUTPUT
459	200.0	381.3	78.5		54.0	OUTPUT
460	160.0	371.7	91.6		54.0	OUTPUT
461	290.0	369.9	34.6		43.8	OUTPUT
462	360.0	384.0	10.4	92.6		CONSTANT HEAD
465	9.5	350.2	147.5	456.5		CONSTANT HEAD
466	100.0	315.8	93.4		15.2	OUTPUT
467	360.0	312.0	-20.5		420.0	OUTPUT
468	335.0	449.3	49.5			OUT BOOSTER
469	60.0	338.6	120.0			

BOOSTER PUMPS

FROM	TO	PRESSURE(FT)	FLOW(GPM)
451	468	84.4	200.0

BOOSTER CURVE COEFFICIENTS HEAD

458 462 -.715E-02 0. .128E+03 41.82

NODE 445 IS DATUM

26 ITERATIONS REQUIRED

MAXERR= .853

07931 GUAM AREA D4
(INARAJAN-UMATAC) PAGE 1

FROM	TO	DIA (IN)	LENGTH (FT)	C	HEAD LOSS/FT	HEAD LOSS (FT)	FLOW (GPM)	VELOCITY (FPS)
402	403	4.0	1750.0	110.0	0.00000	0.00	0.0	0.00
419	417	2.0	33175.0	110.0	.00460	152.52	14.5	1.07
424	403	4.0	2700.0	110.0	.00621	16.75	[illegible]	1.95
403	406	3.0	2200.0	110.0	.00522	[illegible]	[illegible]	1.48
406	405	8.0	2050.0	110.0	0.00000	0.00	0.0	0.00
406	407	2.0	1300.0	110.0	.03747	48.71	32.5	3.33
407	408	2.0	5000.0	110.0	.03747	187.34	32.5	3.33
408	409	8.0	5200.0	110.0	.00002	.12	23.0	.15
412	410	8.0	2650.0	110.0	.00001	.02	15.3	[illegible]
412	411	12.0	300.0	110.0	.00043	.13	[illegible]	.92
415	412	3.0	1100.0	110.0	[illegible]	[illegible]	[illegible]	2.25
414	415	6.0	100.0	110.0	.01612	1.61	371.9	4.22
415	497	6.0	2750.0	110.0	.00007	.20	20.3	.23
498	416	6.0	1.0	110.0	.00022	.00	36.1	.41
409	416	8.0	2000.0	110.0	.00000	.01	[illegible]	.06
416	417	8.0	6750.0	110.0	.00004	.30	32.5	.21
414	413	6.0	3000.0	110.0	0.00000	0.00	0.0	0.00
418	414	12.0	25000.0	110.0	.00059	13.53	385.2	1.09
419	418	12.0	10175.0	110.0	.00059	5.99	385.2	1.09
420	419	6.0	3000.0	110.0	.01869	56.06	402.8	4.57
421	420	8.0	2500.0	110.0	.00475	11.89	403.9	2.62
421	422	6.0	2400.0	110.0	.00003	.07	12.6	.14
425	421	8.0	2500.0	110.0	.00531	13.27	435.1	2.75
425	426	8.0	3200.0	110.0	.00003	.10	20.4	.13
426	424	8.0	100.0	110.0	.00000	.00	10.0	.06
427	425	8.0	4000.0	110.0	.00632	25.26	477.9	3.25

1 GUAM AREA/D4 (INARAJAN-UMATAC) PAGE 2

JUNCTION	ELEVATION (FT)	HGL (FT)	PRESSURE (PSI)	NET FLOW INPUT (GPM)	NET FLOW OUTPUT (GPM)	
402	30.0	113.2	36.0			
403	30.0	113.2	36.0		43.8	OUTPUT
404	50.0	130.0	34.6	76.3		CONSTANT HEAD
405	180.0	327.5	63.9			
406	100.0	327.5	98.5			
407	150.0	278.8	55.8			
408	20.0	91.4	30.9		9.5	OUTPUT
409	30.0	91.3	26.5		13.3	OUTPUT
410	170.0	319.1	64.6		13.3	OUTPUT
411	294.0	319.0	10.8		[illegible].3	CONSTANT HEAD
412	280.0	319.2	17.0		13.3	OUTPUT
413	140.0	224.0	36.4			
414	50.0	224.0	75.3		13.3	OUTPUT
415	50.0	323.1	118.3			
416	5.0	91.3	37.4		13.3	OUTPUT
417	5.0	91.0	37.2		43.0	OUTPUT
418	50.0	237.5	81.2			
419	20.0	243.5	96.8		7.1	OUTPUT
420	20.0	299.6	121.1		7.1	OUTPUT
421	20.0	311.5	126.2		12.6	OUTPUT
422	30.0	311.4	121.8		12.6	OUTPUT
424	230.0	745.2	223.1		10.0	OUTPUT
425	20.0	324.7	132.[illegible]		16.4	OUTPUT
426	220.0	745.[illegible]	227.4		1[illegible].4	OUTPUT
427	120.0	350.0	99.6	477.9		CONSTANT HEAD
497	5.0	322.9	137.6		20.0	OUTPUT
498	5.0	91.3	37.4	36.1		CONSTANT HEAD

BOOSTER CURVE COEFFICIENTS

					HEAD
403	406	-.653F-01	0.	.294E+03	224.63
415	414	-.239E-02	0.	.471E+03	1[illegible].72
425	426	-.243F-01	0.	.438E+03	420.56

NODE 427 IS DATUM

11 ITERATIONS REQUIRED

MAXERR= .079

.06111 UMATAC-INAR
AJAN (2X AVE FLOW)   PAGE 1

FROM	TO	DIA (IN)	LENGTH (FT)	HEAD C	HEAD LOSS/FT	LOSS (FT)	FLOW (GPM)	VELOCITY (FPS)
402	403	4.0	1750.0	110.0	0.00000	0.00	0.0	0.00
419	417	2.0	33175.0	110.0	.00417	138.45	9.9	1.02
404	403	4.0	2700.0	110.0	.01410	38.08	119.0	3.04
403	406	3.0	2000.0	110.0	.00486	9.73	31.4	1.43
406	405	8.0	2050.0	110.0	0.00000	0.00	0.0	0.00
406	407	2.0	1300.0	110.0	.03503	45.54	31.4	3.21
407	408	2.0	5000.0	110.0	.03503	175.16	31.4	3.21
408	409	8.0	5200.0	110.0	.00001	.04	12.4	.08
412	410	3.0	2650.0	110.0	.00003	.08	26.6	.17
412	411	12.0	300.0	110.0	.00030	.09	270.0	.77
415	412	8.0	1100.0	110.0	.00306	3.37	323.2	2.06
414	415	6.0	100.0	110.0	.01543	1.54	363.2	4.12
415	497	6.0	2750.0	110.0	.00026	.72	4[illegible].0	.45
498	416	6.0	1.0	110.0	.00199	.00	116.9	1.33
410	409	8.0	2200.0	110.0	.00001	.02	14.2	.09
416	417	8.0	6750.0	110.0	.00021	1.42	76.1	.49
414	413	6.0	3000.0	110.0	0.00000	0.00	0.0	0.00
418	414	12.0	23000.0	110.0	.00060	13.83	389.8	1.11
419	418	12.0	10175.0	110.0	.00060	6.1[illegible]	389.8	1.11
420	419	6.0	3000.0	110.0	.01966	58.97	414.0	4.70
421	420	8.0	2500.0	110.0	.00515	12.8[illegible]	42[illegible].2	2.73
421	422	6.0	2400.0	110.0	.00011	.27	25.2	.2[illegible]
425	421	8.0	2500.0	110.0	.00633	15.83	[illegible]	[illegible]
425	426	9.0	3200.0	110.0	.00011	.3[illegible]	[illegible]	.3[illegible]
420	424	8.0	1[illegible].0	110.[illegible]	[illegible]	[illegible]	[illegible]	.1[illegible]
427	425	8.0	4000.0	110.0	[illegible]	[illegible]	[illegible]	[illegible]

1 UMATAC-INARAJAN ([illegible] FLOW)   PAGE 2

JUNCTION	ELEVATION (FT)	HGL (FT)	PRESSURE (PSI)	NET FLOW INPUT (GPM)	NET FLOW OUTPUT (GPM)	
402	30.0	91.9	26.8			
403	30.0	91.9	26.8		[illegible]7.6	OUTPUT
404	50.0	130.0	34.6	119.0		CONSTANT HEAD
405	180.0	311.7	57.0			
406	100.0	311.7	91.7			
407	150.0	266.1	50.3			
408	20.0	91.0	30.7		19.0	OUTPUT
409	30.0	90.9	26.4		26.6	OUTPUT
410	170.0	319.0	64.5		26.6	OUTPUT
411	294.0	319.0	10.8		270.0	CONSTANT HEAD
412	280.0	319.1	16.9		26.6	OUTPUT
413	140.0	208.0	29.5			
414	50.0	208.0	68.4		26.6	OUTPUT
415	50.0	322.5	118.0			
416	5.0	90.9	37.2		26.6	OUTPUT
417	5.0	89.5	36.6		56.0	OUTPUT
418	50.0	221.8	74.4			
419	20.0	228.0	90.1		14.2	OUTPUT
420	20.0	286.9	115.6		14.2	OUTPUT
421	20.0	299.8	121.2		25.2	OUTPUT
422	30.0	299.6	116.7		25.2	OUTPUT
424	230.0	685.1	197.0		22.0	OUTPUT
425	20.0	315.7	128.0		32.8	OUTPUT
426	220.0	685.1	201.4		32.8	OUTPUT
427	120.0	350.0	99.6	564.2		CONSTANT HEAD
497	5.0	321.6	137.2		42.0	OUTPUT
498	5.0	90.9	37.2	116.9		CONSTANT HEAD

BOOSTER CURVE COEFFICIENTS   HEAD

403	406	-.653E-01	0.	.234E+03	22[illegible].47
415	414	-.239E-02	0.	.431E+03	116.01
425	426	-.243E-01	0.	.438E+03	36[illegible].74

NODE   427 IS DATUM
   23 ITERATIONS REQUIRED
MAXERR=   .061

00051 AGAT-SANTA R
ITA (2X AVE FLOW) PAGE 1

FROM	TO	DIA (IN)	HEAD LENGTH (FT)	C	HEAD LOSS/FT	LOSS (FT)	FLOW (GPM)	VELOCITY (FPS)
316	317	2.0	600.0	110.0	.11055	66.33	58.4	5.97
315	316	8.0	2750.0	110.0	.00013	.36	58.4	.37
315	313	8.0	2000.0	110.0	.00160	3.19	227.2	1.45
313	314	6.0	1500.0	110.0	.00085	1.28	76.0	.86
303	315	8.0	4500.0	110.0	.00459	20.66	402.2	2.57
321	301	12.0	900.0	110.0	.00108	.98	536.0	1.52
300	321	12.0	3500.0	110.0	.00135	4.73	604.0	1.71
301	320	12.0	1000.0	110.0	.00084	.84	468.0	1.33
304	312	8.0	1850.0	110.0	.00010	.19	52.0	.33
320	325	12.0	1100.0	110.0	.00063	.69	400.0	1.14
326	304	12.0	2000.0	110.0	.00052	1.04	360.6	1.02
313	312	3.0	2800.0	110.0	.02448	68.54	75.2	3.41
312	311	6.0	1950.0	110.0	.00040	.77	50.2	.57
305	311	2.0	1000.0	110.0	.00016	.16	1.7	.18
304	305	12.0	3500.0	110.0	.00023	.80	231.6	.66
308	311	8.0	1200.0	110.0	.00003	.03	25.1	.16
306	308	12.0	300.0	110.0	.00008	.02	129.3	.37
305	306	12.0	1000.0	110.0	.00011	.11	152.9	.43
306	307	6.0	1500.0	110.0	.00003	.04	11.8	.13
308	309	12.0	7750.0	110.0	.00004	.33	92.4	.26
309	310	8.0	1250.0	110.0	0.00000	0.00	0.0	0.00
300	326	1.0	100.0	1.0	1.51154	151.15	.4	.14

1 AGAT-SANTA RITA (2X AVE FLOW) PAGE 2

JUNCTION	ELEVATION (FT)	HGL (FT)	PRESSURE (PSI)	NET FLOW INPUT (GPM)	OUTPUT (GPM)	
300	350.0	355.0	2.2	604.4		CONSTANT HEAD
301	250.0	349.3	43.0		68.0	OUTPUT
303	290.0	295.0	2.2	402.2		CONSTANT HEAD
304	125.0	202.8	33.7		77.0	OUTPUT
305	30.0	202.0	74.5		77.0	OUTPUT
306	15.0	201.9	80.9		11.8	OUTPUT
307	40.0	201.9	70.1		11.8	OUTPUT
308	10.0	201.9	83.1		11.8	OUTPUT
309	10.0	201.5	82.9		92.4	OUTPUT
310	10.0	201.5	82.9			
311	10.0	201.8	83.1		77.0	OUTPUT
312	10.0	202.6	83.4		77.0	OUTPUT
313	10.0	271.2	113.1		76.0	OUTPUT
314	10.0	269.9	112.5		76.0	OUTPUT
315	50.0	274.3	97.1		116.6	OUTPUT
316	110.0	274.0	71.0			
317	150.0	207.7	25.0		58.4	OUTPUT
320	230.0	348.4	51.3		68.0	OUTPUT
321	240.0	350.3	47.7		68.0	OUTPUT
325	100.0	347.8	107.3		400.0	OUTPUT
326	100.0	203.8	45.0	360.3		CONSTANT HEAD

NODE 326 IS DATUM
26 ITERATIONS REQUIRED
MAXERR= .001

FROM	TO	DIA (IN)	LENGTH (FT)	C	HEAD LOSS/FT	HEAD LOSS (FT)	FLOW (GPM)	VELOCITY (FPS)
124	101	6.0	4500.0	110.0	.00015	.68	29.7	.34
102	103	6.0	6100.0	110.0	.00480	29.30	193.3	2.19
103	105	6.0	1300.0	110.0	.00237	3.09	132.1	1.50
106	105	12.0	4600.0	110.0	.00098	4.50	507.0	1.44
106	107	8.0	1400.0	110.0	.00012	.17	55.6	.36
105	108	12.0	5400.0	110.0	.00109	5.90	538.5	1.53
109	108	12.0	2000.0	110.0	.00002	.03	56.2	.16
108	110	12.0	2300.0	110.0	.00107	2.47	533.5	1.51
110	111	8.0	2800.0	110.0	.02458	68.63	996.2	6.36
102	106	8.0	5600.0	110.0	.00496	27.75	419.2	2.68
111	114	12.0	800.0	110.0	.00156	1.25	653.4	1.85
111	112	8.0	3200.0	110.0	.02080	66.55	910.1	5.81
112	113	8.0	500.0	110.0	.07187	35.94	1779.[illegible]	11.36
117	112	6.0	7000.0	110.0	.00600	42.03	215.1	2.43
113	253	14.0	11000.0	110.0	.00764	84.00	2310.4	4.82
114	115	8.0	4500.0	110.0	.00110	4.93	185.4	1.18
125	115	12.0	3700.0	110.0	.00085	3.15	468.8	1.33
115	116	12.0	1400.0	110.0	.00124	1.74	576.3	1.64
116	117	8.0	1000.0	110.0	.01614	16.14	793.6	5.07
117	119	12.0	1100.0	110.0	.00094	1.04	497.2	1.41
123	119	12.0	3400.0	110.0	.00299	10.15	926.8	2.63
119	122	12.0	1800.0	110.0	.00294	5.30	919.7	2.61
122	121	8.0	1200.0	110.0	.00326	3.91	334.1	2.13
121	120	8.0	2700.0	110.0	.00049	1.33	122.3	.77
119	126	8.0	4200.0	110.0	.00251	10.56	290.4	1.85
114	125	12.0	1600.0	110.0	.00112	1.80	546.6	1.55
120	252	12.0	1600.0	110.0	.00331	5.30	980.0	2.78
122	120	12.0	2400.0	110.0	.00219	5.25	783.0	2.22
253	245	14.0	3100.0	110.0	.00727	22.55	2252.4	4.69
245	241	14.0	4500.0	110.0	.00692	31.14	2190.4	4.57
241	268	14.0	3000.0	110.0	.00259	7.77	1288.1	2.69
268	240	8.0	2300.0	110.0	.00256	5.88	293.0	1.87
268	239	14.0	2000.0	110.0	.00112	2.24	817.[illegible]	1.71
240	237	12.0	4400.0	110.0	.00167	7.35	677.1	1.92
237	236	14.0	2000.0	110.0	.00045	.90	499.[illegible]	1.04
236	231	16.0	2600.0	110.0	.00041	1.07	677.3	1.08
232	231	12.0	8600.0	110.0	.00028	2.40	257.2	.73
232	235	10.0	3100.0	110.0	.00023	.72	144.6	.59
235	236	10.0	500.0	110.0	.00123	.61	354.6	1.45
232	235	8.0	2500.0	110.0	.00428	10.70	387.3	2.47
239	238	12.0	400.0	110.0	.00151	.60	641.7	1.82
236	242	8.0	2600.0	110.0	.00021	.55	76.2	.49
234	242	12.0	1500.0	110.0	.00005	.07	101.0	.29
234	233	12.0	1600.0	110.0	.00144	2.30	623.8	1.77
233	232	10.0	2400.0	110.0	.00304	7.29	579.0	2.37
243	234	12.0	3800.0	110.0	.00212	8.04	769.5	2.18
241	243	12.0	1600.0	110.0	.00190	3.03	725.1	2.06
246	244	4.0	1600.0	110.0	.01759	28.14	134.1	3.42
247	246	6.0	900.0	110.0	.00416	3.75	178.9	2.03
248	247	8.0	1600.0	110.0	.00155	2.48	223.7	1.43

FROM	TO	DIA (IN)	LENGTH (FT)	C	HEAD LOSS/FT	HEAD LOSS (FT)	FLOW (GPM)	VELOCITY (FPS)
257	123	12.0	5500.0	110.0	.00515	28.31	1244.1	3.53
258	257	12.0	2000.0	110.0	.00118	2.36	561.1	1.59
259	258	12.0	50.0	110.0	.00217	.11	782.7	2.22
257	255	8.0	4500.0	110.0	.00009	.42	47.0	.30
256	255	8.0	1200.0	110.0	.02594	31.01	1020.4	6.54
250	249	8.0	400.0	110.0	.00014	.05	60.0	.38
252	250	8.0	2800.0	110.0	.02122	59.41	920.0	5.88
255	254	8.0	2100.0	110.0	.02066	43.39	907.0	5.79
254	221	12.0	22500.0	110.0	.00287	64.54	907.0	2.57
221	219	8.0	2100.0	110.0	.00300	6.31	319.7	2.04
219	220	8.0	500.0	110.0	.00489	2.44	416.1	2.66
218	219	8.0	6000.0	110.0	.00172	10.31	236.5	1.51
222	218	12.0	4200.0	110.0	.00023	.96	231.1	.66
221	222	12.0	2100.0	110.0	.00077	1.63	447.0	1.27
222	224	12.0	2700.0	110.0	.00231	6.23	806.4	2.29
224	225	10.0	4800.0	110.0	.00121	5.81	351.9	1.44
225	226	8.0	1800.0	110.0	.00243	4.37	285.2	1.82
226	227	8.0	1600.0	110.0	.00047	.76	115.0	.75
226	228	8.0	2000.0	110.0	.00009	.19	49.2	.31
228	229	8.0	5500.0	110.0	.00047	2.61	118.0	.75
230	228	8.0	4700.0	110.0	.00111	5.22	180.8	1.19
266	230	8.0	900.0	110.0	.00366	3.30	350.1	2.27
224	266	12.0	4000.0	110.0	.00046	1.83	336.5	.96
230	225	6.0	1600.0	110.0	.00041	.66	51.3	.58
218	216	12.0	6800.0	110.0	.00010	.68	147.9	.42
216	217	12.0	4800.0	110.0	.00041	1.96	316.6	.90
216	215	8.0	800.0	110.0	.00021	.17	76.7	.49
215	265	6.0	5300.0	110.0	.00025	1.33	39.3	.45
266	267	12.0	1000.0	110.0	.00207	.07	113.0	.33
214	265	6.0	1600.0	110.0	.00437	6.98	183.5	2.08
214	215	8.0	5200.0	110.0	.00110	5.70	185.4	1.18
223	214	6.0	4500.0	110.0	.00149	6.71	102.8	1.17
214	211	8.0	7300.0	110.0	.00657	47.95	496.1	3.17
212	213	12.0	8300.0	110.0	.00016	1.33	188.3	.53
206	205	12.0	3200.0	110.0	.00158	5.07	658.1	1.87
206	207	12.0	800.0	110.0	.00211	.08	151.8	.43
208	206	12.0	1600.0	110.0	.00418	6.69	1111.9	3.16
209	208	10.0	3400.0	110.0	.00862	29.31	1017.4	4.16
211	209	8.0	2200.0	110.0	.03306	72.72	1160.2	7.47
205	203	18.0	1700.0	110.0	.00014	.23	505.1	.64
203	204	12.0	1800.0	110.0	.00207	.37	379.3	1.08
201	204	16.0	7600.0	110.0	.00051	3.85	757.3	1.21
203	202	8.0	5400.0	110.0	.00033	1.77	96.6	.62
202	201	6.0	1000.0	110.0	.00063	.63	64.4	.73
201	200	8.0	1200.0	110.0	.00004	.05	32.2	.21
171	118	12.0	1798.0	110.0	.00029	.52	262.8	.75
123	171	12.0	2.0	110.0	.00078	.00	450.0	1.28
101	172	8.0	6298.0	110.0	.00521	32.84	430.9	2.75
172	102	8.0	2.0	110.0	.00404	.01	375.3	2.40
124	174	6.0	4498.0	110.0	.00051	2.32	57.6	.66

FROM	TO	DIA (IN)	LENGTH (FT)	C	HEAD LOSS/FT	HEAD LOSS (FT)	FLOW (GPM)	VELOCITY (FPS)
174	100	6.0	2.0	110.0	0.00000	0.00	2.0	0.00
279	222	8.0	8998.0	110.0	.00044	3.98	113.5	.72
223	279	8.0	2.0	110.0	.00084	.00	102.7	1.23
217	290	8.0	3000.0	110.0	0.00000	0.02	0.0	3.00
212	216	12.0	8700.0	110.0	.00000	.02	19.0	.05
212	210	12.0	7900.0	110.0	.00011	.83	151.3	.43
281	248	12.0	850.0	110.0	.00032	.26	258.5	.76
250	282	12.0	650.0	110.0	.00227	1.33	800.0	2.27
283	243	6.0	100.0	110.0	.00115	.12	88.3	1.01
243	284	6.0	2900.0	110.0	.00115	3.34	88.3	1.01
284	283	6.0	100.0	10.0	.09714	9.71	88.3	1.01
510	124	6.0	1.0	110.0	.00111	.00	87.3	.99
511	112	6.0	1.0	110.0	.04541	.05	652.9	7.39
512	114	6.0	1.0	110.0	.00091	.00	78.5	.89
513	111	6.0	1.0	110.0	.12842	.13	1141.2	12.95
514	110	6.0	1.0	110.0	.02415	.02	462.7	5.25
515	129	6.0	1.0	110.0	.00191	.00	117.4	1.33
516	123	6.0	1.0	110.0	.00239	.00	132.7	1.51
517	116	6.0	1.0	110.0	.00597	.01	217.3	2.47
501	211	6.0	1.0	110.0	.04937	.05	681.1	7.73
502	214	6.0	1.0	110.0	.05964	.06	754.3	8.56
503	212	6.0	1.0	110.0	.01503	.02	355.1	4.03
504	223	6.0	1.0	110.0	.00852	.01	263.5	2.99
505	222	6.0	1.0	110.0	.02554	.03	476.9	5.41
506	266	6.0	1.0	110.0	.00256	.00	137.6	1.56
507	218	6.0	1.0	110.0	.00313	.00	153.4	1.74
508	216	6.0	1.0	110.0	.00649	.01	227.4	2.58
526	102	6.0	1.0	110.0	.00702	.01	237.2	2.69
518	101	6.0	1.0	110.0	.01854	.02	401.1	4.55
519	106	6.0	1.0	110.0	.00740	.01	244.1	2.77
103	520	6.0	1.0	110.0	0.00000	0.00	0.0	0.00
523	256	6.0	1.0	110.0	.10489	.10	1023.4	11.62
524	257	6.0	1.0	110.0	.05622	.06	730.0	8.29
525	122	6.0	1.0	110.0	.02500	.02	197.5	2.24

JUNCTION	ELEVATION (FT)	HGL (FT)	PRESSURE (PSI)	NET FLOW INPUT (GPM)	NET FLOW OUTPUT (GPM)	
100	593.0	630.6	16.3			
101	460.0	632.3	74.6			
102	435.0	599.4	71.2			
103	432.0	570.1	60.7		61.2	OUTPUT
105	430.0	567.0	59.3		100.6	OUTPUT
106	360.0	571.5	91.6		102.6	OUTPUT
107	390.0	571.4	78.5		55.6	OUTPUT
108	410.0	561.1	65.4		61.2	OUTPUT
109	410.0	561.2	65.5		61.2	OUTPUT
110	410.0	558.7	64.4			
111	380.0	489.8	47.6		574.4	OUTPUT
112	370.0	423.3	23.1			
113	368.0	387.3	8.4	531.3		CONSTANT HEAD
114	375.0	488.6	49.2			
115	310.0	483.6	75.2		77.8	OUTPUT
116	300.0	481.4	78.6			
117	300.0	465.3	71.6		73.4	OUTPUT
118	300.0	473.9	75.3		262.8	OUTPUT
119	285.0	464.3	77.6		213.8	OUTPUT
120	280.0	453.7	75.2		213.8	OUTPUT
121	290.0	455.1	71.5		213.8	OUTPUT
122	290.0	459.0	73.2			
123	320.0	474.4	66.9			
124	455.0	632.9	77.0			
125	355.0	486.8	57.1		77.8	OUTPUT
171	320.0	474.4	66.9		187.2	OUTPUT
172	435.0	599.4	71.2		55.6	OUTPUT
174	540.0	630.6	39.2		57.8	OUTPUT
200	5.0	221.0	93.5		32.2	OUTPUT
201	5.0	221.0	93.5		32.2	OUTPUT
202	5.0	221.6	93.8		32.2	OUTPUT
203	5.0	223.4	94.6		32.2	OUTPUT
204	5.0	222.8	94.3		1136.6	OUTPUT
205	15.0	223.6	90.3		150.0	OUTPUT
206	170.0	228.7	25.4		322.0	OUTPUT
207	150.0	228.6	34.0		151.8	OUTPUT
208	196.0	235.4	17.1	34.5		CONSTANT HEAD
209	200.0	264.7	28.0		151.8	OUTPUT
210	30.0	379.1	151.1		151.8	OUTPUT
211	145.0	337.4	83.3			
212	125.0	379.9	113.4			
213	349.0	378.6	12.8		188.3	CONSTANT HEAD
214	145.0	385.4	104.1			
215	145.0	379.7	101.0		[illegible]	OUTPUT
216	150.0	379.9	99.5			
217	8.0	377.9	160.3		[illegible]	OUTPUT
218	210.0	380.5	73.3			
219	300.0	388.1	38.1		140.2	OUTPUT
220	341.0	385.6	19.3		416.1	CONSTANT HEAD
221	225.0	394.4	73.3		140.2	OUTPUT

JUNCTION	ELEVATION (FT)	HGL (FT)	PRESSURE (PSI)	NET FLOW INPUT (GPM)	NET FLOW OUTPUT (GPM)	
222	205.0	392.8	81.3			
223	70.0	392.1	139.5			
224	235.0	386.5	65.6		118.0	OUTPUT
225	220.0	380.7	69.6		118.0	OUTPUT
226	220.0	376.4	67.7		118.0	OUTPUT
227	200.0	375.6	76.0		118.0	OUTPUT
228	220.0	376.2	67.6		118.0	OUTPUT
229	220.0	373.6	66.5		118.0	OUTPUT
230	220.0	381.4	69.9		118.0	OUTPUT
231	5.0	226.7	96.0		177.2	OUTPUT
232	60.0	229.1	73.2		177.2	OUTPUT
233	130.0	236.3	46.0		44.8	OUTPUT
234	110.0	238.6	55.7		44.8	OUTPUT
235	75.0	228.4	66.4		177.2	OUTPUT
236	55.0	227.8	74.8		177.2	OUTPUT
237	93.0	228.7	58.7		177.2	OUTPUT
238	111.0	239.0	55.4		177.2	OUTPUT
239	93.0	239.6	63.5		177.2	OUTPUT
240	196.0	236.0	17.3	384.0		CONSTANT HEAD
241	110.0	249.7	60.5		177.2	OUTPUT
242	100.0	238.5	60.0		177.2	OUTPUT
243	75.0	246.6	74.3		44.8	OUTPUT
244	20.0	259.8	103.8		44.8	OUTPUT
245	162.0	280.8	51.4		60.0	OUTPUT
246	20.0	287.9	116.2		44.8	OUTPUT
247	20.0	291.7	117.6		44.8	OUTPUT
248	95.0	294.1	86.2		44.8	OUTPUT
249	187.0	389.0	87.4		60.0	OUTPUT
250	205.0	389.0	79.7		60.0	OUTPUT
252	223.0	448.4	97.6		60.0	OUTPUT
253	200.0	303.3	44.7		60.0	OUTPUT
254	425.0	458.9	14.7			
255	400.0	502.3	44.3		164.0	OUTPUT
256	395.0	533.3	59.9			
257	405.0	502.7	42.3			
258	462.0	505.1	19.5		218.8	OUTPUT
259	458.0	505.2	20.4	780.7		CONSTANT HEAD
265	105.0	378.4	118.4		222.8	OUTPUT
266	235.0	384.7	64.8			
267	235.0	384.6	64.8		118.0	OUTPUT
268	112.0	241.9	56.0		177.2	OUTPUT
279	70.0	392.1	139.5		47.2	OUTPUT
281	155.0	294.4	60.4	208.5		CONSTANT HEAD
282	155.0	387.1	100.5		820.3	OUTPUT
283	50.0	246.7	85.2			
284	50.0	256.4	89.4			
290	65.0	377.9	135.5			
510	455.0	632.9	77.0	87.5		CONSTANT HEAD
511	370.0	423.3	23.1	650.9		CONSTANT HEAD
512	375.0	488.6	49.2	78.5		CONSTANT HEAD

JUNCTION	ELEVATION (FT)	HGL (FT)	PRESSURE (PSI)	NET FLOW INPUT (GPM)	NET FLOW OUTPUT (GPM)	
513	380.0	490.0	47.0	1141.8		CONSTANT HEAD
514	410.0	558.7	64.4	402.7		CONSTANT HEAD
515	410.0	561.2	65.5	117.[illegible]		CONSTANT HEAD
516	320.0	474.4	66.9	132.7		CONSTANT HEAD
517	300.0	481.[illegible]	73.6	217.3		CONSTANT HEAD
501	145.0	337.5	83.3	681.1		CONSTANT HEAD
502	145.0	[illegible]	104.1	754.3		CONSTANT HEAD
503	125.0	379.3	110.4	358.1		CONSTANT HEAD
504	70.0	392.1	139.5	263.5		CONSTANT HEAD
505	205.0	392.8	81.3	476.9		CONSTANT HEAD
506	235.0	384.7	64.6	137.6		CONSTANT HEAD
507	210.0	380.3	73.3	153.4		CONSTANT HEAD
508	150.0	379.9	99.5	227.4		CONSTANT HEAD
526	435.0	[illegible]	71.2	237.2		CONSTANT HEAD
518	460.0	632.3	74.6	421.1		CONSTANT HEAD
519	360.0	571.5	91.5	244.1		CONSTANT HEAD
520	430.0	570.1	60.7	0.0		CONSTANT HEAD
523	395.0	533.4	59.9	1023.4		CONSTANT HEAD
524	405.0	502.8	42.5	730.6		CONSTANT HEAD
525	290.0	459.0	73.2	197.[illegible]		CONSTANT HEAD

PUMP CURVE COEFFICIENTS

510	-.471E-22	0.	.213E+03
511	-.997E-05	0.	.563E+02
512	-.410E-04	0.	.513E+02
513	-.913E-04	0.	.228E+03
514	-.415E-03	0.	.236E+03
515	-.492E-02	0.	.218E+03
516	-.127E-02	0.	.173E+03
517	-.228E-02	0.	.285E+03
501	-.942E-04	0.	.238E+03
502	-.995E-04	0.	.209E+03
503	-.148E-03	0.	.275E+03
504	-.756E-03	0.	.375E+03
505	-.121E-03	0.	.211E+03
506	-.964E-03	0.	.165E+03
507	-.109E-02	0.	.196E+03
508	-.155E-02	0.	.312E+03
526	-.428E-03	0.	.188E+03
518	-.194E-03	0.	.203E+03
519	-.114E-02	0.	.303E+03
520	-.132E-02	0.	.110E+03
523	-.357E-04	0.	.174E+03
524	-.105E-03	0.	.153E+03
525	-.411E-03	0.	.185E+03

NODE 240 IS DATUM
70 ITERATIONS REQUIRED
MAXERR= 9.895

APPENDIX D: MAPS

Appendix D consists of maps of the water distribution system, including a set of *original maps* in color, plus several blue line copies. The set consists of two maps: (1) service areas A and B (northern portion of Guam) and (2) service areas C and D (southern portion of Guam). The maps are 1:2400-scale and are intended to be overlaid on 7.5-min USGS quad sheets. The maps are color coded as follows:

———	(black) roads and other features
----	(black) pipes (in areas C and D)
446	(black) node numbers
360	(red) node elevations
8″	(green) pipe diameter
3400'	(green) pipe length
———	(blue) pipes (in areas A and B)
D-14	(orange) well numbers
523	(blue) well *node* numbers

These maps have been transmitted to POD under separate cover.

APPENDIX E: COMPUTER TAPE

This appendix consists of the computer tape of the water distribution model and listing thereof. The tape (volume serial number 536164) was created on a CDC Cyber 175 machine using a 9 tack, 1600 bpi, unlabeled tape with 80 characters per block and EBCDIC character set. It can be read on an IBM computer by specifying:

DCB = (LRECL = 80, RECFM = FB, BLKSIZE = 80)

There are 2082 records on the tape.

The tape contains:

1. The MAPS water distribution main program
2. Subroutine SCAN
3. Subroutine PARA
4. Data file for example problem in Appendix A
5. Data file for subarea AB
6. Data file for subarea C
7. Data file for subarea D1
8. Data file for subarea D4

Also inclosed with the tape is a listing of its contents. The contents of the tape are also stored on the Boeing Computer Services computer under POD account CEJOP1 in the file named GTAPE on archive tape 536232. It can be retrieved with the ARCHIV program:

```
GET,ARCHIV/UN = CEBBLB
ARCHIV
```

it is the 26th file on 536232

The computer tape has been transmitted to POD under separate cover.

PART II: ECONOMIC ANALYSIS OF ALTERNATIVES

1. Introduction

Background

The U. S. Army Engineer Waterways Experiment Station (WES) is providing technical assistance to the U. S. Army Engineer Division, Pacific Ocean (POD), relative to the water supply task of the Guam Comprehensive Study (GCS). In Part I of this report WES analyzed water source and transmission problems on Guam, first with a macroscopic water balance, and then with a mathematical model of the hydraulics of the distribution system. The costs of the alternative water supply plans are developed and presented in this portion of the report.

Estimating the cost of alternative water supply systems is very important to the economic analysis for the GCS water supply task because the benefits, as well as the costs, of alternative plans are directly related to facility costs. According to the Federal Register (44FR72894) "(in absence of marginal cost pricing)...the benefits from a water supply plan shall be measured instead by the resource cost of the alternatives most likely to be implemented in the absence of that plan." The cost data presented in this report will, therefore, be used by Honolulu District personnel for determining both National Economic Development (NED) benefits and costs of water supply facilities as part of the final GCS report or a survey report for a specific project.

In most Corps of Engineers water supply studies, only source, treatment, and long distance transmission facilities need be considered in the economic analysis since distribution systems are usually unaffected by the choice of water source. The situation is considerably more complicated in the case of the Public Utility Agency of Guam (PUAG) water supply systems because the well sources are an integral part of the distribution system. Hence, changes affecting the sizing and construction staging of wells will also affect the sizing, staging, and cost of the distribution piping. Therefore, the cost analysis in this report must include consideration of alternative distribution facilities.

Purpose

The purpose of this work is to determine average annual cost, including capital, operation and maintenance (O&M), and replacement cost, for every major water supply facility, for each alternative plan, for each water use projection. The facilities considered will include dams, wells, treatment plants, and pumping stations as well as major transmission and distribution lines. Costs will not be developed for minor distribution lines (i.e., those unaffected by source selection), valves, and appurtenance and storage tanks.*

Preliminary Designs

In the Master Plan (Barrett, Harris and Associates 1979), the size, year of construction, and first cost (in 1980 dollars) has been prsented for a single plan using groundwater to meet future water requirements. To the extent possible, this information is used in the cost estimates included in this report. The cost estimates in the Master Plan are incomplete in that they do not contain O&M and replacement costs, which can be significant (e.g., pumping at wells). The average annual costs of facilities are also not presented ir the Master Plan.

Costs must also be developed for facilities not included in the Master Plan. The report for the Ugum River Interim Study (Honolulu District 1980) includes a detailed estimate of first costs for the Ugum River Dam and cost estimate summaries for the Inarajan River and Ylig River Dams. These costs will be used in this report, except for the cost of "Water Treatment Works" (which includes pumping stations and some water and sewer lines). An estimate is made of O&M and replacement costs for these dams in the Ugum River Report.

The remainder of the costs used in this report were generated using the Methodology for Areawide Planning Studies (MAPS) computer program developed at WES. Documentation of the costs functions used

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* In the Master Plan, storage tanks are referred to as "reservoirs." Because of possible confusion between this use of the word "reservoir" and its use to describe surface impoundments (dams), the less ambiguous terms "storage tank" and "dam" are used in this report.

in MAPS is given in EM 1110-2-502. The functions were modified based on costs presented in the Master Plan to account for local conditions on Guam.

Definition of Alternatives

In this report water supply cost estimates are developed for five types of alternatives based on the source used as defined below.

Alternative Type	Source
1	Groundwater development plus Navy
2	Groundwater development only
3	Groundwater and Ugum River development
4	Ugum River and Inarajan River dams
5	Ugum River development plus Navy

Three sets of cost estimates are presented for each of the five types of alternatives. These estimates are based on the three levels of projected water use utilized for the water balance analysis presented in Part I. (See Part I for definition of "Low," "Medium," and "High" water use.) Alternatives are referred to in this report using the plan type and use projection. For example, plan type 3 under the high-use projection is called 3-H.

If present water use rates continue, the high projections will be applicable. The medium projection can be reached by reducing unaccounted for water. This would include leak detection and repair, increased metering, and meter testing. The low projection can be reached, but only through widespread installation of water-saving devices and major changes in the water use habits of consumers. In the absence of a major educational campaign and a significant increase in the price of water, both are considered highly unlikely.

The ratios of the different water use rates in the year 2035 are shown below.

Water Use Projection	Relative Water Use Low	Med	High
Low	1.00	1.22	1.63
Med	0.82	1.00	1.34
High	0.61	0.75	1.00

The values given above are not based on a detailed study of conservation measure effectiveness of Guam, but merely represent a reasonably broad range of values selected to cover possible variations in water use in order to determine the sensitivity of costs to water use.

If this study proceeds beyond reconnaissance, a detailed evaluation of conservation effectiveness must be made, in accordance with the conservation procedure manual (IWR CR80-1), to accurately forecast water use for a specific set of conservation measures. While the water use reductions utilized in this report are not necessarily identical with those that might be determined in a later stage of this study, development of costs for three use rates is an important step in developing a foregone cost function (as shown in Figure 3-2 of ETL 110-2-259, "Interim Guidance on Use of MAPS Computer Program for Water Supply and Conservation Studies").

Effects of Use Reduction

The water supply facility size and construction staging data given in the Master Plan and the Ugum River Report correspond roughly to the high water use projection. Since conservation must be considered as an alternative to construction, it is necessary to ascertain the effect of water use reduction on construction. There are three possibilities: (1) reduction of size, (2) delay of construction, or (3) some combination of both. The case in which the facility is not built at all is obviously the limiting case (i.e., size = 0 or year built is outside of planning horizon). In the Master Plan and the Ugum River Report, facilities were planned to develop the source in the optimal manner or to transport water to meet the ultimate demand whenever it might occur. Therefore, the size of the recommended facilities selected in the above reports will generally not be altered in this report. Instead, the year in which the facility is to be built will be adjusted to account for reduction in water use. A few minor exceptions (e.g., Ugum River pipeline) are discussed later in the report.

Naming Conventions

Each of the alternative plans is assigned a name based on the type of plan and the water use projection (e.g., alternative 3-H is the Ugum River Dam supplemented by groundwater for the high water use projection). For each type of alternative (i.e., 1, 2, 3, 4, 5), the facilities are generally the same for each water use projection (i.e., high, med, low), but the staging of construction is different. The facilities associated with each type of alternatives are shown in Figures 1-1 and 1-2 and the facilities making up each plan are described in Tables 1-1 through 1-4. Each facility is assigned a name for the GCS (e.g., T-1 is transmission project 1). Each of these facilities actually consists of several "projects" described in the Master Plan (e.g., T-1 consists of A-5, 6, 9, and AB-1, 2, 3). These relationships are described in the above-referenced tables. The abbreviations WTP and BPS are used to indicate water treatment plants and booster pumping stations, respectively.

In the tables, the facility name consists of a prefix for the type of facility followed by a number. The prefixes are defined below:

Prefix	Meaning
S	Source Project
T	Transmission Project
P	Pump Project
M	Miscellaneous Project

The locations of some of the major projects are shown in Figures 1-1 and 1-2.

Note that the facilities required for type 1 and type 2 plans are virtually identical. The main difference between the plans is that, for type 1, the Navy source will supplement the northern lens groundwater sources, delaying much of the construction significantly and eliminating the need for the Cross-Island pipeline (T-3) completely.

Similarly, the type 3, 4, and 5 alternatives are all based on

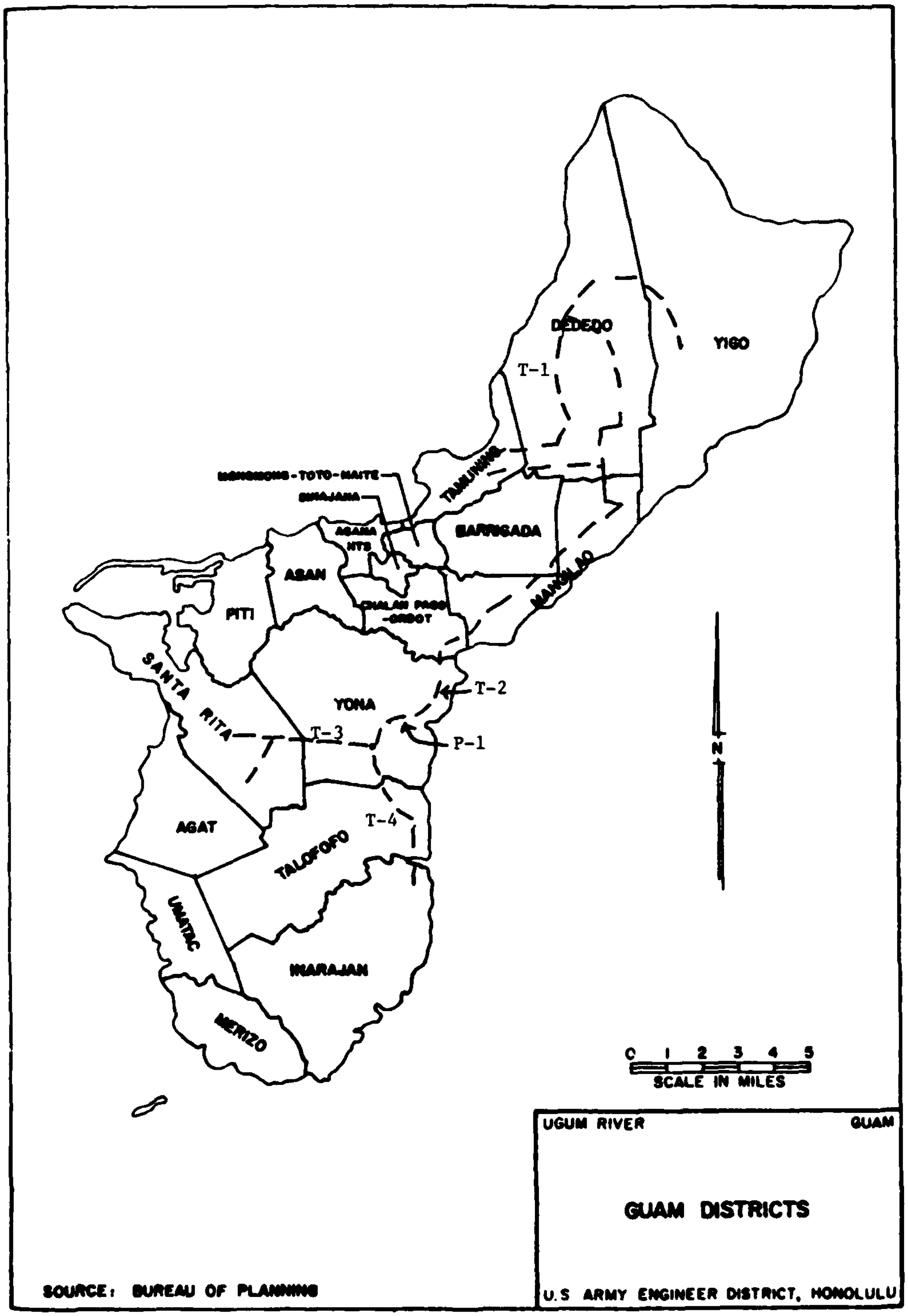

Figure 1-1. Location of Transmission Lines for
Alternative Types 1 and 2

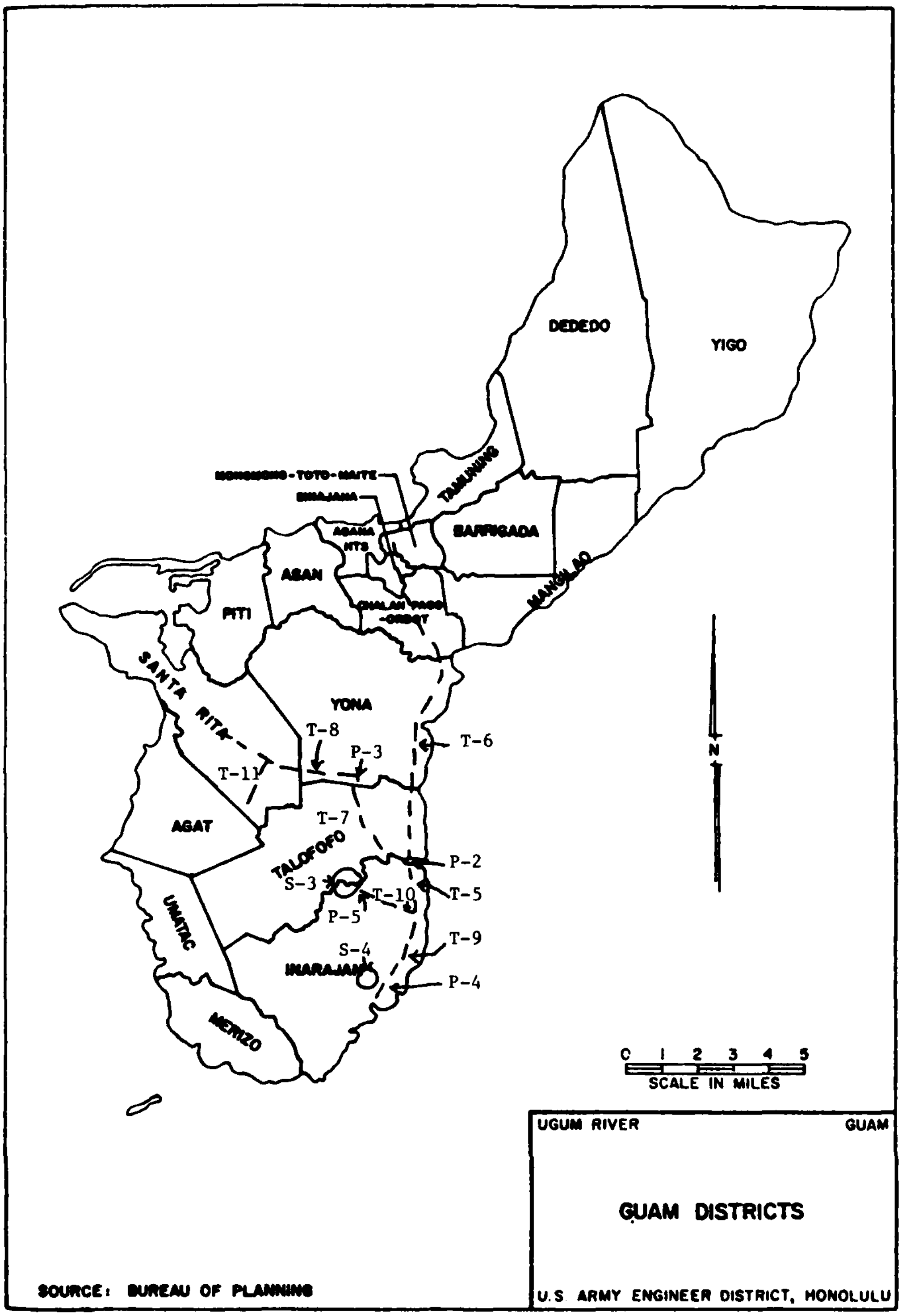

Figure 1-2. Location of Transmission Lines for Alternative Types 3, 4, and 5

Table 1-1

Alternative Types 1 and 2

Facility Name	Name in Master Plan	Description
S-1	AW-1; BW-1	Northern lens wells
S-2	--	Purchase of military water
T-1	A-5,6,9; AB-1,2,3	Major transmission lines from northern lens wells to major use areas
T-2	B-23,24; BD-1; D-17,19	Major transmission lines connecting service area B (Mangilao) with service area D (Yona-Windward Hills)
T-3	CD-1; D-13,16	Cross Island Pipeline (2 only)
T-4	D-9,10,11	Major transmission lines connecting Windward Hills to Talofofo Bay
T-11	C-4,5	Lines from Sinifa to Santa Rita and Santa Rosa (2 only)
M-1	ABM-2,3	Typhoon proofing and backup generators for wells
P-1	DPS-1,2	Pumping stations from Brigade to Sinifa

Table 1-2

Alternative Type 3

Facility Name	Name in Master Plan	Description
S-1	AW-1; BW-1	Northern lens wells
S-3	--	Ugum River Dam, Malojloj WTP
T-1	A-5,6,9,; AB-1,2,3	Major transmission lines from northern lens wells to major use areas
T-5	--	Transmission line connecting Malojloj WTP to Talofofo Bay BPS
T-6	--	Transmission line connecting Talofofo Bay BPS to Agana
T-7	--	Transmission line connecting Talofofo Bay BPS to Windward Hills BPS
T-8	--	Transmission line connecting Windward Hills BPS to Sinifa
T-10	--	Ugum River Raw Water Line
T-11	C-4,5	Transmission line connecting Sinifa to Santa Rita and Santa Rosa
P-2	--	Talofofo Bay BPS
P-3	--	Windward Hills BPS
P-5	--	Raw Water Pumping from Ugum

Table 1-3

Alternative Type 4

Facility Name	Name in Master Plan	Description
S-3	--	Ugum River Dam, Malojloj WTP
S-4	--	Inarajan River Dam, and WTP
T-5	--	Transmission line connecting Malojloj WTP to Talofofo Bay BPS
T-6	--	Transmission line connecting Talofofo Bay BPS to Agana
T-7	--	Transmission line connecting Talofofo Bay BPS to Windward Hills BPS
T-8	--	Transmission line connecting Windward Hills BPS to Sinifa
T-9	--	Inarajan-Malojloj Raw Water Line
T-10	--	Ugum River Raw Water Line
T-11	C-4,5	Transmission line connecting Sinifa to Santa Rita and Santa Rosa
P-2	--	Talofofo Bay BPS
P-3	--	Windward Hills BPS
P-4	--	Inarajan Raw Water Pumping Station
P-5	--	Raw Water Pumping from Ugum

Table 1-4

Alternative Type 5

Facility Name	Name in Master Plan	Description
S-3	--	Ugum River Dam, Malojloj WTP
T-5	--	Transmission line connecting Malojloj WTP to Talofofo Bay BPS
T-6	--	Transmission line connecting Talofofo Bay BPS to Agana
T-10	--	Ugum River Raw Water Line
P-2	--	Talofofo Bay BPS
P-5	--	Raw Water Pumping from Ugum

construction of the Ugum River Dam supplemented by other facilities. Water supply from the dam is supplemented under 3 by the northern lens wells, under 4 by the Inarajan Dam, and under 5 by Navy sources.

Overview of Report

The next section of the report focuses on plans for the southwestern river dams since distribution lines from these dams were not discussed in the Master Plan. In subsequent sections, costs are developed for each type of facility. Construction and O&M costs are presented first, followed by the development of average annual costs based on construction staging considerations. The costs of individual types of facilities are then combined to form cost estimates for the alternative plans.

2. Conceptional Design for Southeast Dam Projects

The Master Plan contains descriptions of the facilities required for alternative types 1 and 2. Appendix A from the Financial Analysis portion of the Master Plan is included as Appendix A to this report. While the Ugum River Report contains fairly detailed design information for the Ugum and Inarajan River Dams, there is very little discussion of specific treatment, pumping, and distribution systems required for these projects. Therefore, to equitably compare total project costs among the alternatives, it is necessary to prepare a conceptual design of the system required for alternative types 3, 4, and 5.

In order to correctly size and locate the pipes, pumps, and plants, it was necessary to screen a large number of piping and pumping arrangements to arrive at the least costly. This was accomplished with the aid of the MAPS Computer Program which was developed at WES. The sizes of pipes and pumps determined using MAPS represent virtually optimal sizes as opposed to sizing decisions based on rules-of-thumb.

In this section physical and hydraulic features of alternatives relying upon the southeastern rivers are described. While decisions with regard to size and location of the facilities were based on cost, the costs are generally not presented until Section 3.

Design Flows

The size of transmission facilities depends upon how the water is divided among: (1) the southern portion of the island (i.e., Inarajan, Merizo, Umatac), (2) the Agat-Santa Rita area plus Talofofo, and (3) the northern portion of the island (Yona and beyond). This in turn depends on the yields of the various reservoirs.

The water supply yield (i.e. safe yield minus instream release) for the Ugum River Dam is 9.0 mgd (6246 gpm) and from the Inarajan River Dam is 6.9 mgd (4789 gpm). This results in a total water supply yield from the southeastern dams of 15.9 mgd (11,034 gpm).

Once the yields are known for plans 3 and 5 (9.0 mgd) and 4 (15.9 mgd), it is necessary to divide the flows in the directions described above. This distribution is described for each plan in Table 2-1. Note that the numbers in Table 2-1 do not always agree with the numbers presented in the water balance in Section 2, Part I, of this study. For example, the flow from Village 9 (Yona) to Village 4 (Barrigada) under the low use projection in the year 2035 is 2669 gpm in Figure 2-3, Part I. In Table 2-1, Part II, the flow from Talofofo Bay toward Agana is given as 3789 gpm. The difference is due to water use along the line (Yona, Talofofo). When there are differences, the flows in Table 2-1 are used as the basis for design.

Table 2-1

Flow Distribution for Southeastern Reservoirs

Alternative	Reservoir Yield gpm	To Inarajan and South gpm	To Santa Rita gpm	To Agana gpm	Through Talofofo Bay BPS gpm
3-H	6,246	1000	2327	2,919	5,246
3-M	6,246	698	1759	3,789	5,548
3-L	6,246	543	1421	4,282	5,703
4-H	11,034	1000	2327	7,707	10,034
4-M	11,034	698	1759	8,577	10,336
4-L	11,034	543	1421	9,070	10,493
5-H	11,034	1000	--	10,034	10,034
5-M	6,246	698	--	5,548	5,548
5-L	6,246	543	--	5,703	5,703

The next question concerning flows was whether the transmission line should be designed to meet peak demand or to operate at constant capacity allowing daily fluctuations in use to be dampened out by storage tanks. Since the most efficient way to operate the treatment plant and pumping station is at capacity, the latter approach is desirable. Furthermore, since seasonal fluctuations in use are small, they can be neglected at this stage of planning.

Overview of Southeastern Dam Plans

Plans involving southeastern dams (i.e. alternatives types 3, 4, and 5) have many features in common. The primary differences are that

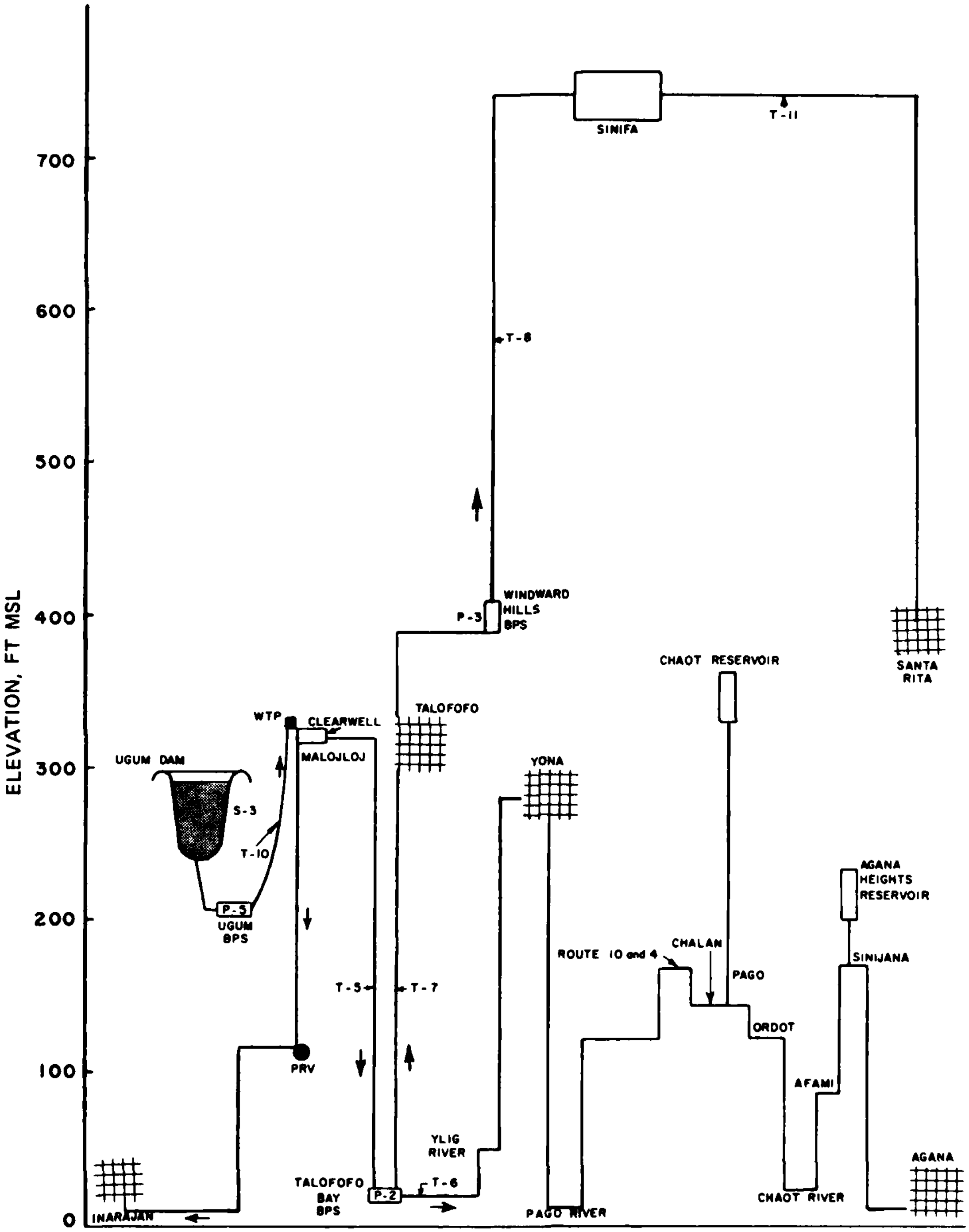

Figure 2-1. Profile of Major Transmission Lines--Plan Type 3

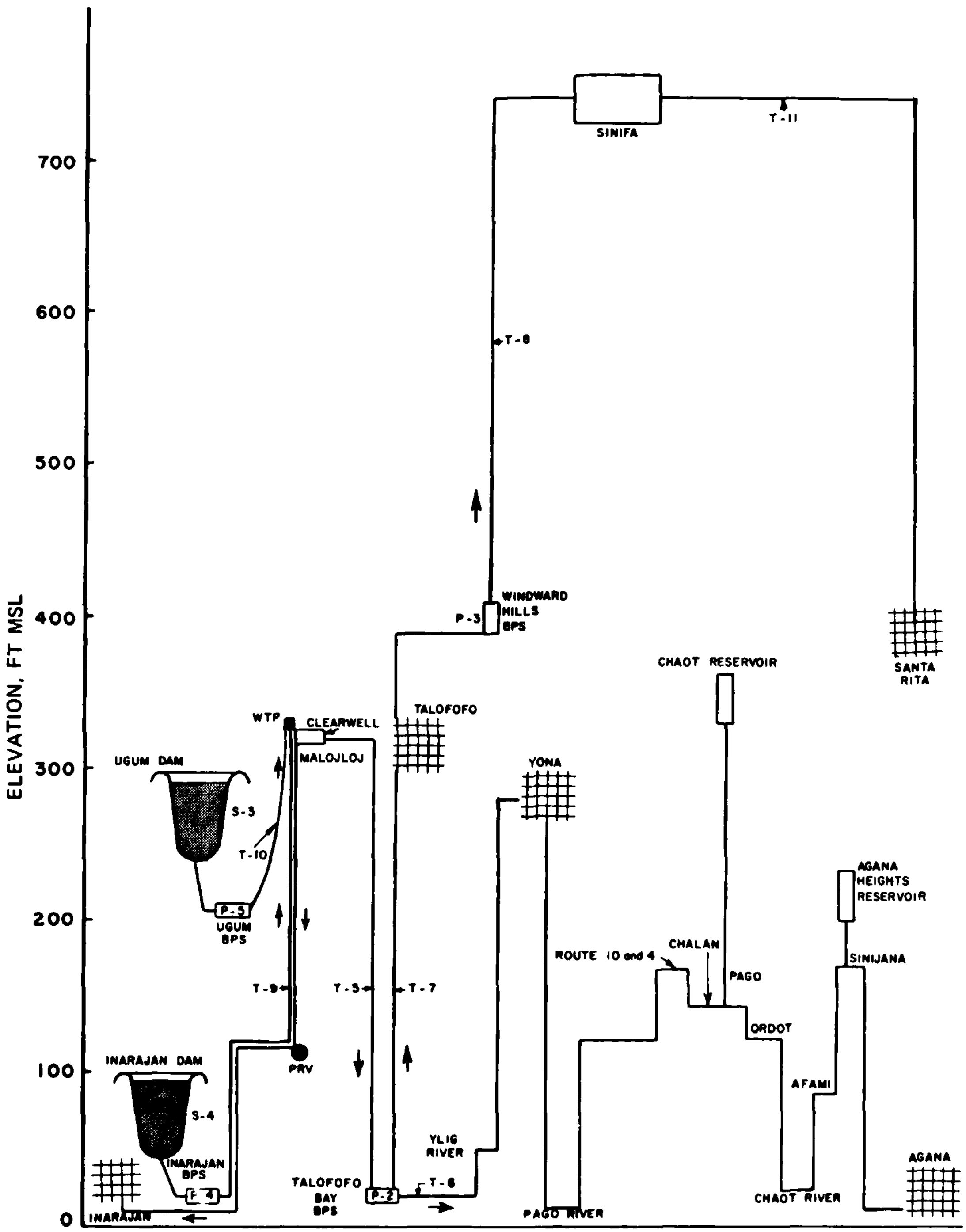

Figure 2-2. Profile of Major Transmission Lines--Plan Type 4

4 and 5-H include the Inarajan Dam and 5 does not include the Cross Island pipeline to Agat-Santa Rita.

The alternatives are shown in profile in Figures 2-1 and 2-2. All include pumping raw water through transmission lines to a central treatment plant and clearwell at Malojloj. The main pumping station is located at Talofofo Bay, the last point before the flow splits to Agana or Santa Rita. An additional pumping station is required at Windward Hills to provide adequate lift to raise water to the Sinifa storage tank (elevation 765 ft). A description of each of the facilities is presented below.

Dams

Hydrologic data and design parameters for the southeastern river dams are contained in Appendixes D and E of the Ugum River study. The reader is referred to that report for details; however, the only changes in facilities recommended in this report are the size and location of raw water pipes and pumps, and the water treatment plant. These are discussed in more detail later.

Raw Water Transmission Lines

Raw water pipes and pumps required by the Ugum River Dam (T-10, P-5) and the Inarajan River Dam (T-9, P-4) were sized using the MAPS pipeline routine, which selects pipe sizes to minimize life cycle costs. The output from these runs is included in Appendix B as an example of output from MAPS. (This is not done for other pipes because of the volume of output.) The results of the design are summarized in Table 2-2. Note that because of the high cost of piping and pumping equipment on Guam, the 24-in.-diameter pipe proved to be optimal instead of the 30-in. pipe recommended in the Master Plan (Appendix B shows its cost to be 5.4 percent greater).

Treatment Facilities

Considerable savings could be realized by constructing and operating a centralized water treatment plant at Malojloj rather than separate plants at each dam. This occurs because of economies of scale that exist in treatment plant construction and operation (i.e. one large plant costs less than two small ones) and because most of the water from

Table 2-2

Hydraulic Design Summary for Raw Water Lines

Facility	Pipe Diameter in.	Flow mgd	Pump Head ft	Plan Type
T-9, P-4	24	9.0	100	3,4,5
T-10, P-5	24	6.9	254	4, 5-H

the Inarajan Dam must pass Malojloj anyway on its way north. Locating the plant on a plateau in Malojloj also makes best use of the elevation head from the Ugum Dam which would be lost if the plant was located at the base of the dam. Water treatment plant costs are shown in Table 2-3.

Table 2-3

Water Treatment Plant Cost

	Flow mgd	Capital (10^6 \$)	O&M (10^3 \$/yr)	Average Annual Cost* (10^3 \$/yr)
Direct Filtration	9.0	2.23	118	339
	15.9	3.27	185	509
Flocculation Clarification Filtration	9.0	4.46	219	661
	15.9	6.52	325	971

* If built in base year.

The treatment train selected in the Ugum River Report consisted of screening, rapid mix, flocculation, clarification, filtration, and chlorination. This is a typical choice for a surface water plant, and while the water quality analysis of the Ugum River listed in Table E-2 of the Ugum River Report indicates that the water is quite clear (highest turbidity = 28 NTU) agricultural development which will adversely affect water quality is expected in the area of the dam. Since much of the suspended matter in the stream is described as silty clay, only some of the material will settle within the reservoir. Without further study it is difficult to determine if conventional treatment or direct filtration will be required. There, cost estimates are presented in Table 2-3 in both levels of treatment.

Distribution System

Distribution for plan types 3, 4, and 5 is significantly different from 1 and 2 in that for 3, 4, and 5 the net flow of water is from south to north. Therefore, the major transmission lines reported in the Master Plan are not relevant to alternatives that include the southeast dams.

Hydraulic design features for each alternative are given in Table 2-4. This includes the size of each transmission line (for which the flow depends on the alternative), capacity, and suction and discharge pressure for each pumping station. Note that the pressures at the suction side of the pumps are positive for all alternatives, and the pressure on the discharge end are not excessive (i.e. always less than 230 psi). It is important to maintain reasonably low discharge pressure so that very thick-walled pipe is not required. The pressures at Ordot (el. 270 ft) and Agana (el. 10 ft) are presented to show that pressures are not excessively low at high elevations or excessively high at low elevations.

In developing the distribution system shown in Figures 1-2, 2-1, and 2-2, every attempt was made to take advantage of existing water distribution lines. This could result in significant savings in the size of pipe required. The most dramatic savings result from using an existing 12-in. line that runs from Malojloj to Agana.

The principal transmission line is the one that connects Talofofo Bay to Agana T-6. This line was sized to carry water to Agana and not to be used as a local distribution line. As such, the pressures at the higher elevations in Chalan Pago and Ordot along Route 4 will be fairly low (approx 20 psi), but adequate to ensure that, in case of a break, water will not leak into this treated water line. This design will result in minimum use of energy. The Chalan Pago-Ordot area will continue to be served by wells and the Chaot storage tank.

Table 2-4

Hydraulic Design Summary for Southeastern River Treated Water Lines

Alternative	Pipe Diameter (in.)				Pump Flows (gpm)		Pumping Suction/ Discharge Pressure (psi)		Pressure (psi)	
	T-5	T-6	T-7	T-8	P-2	P-3	P-2	P-3	Ordot	Agana
3-H	20	14	14	14	5296	2327	124/185	75/211	20	107
3-M	20	16	14	12	5548	1759	123/188	31/218	20	105
3-L	20	18	12	12	5703	1421	122/178	13/207	20	110
4-H	30	20	14	14	10034	2327	126/228	56/211	20	89
4-M	30	24	14	12	10336	1759	126/195	38/218	20	116
4-L	30	24	12	12	10493	1421	126/192	27/207	20	104
5-H	30	24	-	-	10034	-	126/210	-	20	99
5-M	20	18	-	-	5548	-	123/209	-	20	97
5-L	20	18	-	-	5703	-	122/213	-	20	95

As stated earlier, Talofofo Bay was selected as the location of the main pumping plant (P-2) because it is the last point at which a single pumping station could be built before the flow splits to Agat-Santa Rita and Agana. Furthermore, because of the low elevation, there should be no problem in maintaining positive suction pressures and avoiding cavitation.

Because of the economies of scale in pumping station construction, it would have been desirable to construct only one station for all southeastern dam pumping. Unfortunately, some of the flow from the dams in plan types 3 and 4 must be carried to the Sinifa storage tank at elevation 765 ft. To accomplish this in one lift would require costly high pressure pipe. Therefore, a booster station is used in Windward Hills so that the water can be raised in two lifts. With this location, only the water being carried to Agat-Santa Rita receives the extra boost, thus considerable energy is saved.

Transmission line T-11 from Sinifa storage tank to Hyundai and Santa Rosa is not included in Table 2-4. This is because it will be the same for all alternatives (although the years of construction will vary) and is essentially a distribution line sized for fire flow, not a transmission line.

In developing these plans, it is assumed that surface intakes at La Sa Fua River, LaeLae Spring, Geus River, and Siligen Spring will continue to be used. Therefore, as shown in Table 2-1, only 903 gpm is required from the dams for use in the south even at the highest use rate. Thus, the existing 8-in. line from Malojloj to Inarajan should provide adequate flow. The situation under fire flow conditions could be improved by moving the existing pressure-reducing valve closer to Inarajan.

Transmission line T-5 from the Malojloj Treatment Plant clearwell to the Talofofo Bay Pumping Station is sized to conserve much of the head available at Malojloj and minimize pumping energy costs at Talofofo Bay.

Many of the lines in alternatives 3, 4, and 5 are long straight lines crossing several drainage divides. Waterhammer could become a significant problem especially during startup and shutdown of the Talofofo Bay Pumping Station. A detailed waterhammer analysis should be performed during the design phase of the transmission lines. For example, pipeline T-6 will probably require air release valves at Yona and Chalan Pago and pressure relief valves at the Ylig River and Pago River.

No storage tanks are included in this design as they will be the same as in the Master Plan.

In the following section major facility costs are presented for all alternatives.

3. Development of Facility Cost Estimates

Introduction

In this section costs are developed for each facility required under each alternative based on the preliminary designs presented in the previous section and the Master Plan. In Section 4 these costs are combined to determine the costs of the alternative plans.

For a given type of plan, differences in water use are reflected in the year in which a facility is constructed. In general, the analysis shows that most of the facilities will be constructed by the year 2000. This is to be expected since most of the growth in water use will occur by that year. Operations and maintenance (O&M) costs are based on the average flow for a given facility, even though the flow may vary considerably over the life of the facility. Cost at average flow is generally a good indicator of overall O&M cost.

Construction Staging

In most Corps reservoir studies, selection of year of construction is a fairly simple matter as all of the facilities are staged to come on line at the same time. In this study, the Corps facilities are merely one portion of an integrated surface and groundwater development plan. As such, the staging of any facility depends on that of other facilities and the water use rates.

Since well sources can be developed in small increments (approx. 200 gpm per well), there is considerable flexibility in when they can be built. On the other hand, dams and their associated treatment and distribution facilities must be built simultaneously. Therefore, in plans involving Corps dams, the construction year of the reservoir is fixed and staging of the development of wells to supplement the reservoirs is used to account for different water use rates.

The dams are not down sized to account for staging since the storage capacity selected in the Ugum River Report makes best use of the damsite. Because of economies of scale in dam construction, use of a reduced size dam is generally economically inefficient.

For the purpose of this study, construction of the Ugum River Dam would begin in 1990 and would be completed in 1993; and construction of the Inarajan River Dam would begin in 1994 and would be completed in 1997. For amortization calculations, construction costs would occur in 1993 and 1997, respectively, and O&M costs would begin to accrue only after the completion of the dam.

Figure 3-1 shows the average day water use that must be met for each projection as a function of time. Figures 3-2 through 3-16 on the following pages show the construction staging required to provide the needed water for each scenario.

Economic Input Data

Costs presented in this report correspond to 1980 price levels on Guam. This base year was selected since costs reported in the Master Plan are in 1980 dollars. Estimated 1980 costs can be upgraded to 1985 dollars using the ratios of appropriate cost indices for the two years. The following data on price levels were used to develop the MAPS cost estimates:

ENR Construction Cost Index 3200
Electricity 6 to 11.9¢/kwhr
O&M Labor $10/hr
Local Multiplier 1.5

The 1.5 multiplier is used to correct construction costs from the U. S. National Average (i.e. ENR Construction Cost Index = 3200 for 1980) to Guam. Using the ENR Construction Cost Index of 3200 and the 1.5 multiplier, MAPS was able to reproduce costs given in the Master Plan. The O&M labor costs include overhead. The price of electricity was not corrected using the multiplier. Two electrical energy prices were used--6¢/kwhr, which reflects present costs, and 11.9¢/kwhr, which reflects the current cost of producing energy.

For calculating the average annual cost of alternatives, a base year of 1985 is used and costs are amortized over a 50-year period at 7-5/8% interest. The 50-year economic life was selected as reasonable for many of the water supply facilities.

Most of the facilities built during the study period will have useful life remaining at the end of the study period. This can be

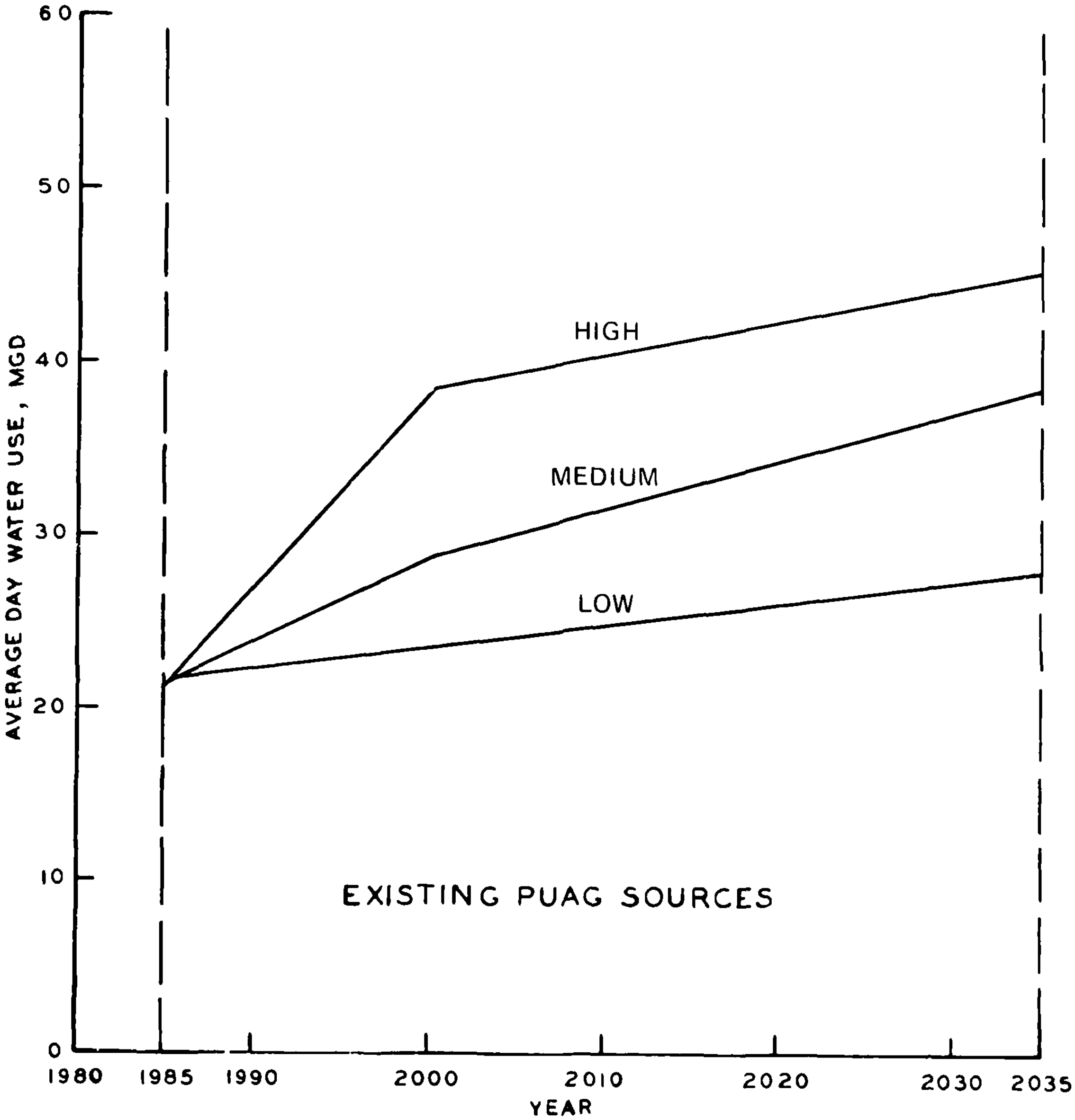

Figure 3-1. Use Projections

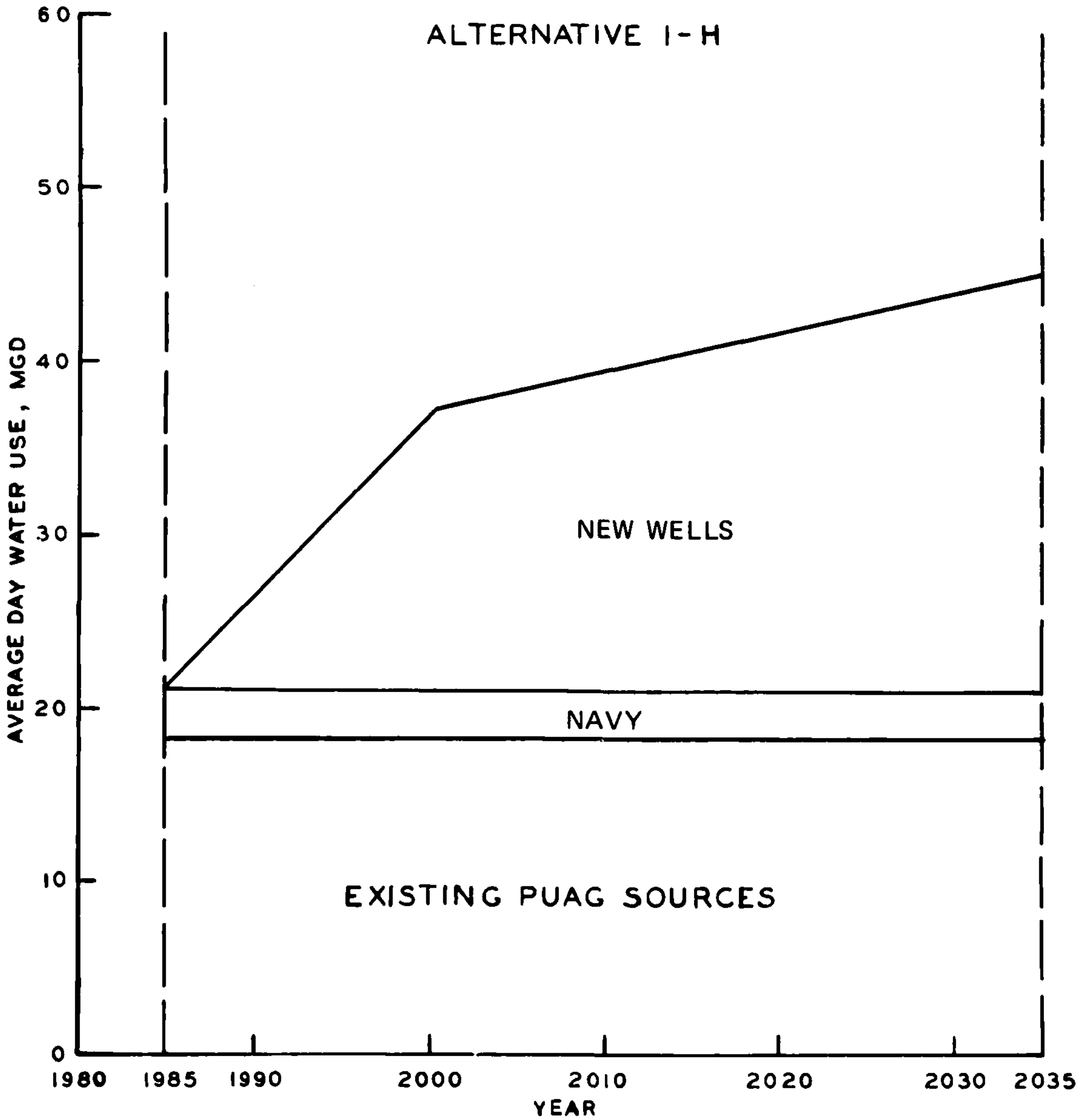

Figure 3-2. Source Staging for Alternative 1-H

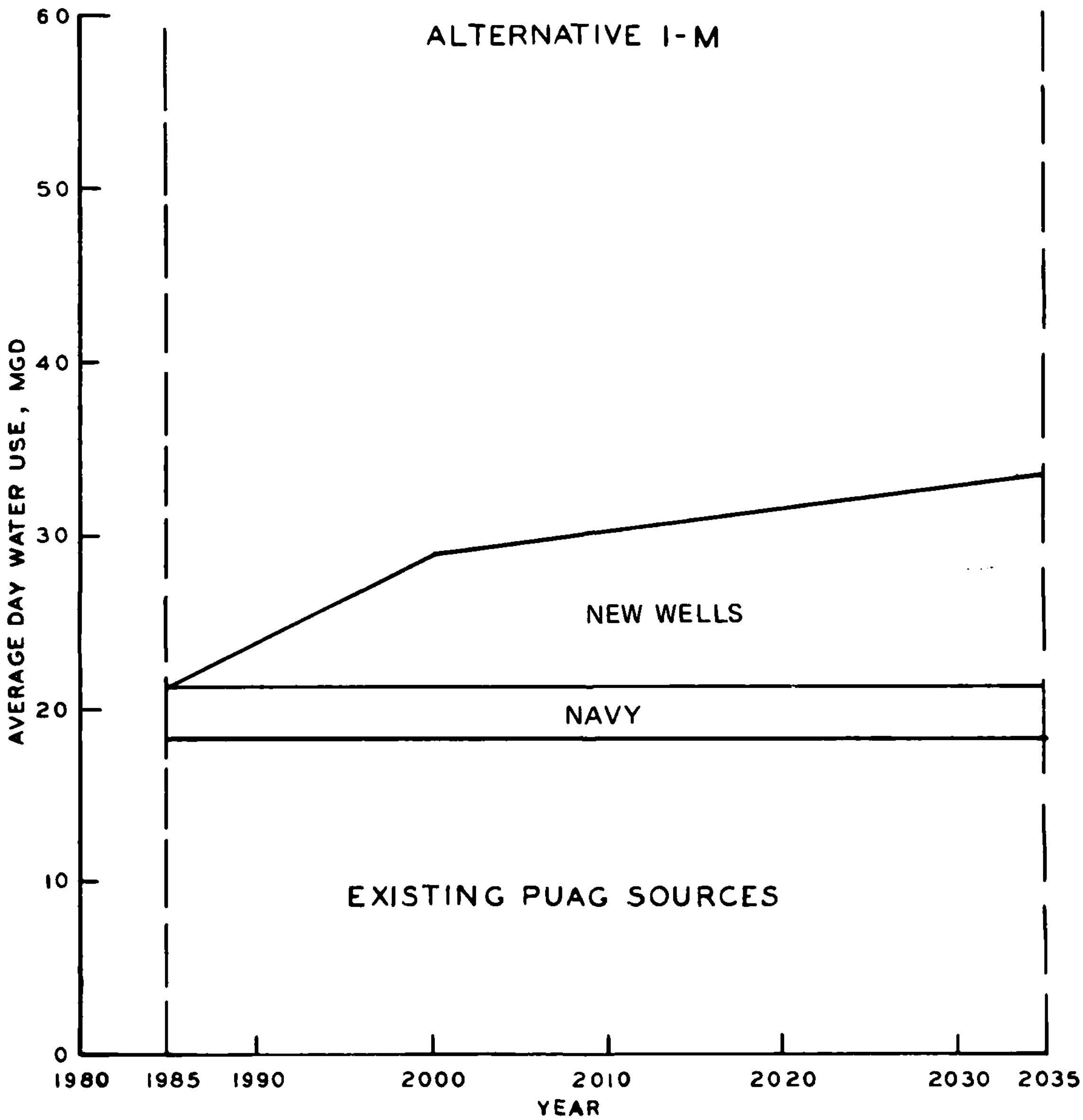

Figure 3-3. Source Staging for Alternative 1-M

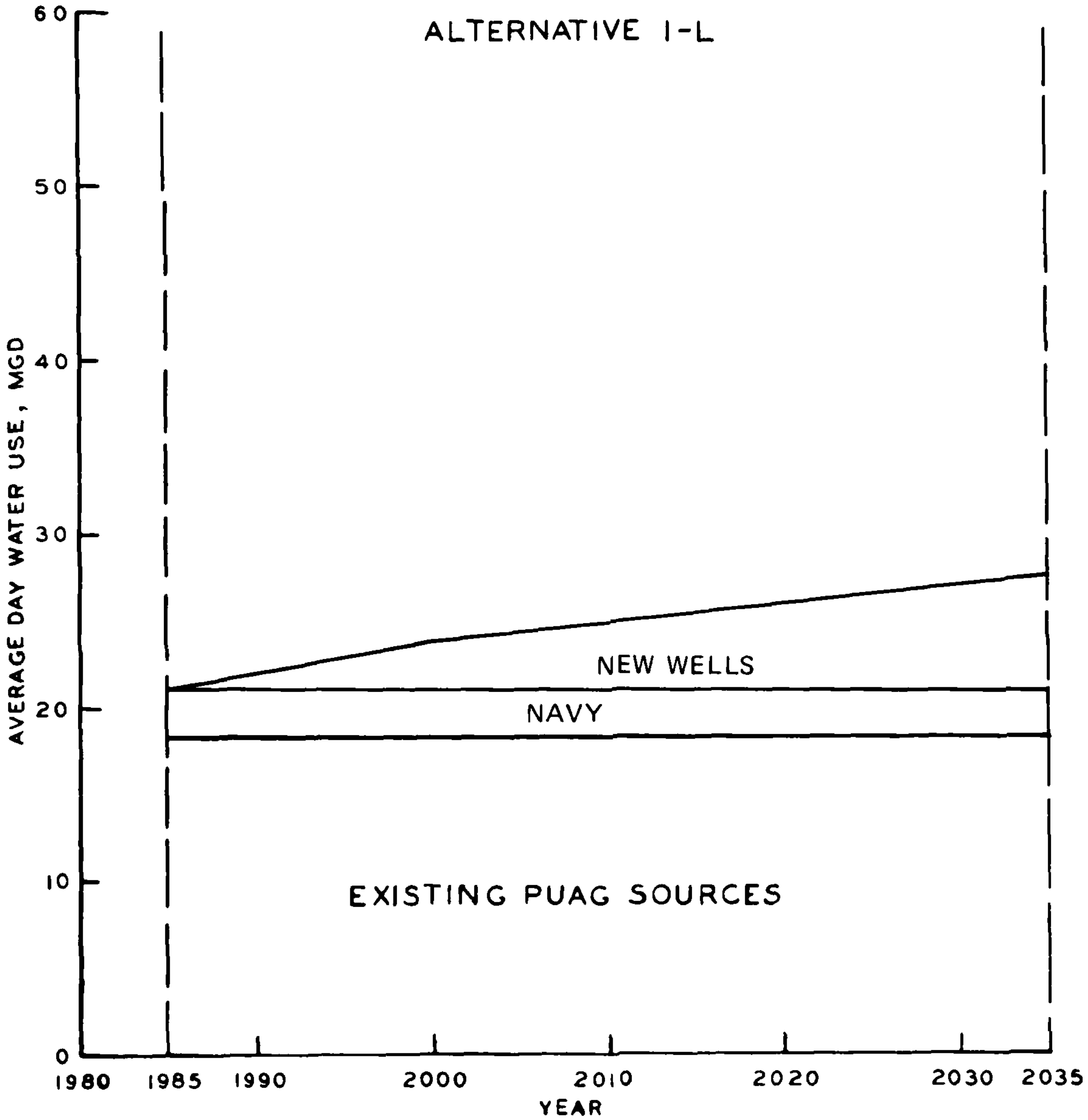

Figure 3-4. Source Staging for Alternative 1-L

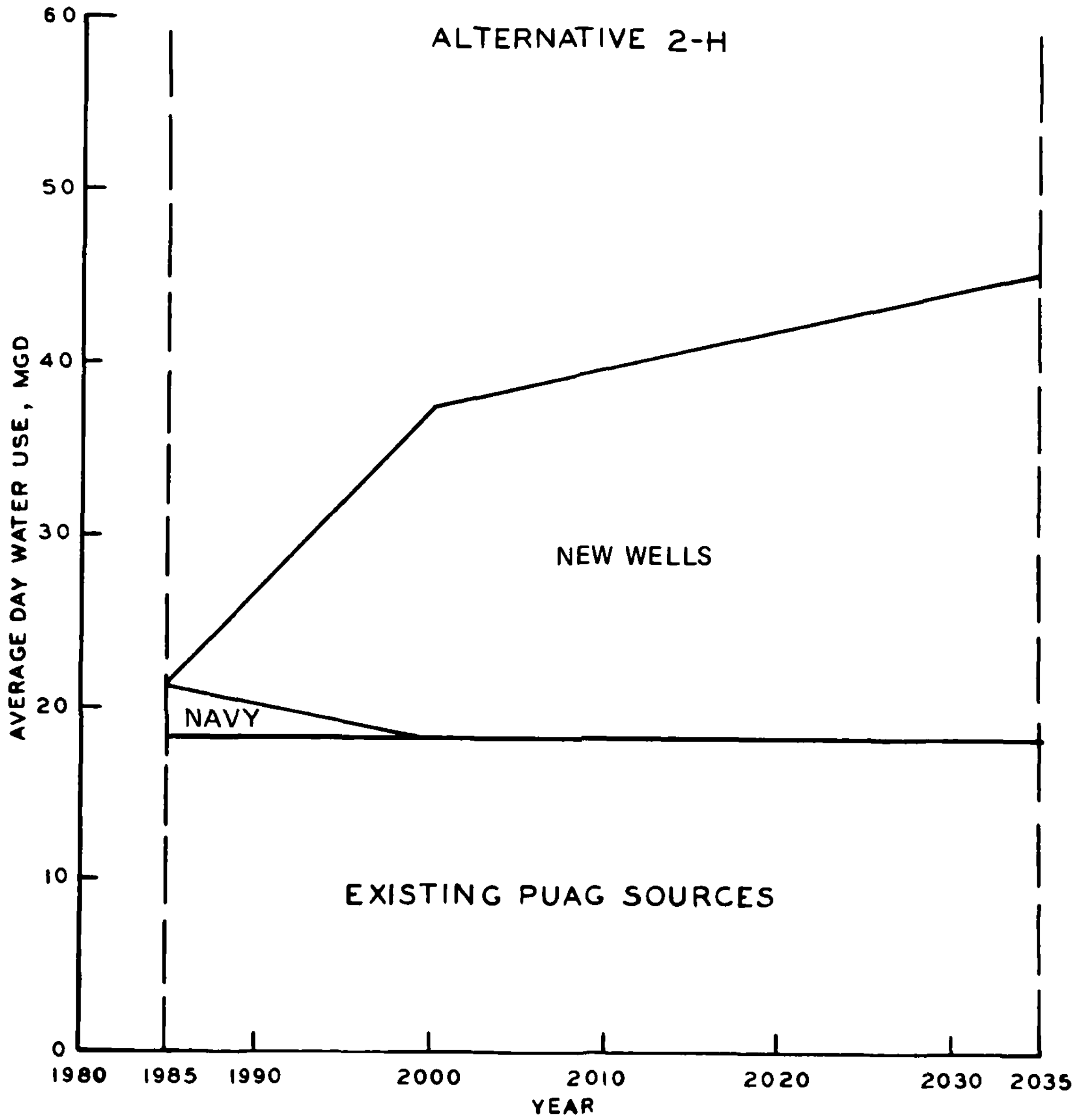

Figure 3-5. Source Staging for Alternative 2-H

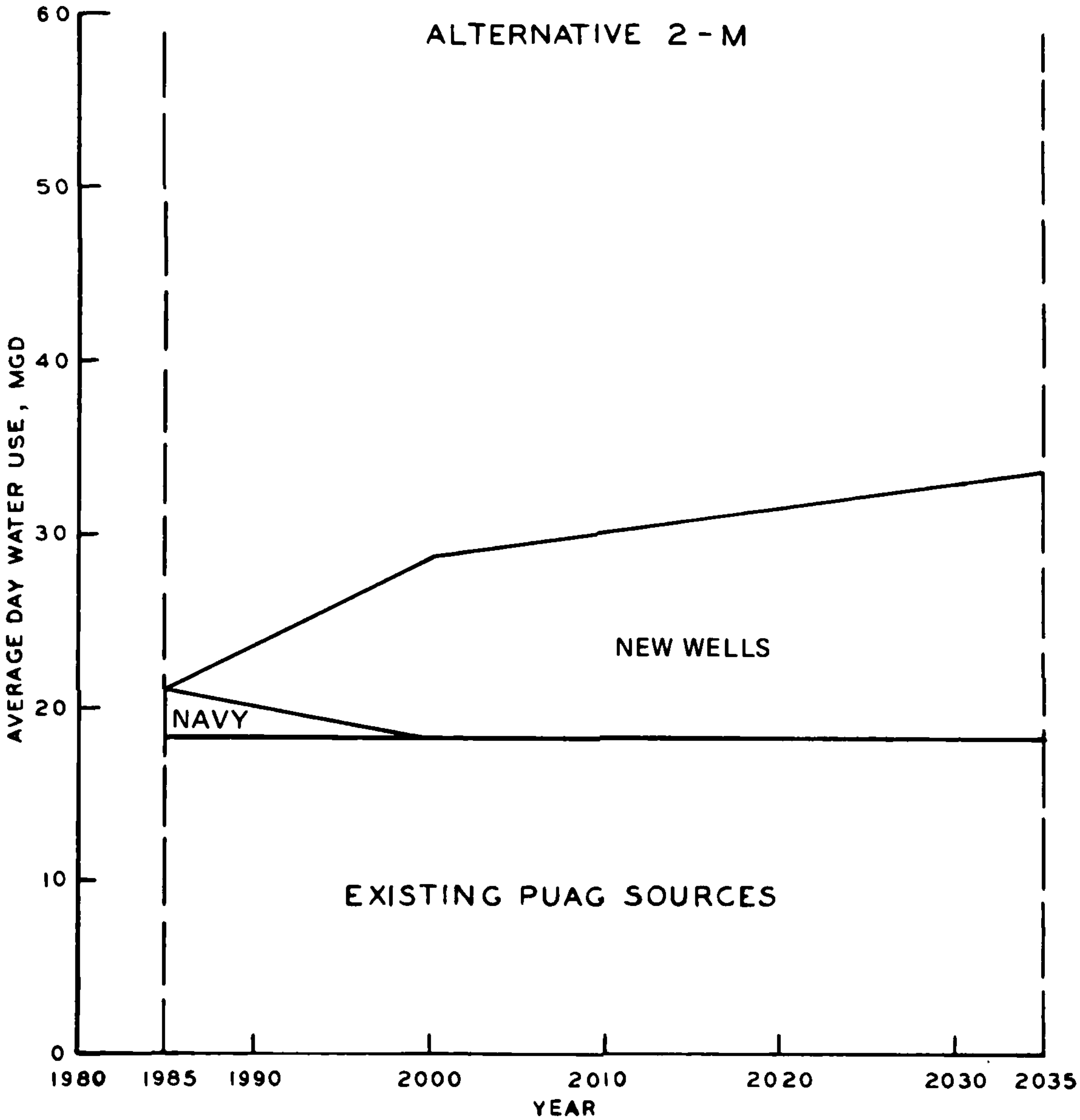

Figure 3-6. Source Staging for Alternative 2-M

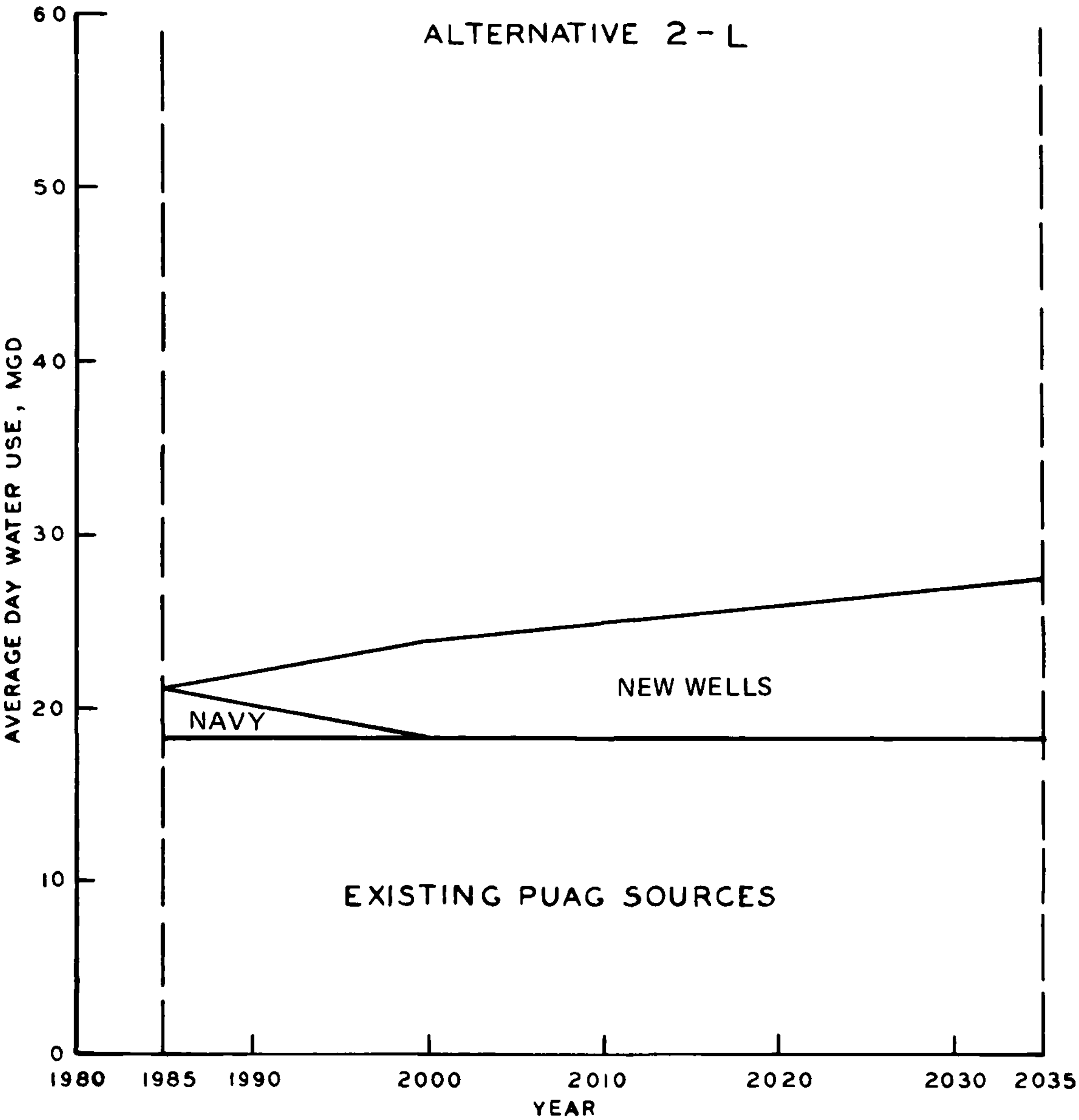

Figure 3-7. Source Staging for Alternative 2-L

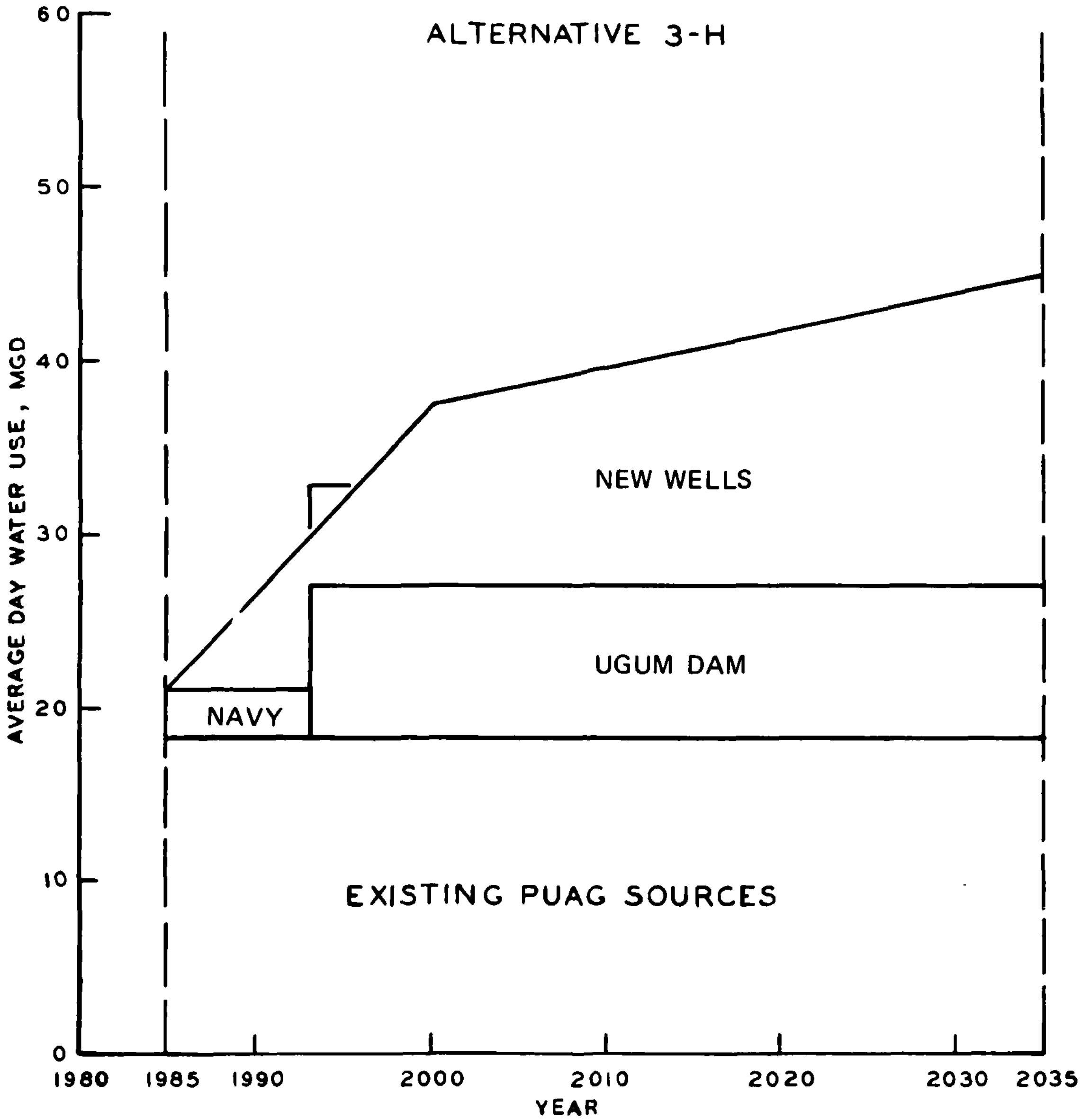

Figure 3-8. Source Staging for Alternative 3-H

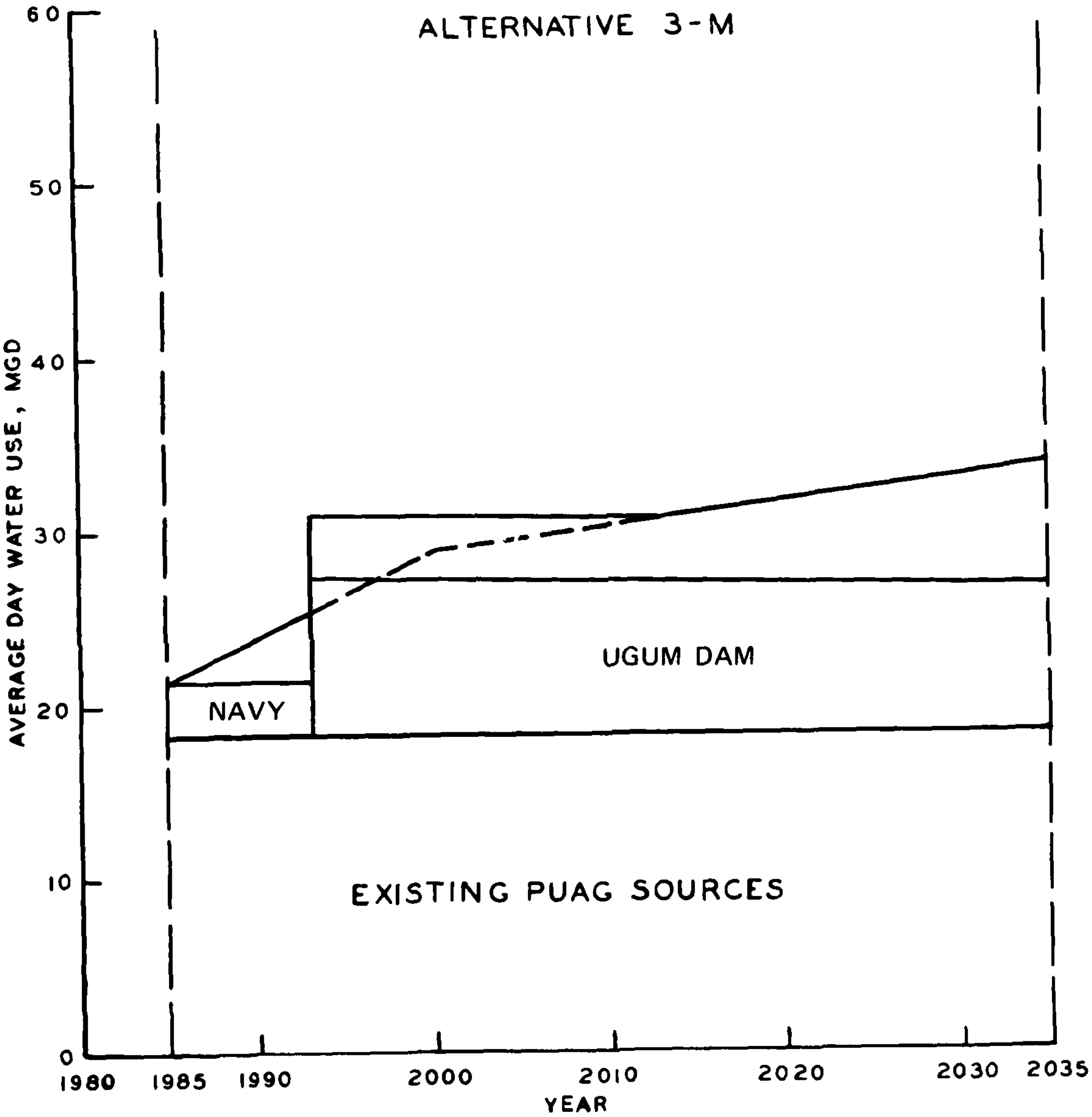

Figure 3-9. Source Staging for Alternative 3-M

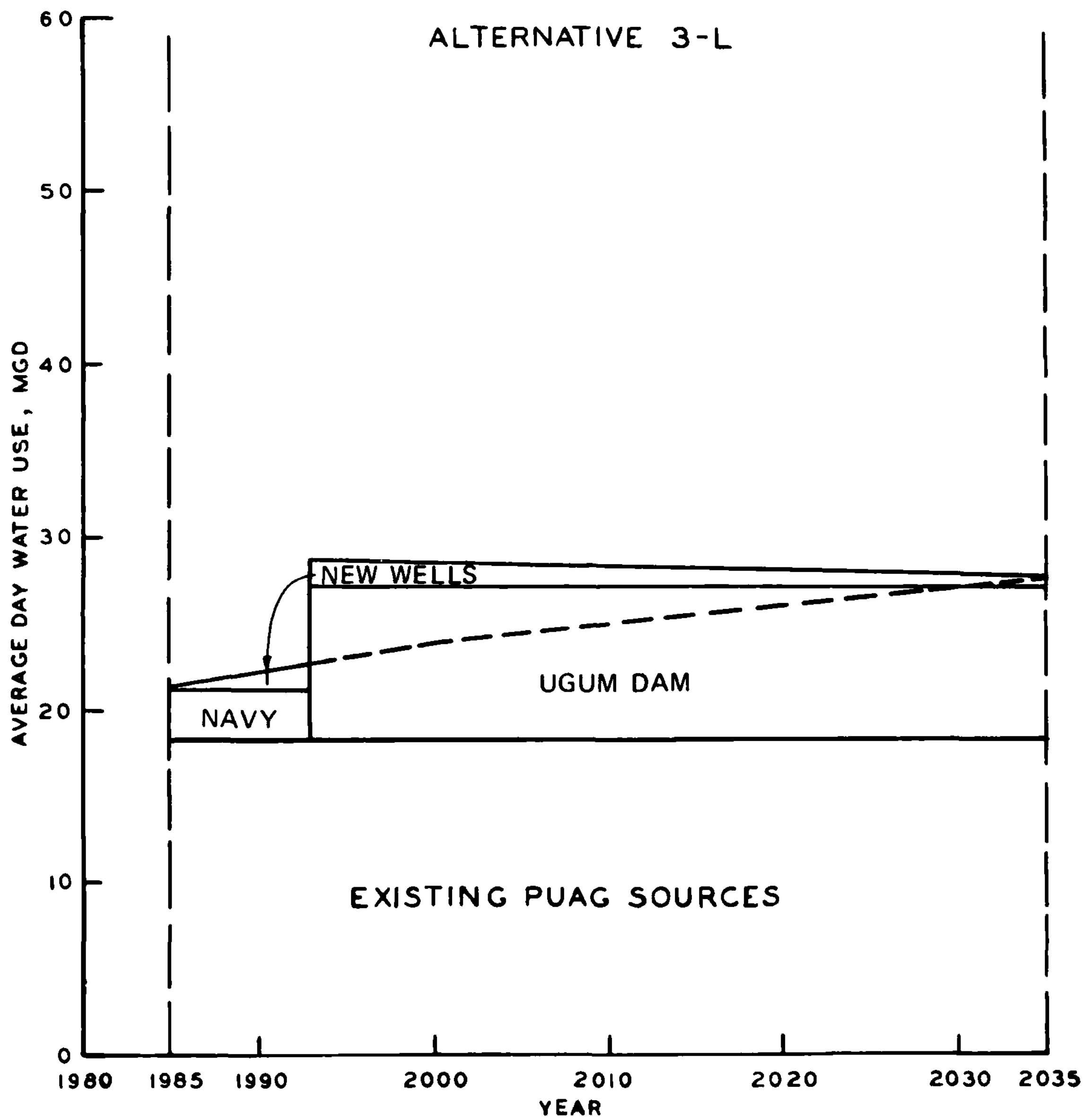

Figure 3-10. Source Staging for Alternative 3-L

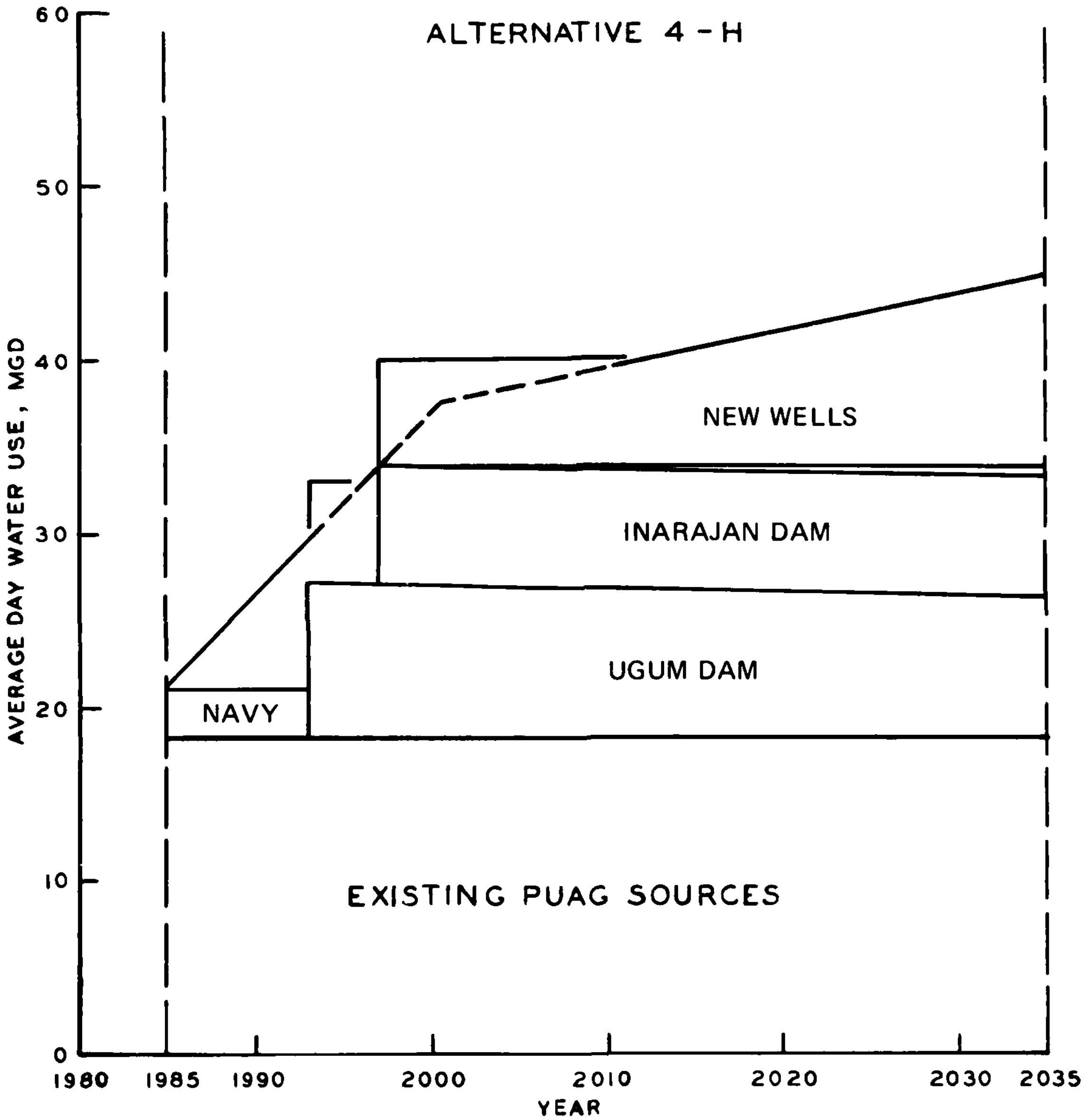

Figure 3-11. Source Staging for Alternative 4-H

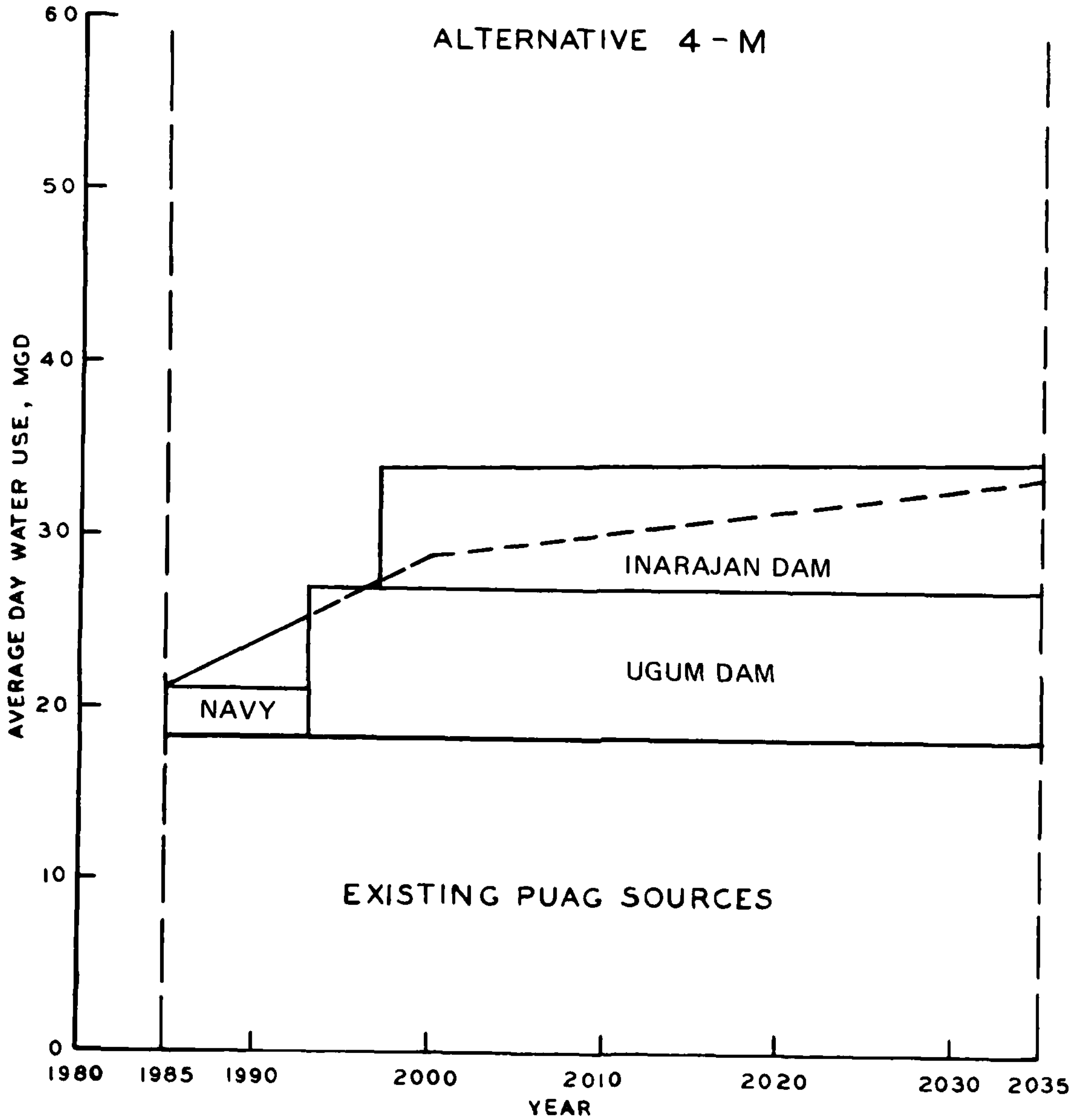

Figure 3-12. Source Staging for Alternative 4-M

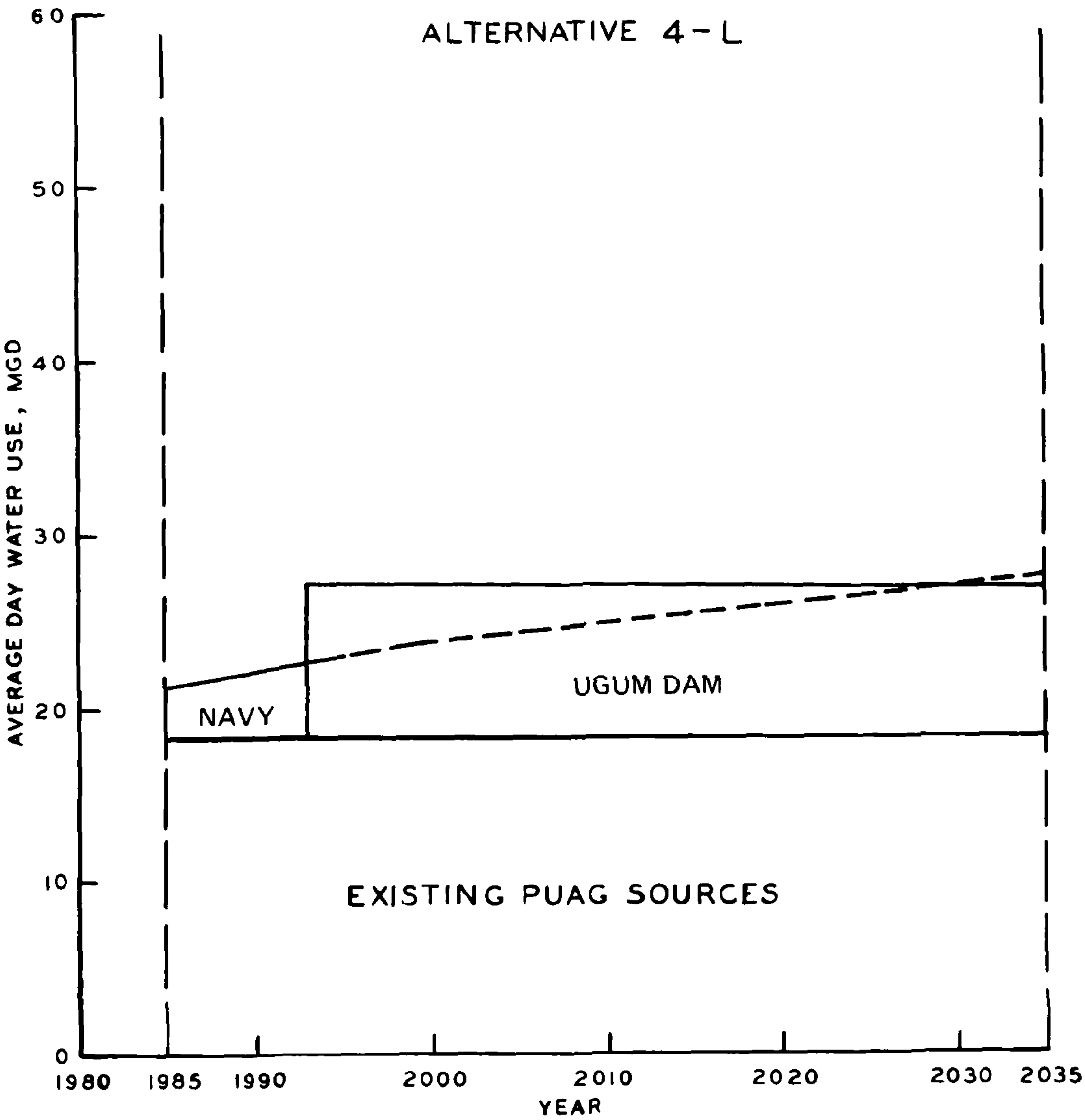

Figure 3-13. Source Staging for Alternative 4-L

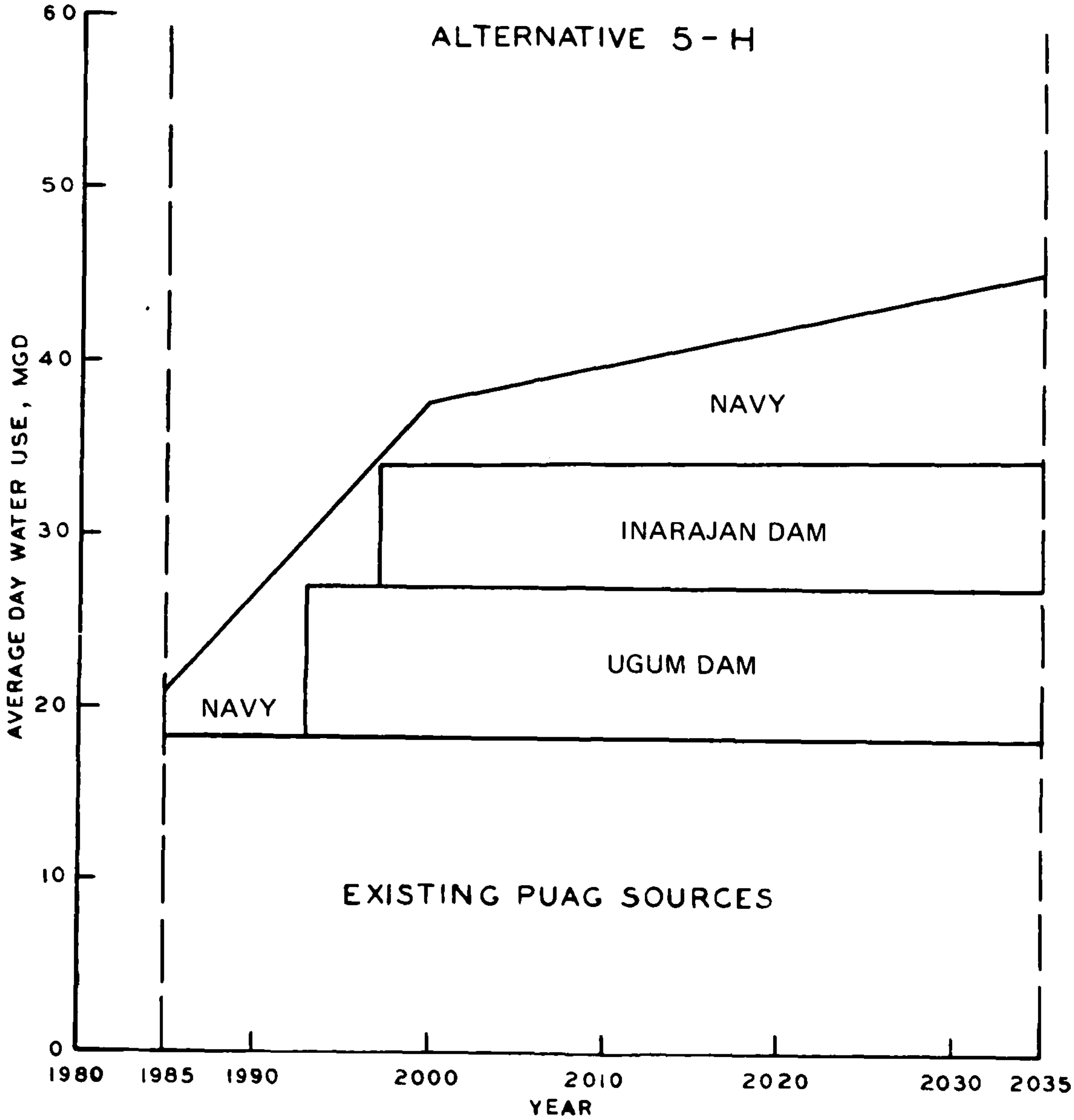

Figure 3-14. Source Staging for Alternative 5-H

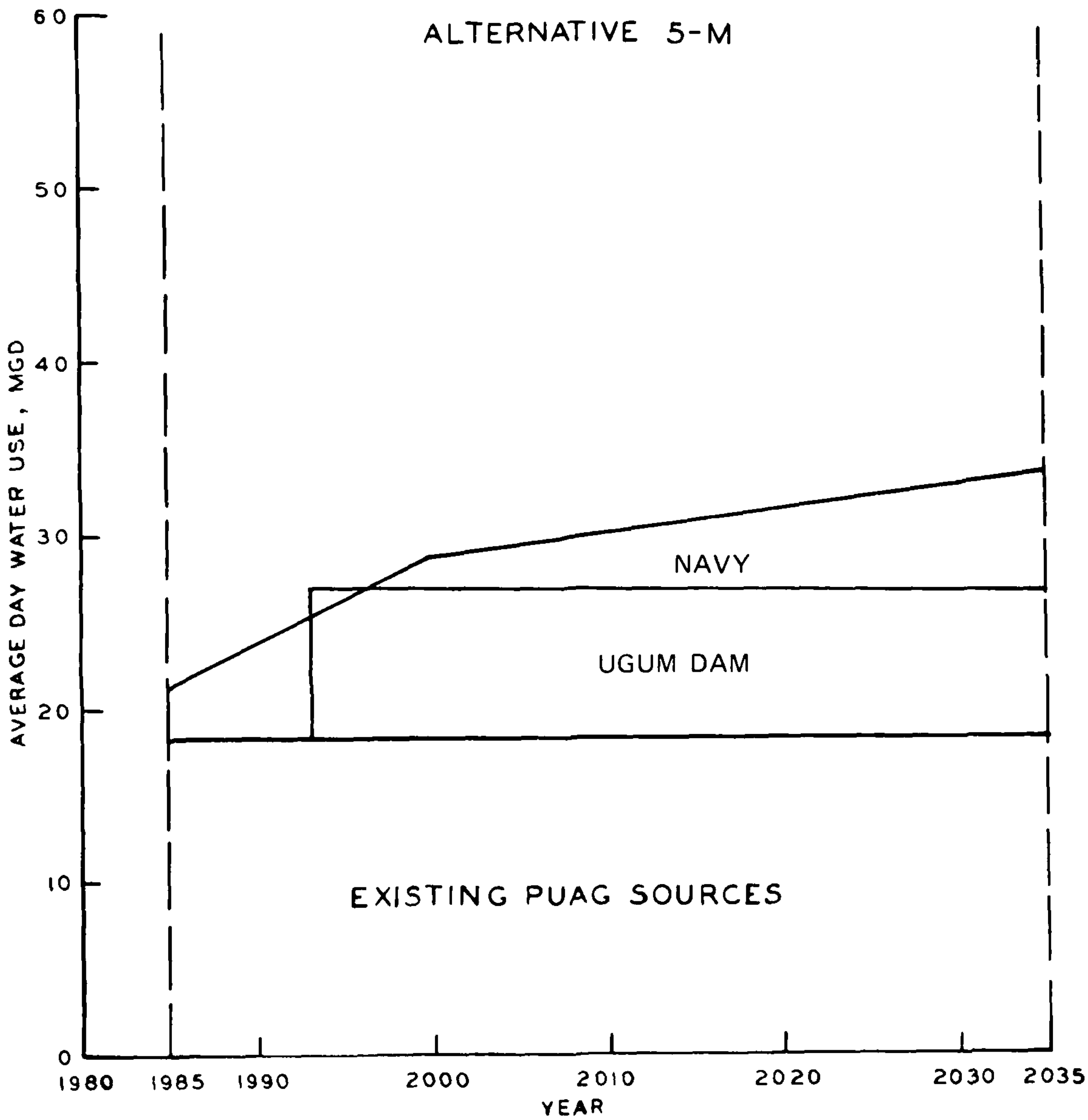

Figure 3-15. Source Staging for Alternative 5-M

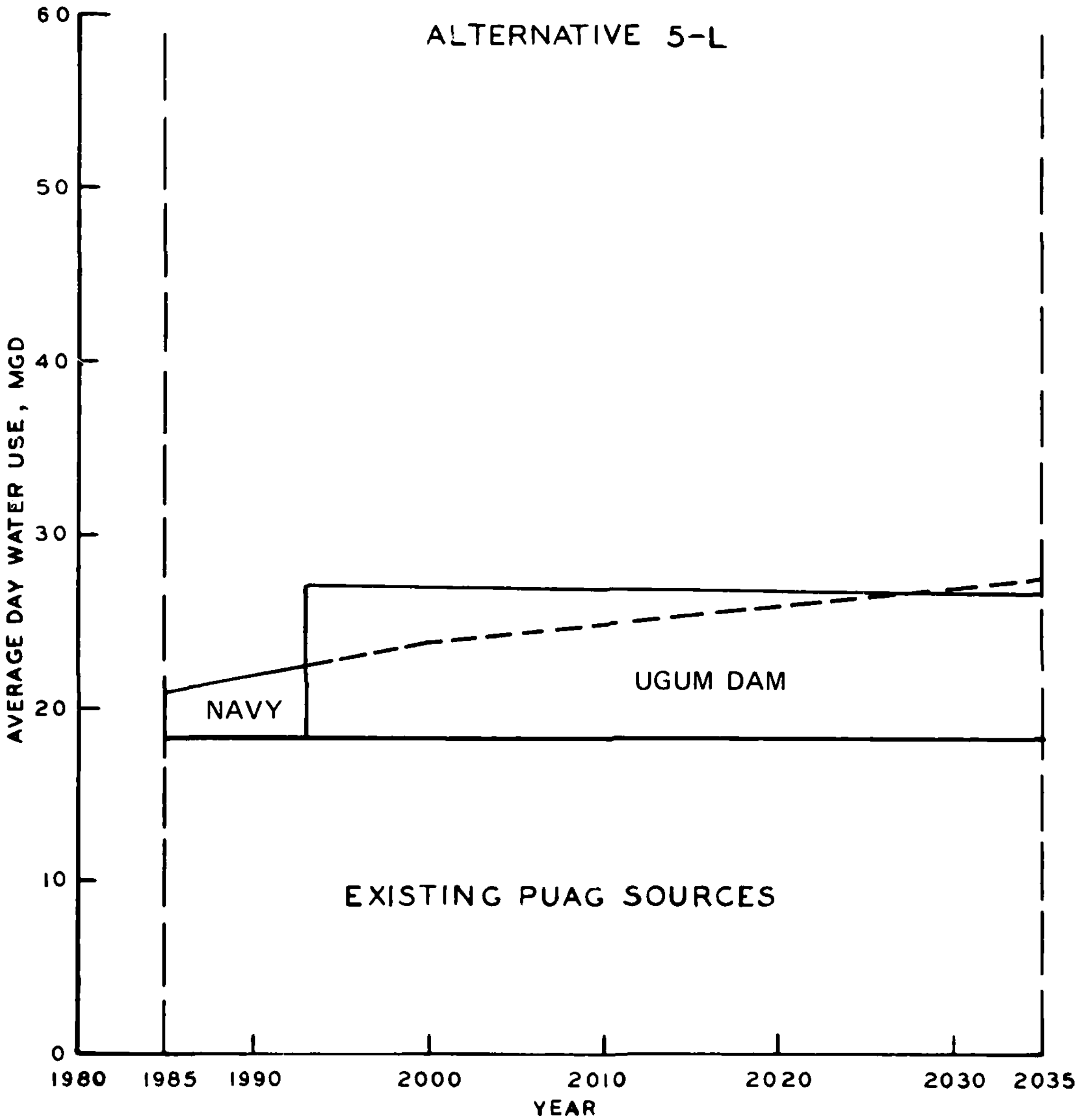

Figure 3-16. Source Staging for Alternative 5-L

accounted for using salvage values. If, for example, a facility costing \$1 million is built in 2000 and has a 50-year life, the present worth of its salvage values in 2035, using linear depreciation, is

$$(\$1,000,000)\ \frac{(2000 + 50) - 2035}{50}\ (1.07625)^{-50} = \$7,600$$

Since most of the facilities are built before 2000, the above calculation shows that salvage value (except for dams) is small enough to be ignored.

In the case of the Ugum and Inarajan Dams, which have an economic life of 100 years, there is a significant amount of useful life after 2035, so the dams will be depreciated linearly and the present worth of their salvage value will be subtracted from the cost. This is roughly equivalent to amortizing the dam over 100 years.

Dams

First costs for both the Ugum River Dam and Inarajan River Dam were taken from Table E-3 and E-5 of the Ugum River Report and are given in Table 3-1. The costs were corrected by subtracting 1.12 (i.e. 4750/4239) times the "Water Treatment Facilities" item, so that the costs would include only the dam and not the treatment plant and raw water pumping facilities. These facilities depend somewhat on the plan and are listed separately.

The annual O&M and replacement costs for the Ugum and Inarajan Dams are given as \$135,000/yr and \$136,000/yr, respectively, on page G-18 of the Ugum River Report. When compared with the cost of operating a complete surface water treatment plant (as shown in Table 2-3), these costs appear low. Therefore, these costs were interpreted to reflect only the costs of operating the dams and not the water treatment facilities.

Since the dams are built over a three-year period, interest during construction of \$4,326,000 and \$5,497,000 is used for the Ugum and Inarajan Dams, respectively. The dams are the only facilities for which interest during construction is calculated because they are the only ones with such long construction times.

Table 3-1

Cost Estimate Summary - Inarajan River

Feature	Cost
Land and Damages	$ 1,439,000
Care and Diversion of Water	636,000
Reservoir	10,529,000
Diversion Channel	162,000
Dam Embankment	16,225,000
Spillway	9,207,000
Outlet Works	4,009,000
Access Road	424,000
Water Treatment Facilities	4,679,000
Construction Facilities	150,000
Subtotal	$47,460,000
Engineering and Design	3,080,000
Supervision and Administration	2,760,000
Total Project First Cost	$53,300,000
Less Water Treatment Facilities	5,240,000
Total Dam First Cost	$48,060,000

Cost Estimate Summary - Ugum River

Feature	Cost
Land and Damages	$ 3,513,000
Care and Diversion of Water	638,000
Reservoir	9,112,000
Diversion Channel	462,000
Dam Embankment	12,115,000
Spillway	1,533,000
Spillway Dikes	1,753,000
Outlet Works	3,847,000
Access Road	653,000
Water Treatment Facilities	8,614,000
Construction Facilities	150,000
Subtotal	$42,390,000
Engineering and Design	2,760,000
Supervision and Administration	2,350,000
Total Project First Cost	$47,500,000
Less Water Treatment Facilities	9,674,000
Total Dam First Cost	$37,826,000

The average annual costs of the dams over the 50-year study period can be calculated as:

$$AAC = crf_n \left[pwf_m \ (CAP + OM/crf_p) - CAP(1 - p/100) \ pwf_n \right]$$

where

AAC = average annual cost over n years, \$/yr

crf_n = capital recovery factor for n years, 1/yr

n = length of amortization period, years

pwf_m = present worth factor for m years

m = year built - base year, years

CAP = capital cost in year m, \$

OM = O&M cost, \$/yr

p = number of years in study period that facility is operating years

For the Ugum Dam ($m = 1993 - 1985$, $n = 50$, $p = 42$, $CAP = 42152$)

$$AAC = 0.0782 \left[0.555 \ (42152 + 135/0.0799) - (42152) \left(1 - \frac{42}{100}\right) 0.0254 \right. = \$1,854,000/\text{year}$$

For the Inarajan Dam ($m = 1997 - 1985$, $n = 50$, $p = 38$, $CAP = 53557$)

$$AAC = 0.0782 \ \ 0.414 \ (53557 + 136/0.0812) - (53557) \left(1 - \frac{38}{100}\right) 0.0254 \Big] = \$1,722,00/\text{year}$$

The above average annual costs could also be generated using the MAPS amortization module described in Chapter 22 of EM 1110-2-502.

Water Treatment

Water Treatment Plant costs based on the capital and O&M costs shown in Table 2-3 are presented below.

	Capacity mgd	Actual Flow mgd	Average Annual Cost $/yr
Plan 3-H, M, L and 5-M, L	9.0	9.0	156,000* 313,000**
Plan 4-H, M, L and 5-H	15.9	9.0† 15.9††	223,000* 445,000**

* Filtration only.
** Conventional treatment.
† For 1993 to 1997.
†† For 1998 to 2035.

The average annual costs differ from those shown in Table 2-3 because they are based on a 9.0-mgd plant built in 1993 and operated from 1994 through 2035 and a 15.9-mgd plant built in 1993, operated at 9.0 mgd from 1994 through 1997 and operated at 15.9 mgd from 1998 through 2035, rather than a plant built during the base year and operated for an amortization life of 25 years.

Transmission Lines

The diameter, length, and capital cost of transmission lines included in the Master Plan are given in Table 3-2. The transmission projects in this study actually consist of several projects from the Master Plan. Most of the smaller distribution lines identified in the Master Plan are not included in Table 3-2 since they are sized for fire flow and their size and staging would be the same for any alternative.

The costs of transmission lines from the southeastern dams are given in Table 3-3. For these pipes, the year of construction depends on the year in which the dam is constructed. The varying water use projections are reflected in changes in pipe sizes (taken from Tables 2-2 and 2-4).

It was felt that lines identified in the Master Plan were adequately sized for the ultimate capacity of the wellfields. Therefore, a reduction in water use would not result in a down sizing of the line, but rather would result in a delay of the construction date. The construction dates are shown in Table 3-4 for each major transmission

Table 3-2

Cost of Transmission Lines from Master Plan

Project	Project in Master Plan	Capital Cost 10^3\$	Diameter in.	Length ft
T-1	A-5	2,232	16	3,600
	A-6	700	12	14,000
	A-9	350	12	7,000
	AB-1	3,007	16	48,500
	AB-2	3,602	24	36,750
	AB-3	589	8	15,500
Total T-1		10,480		
T-2	B-23	558	16	9,000
	B-24	375	12	7,500
	BD-1	527	16	8,500
	D-17	326	16	5,250
	D-19	310	16	5,000
Total T-2		2,096		
T-3	CD-1	620	16	10,000
	D-13	806	16	13,000
	D-16	160	12	3,200
Total T-2		1,586		
T-4	D-9	176	6	5,500
	D-10	527	16	8,500
	D-11	170	12	3,400
Total T-4		873		
T-11	C-4	209	8	5,500
	C-5	613	12	17,250
Total T-11		822		

Table 3-3

Cost of Transmission Lines for Southeastern Dams

Project	Length ft	Plan	Diameter in.	Capital Cost (10^3\$)
T-5	5,000	3-H, M, L; 5-M, L	20	410
		5-H; 4-H, M, L	30	625
T-6	54,300	3-H	14	3040
		3-M	16	3367
		3-L; 5-M, L	18	3801
		4-H	20	4453
		4-M, L; 5-H	24	5321
T-7	12,850	3-L; 4-L	12	642
		3-H, M; 4-H, M	14	720
T-8	11,000	3-M, L; 4-M, L	12	550
		3-H; 4-H	14	616
T-9	6,700	4-H, M, L; 5-H	24	656
T-10	12,000	All 3, 4, 5	24	1176

Table 3-4

Year Built for Transmission Projects

	T-1	T-2,4	T-3,11
1-H	1987	1997	--
1-M	1992	1998	--
1-L	1995	2000	--
2-H	1985	1992	1992
2-M	1990	1995	1995
2-L	1993	1996	1996
3-H	1989	--	1993
3-M	1996	--	1993
3-L	2000	--	1993

T-5, 6, 7, 8, 10 built in 1993

T-9 built in 1997

project. The dates assigned are based on the construction period given in the Master Plan, corrected to account for high or low use rate.

The O&M costs for transmission lines are generally on the order of 0.2 percent of construction cost per year. Since these costs are so small, they are omitted in this analysis.

The average annual cost for each transmission line is shown in Table 3-5. The total average annual cost for transmission lines for each plan is presented in the final column.

Pumping Stations

The cost of pumping stations is a function of capacity, head, and type of structure. The capacity and head at pumping stations associated with the southeastern river dams are taken from Tables 2-2 and 2-4. The capacity of the other pumping stations are taken from the Master Plan. The head to be provided by the pumps is not given in the Master Plan, so head requirements were estimated based on the elevation of the pumping station and expected head losses in the pipes.

The costs of the pumping stations required by each plan are given in Table 3-6. The costs were generated using the MAPS computer program and are based on improved structures at Ugum Dam (P-5), Inarajan Dam (P-4), Talofofo Bay (P-2), and simple structures at Brigade (P-1a), Cross Island Road (P-1b), and Windward Hills (P-3).

Many of the pumping stations described in the Master Plan are not included in Table 3-6 (e.g., BPS-1-Latte Heights) since these stations are primarily for local distribution and would be essentially the same for all plans.

For a given facility, the capital costs given in Table 3-6 are somewhat higher than those in the Master Plan. It is believed the costs reported in the Master Plan are generally too low. Capital costs were actually shown to be a minor component of the average annual costs for the pumping stations. This resulted directly from the fact that energy costs accounted for approximately 80 percent of the total costs.

Wells

The Master Plan gives the capital cost of a well as $200,000. This number is reasonable and is used in the following estimates. It

Table 3-5

Average Annual Cost ($/yr @ 7-5/8% over 1985-2035) of Transmission Projects

Plan	T-1	T-2	T-3	T-4	T-5	T-6	T-7	T-8	T-9	T-10	T-11	Total
1-H	708	68	-	28	-	-	-	-	-	-	-	804
1-M	490	63	-	26	-	-	-	-	-	-	-	579
1-L	393	54	-	23	-	-	-	-	-	-	-	470
2-H	820	98	74	41	-	-	-	-	-	-	38	1071
2-M	568	79	60	33	-	-	-	-	-	-	31	771
2-L	455	73	55	30	-	-	-	-	-	-	39	652
3-H	611	-	69	-	18	132	31	27	-	51	36	975
3-M	365	-	69	-	18	146	31	24	-	51	36	740
3-L	272	-	69	-	18	165	28	24	-	51	36	663
4-H	-	-	-	-	27	194	31	27	21	51	36	387
4-M	-	-	-	-	27	231	31	24	21	51	36	421
4-L	-	-	-	-	27	231	28	24	21	51	36	418
5-H	-	-	-	-	27	231	-	-	21	51	-	330
5-M	-	-	-	-	18	165	-	-	-	51	-	234
5-L	-	-	-	-	18	165	-	-	-	51	-	234

Table 3-6

Cost of Pumping Stations (Energy = 11.9¢/kwhr)

Name	Plan	Capacity mgd	Head ft	Capital Cost ($10^3$$)	O&M Cost (10^3/yr) 6¢/kwhr	O&M Cost (10^3/yr) 11.9¢/kwhr	Year Built	Average Annual Cost ($10^3$$/yr) 6¢/kwhr	Average Annual Cost ($10^3$$/yr) 11.9¢/kwhr
P-1a Brigade (DPS-1)	1-H	4.3	400	486	213	389	1997	101	171
	1-M	4.3	400	486	213	389	1998	93	158
	1-L	4.3	400	486	213	389	2000	91	135
	2-H	4.3	400	486	213	389	1992	148	251
	2-M	4.3	400	486	213	389	1995	118	200
	2-L	4.3	400	485	213	389	1996	109	185
P-1b Cross Island (DPS-2)	2-H	2.5	200	296	68	113	1992	54	80
	2-M	2.5	200	296	68	113	1995	43	64
	2-L	2.5	200	296	68	113	1996	40	59
P-2 Talofofo Bay	3-H	7.6	141	510	146	242	1993	102	154
	3-M	8.0	150	560	164	271	1993	114	172
	3-L	8.2	129	540	144	239	1993	102	153
	4-H	14.5	236	996	439	774	1993	282	464
	4-M	14.9	159	920	313	536	1993	210	331
	4-L	15.1	152	880	303	519	1993	203	321
	5-H	14.5	194	900	361	636	1993	235	385
	5-M	8.0	198	630	207	358	1993	140	222
	5-L	8.2	210	680	225	389	1993	152	251

(Continued)

Table 3-6 (Concluded)

Name	Plan	Capacity mgd	Head ft	Capital Cost (10^3\$)	O&M Cost (10^3/yr) 6¢/kwhr	O&M Cost (10^3/yr) 11.9¢/kwhr	Year Built	Average Annual Cost (10^3\$/yr) 6¢/kwhr	Average Annual Cost (10^3\$/yr) 11.9¢/kwhr
P-3 Windward Hills	3-H	3.3	314	380	135	234	1993	90	144
	3-M	2.5	432	380	140	244	1993	93	149
	3-L	2.0	448	350	116	203	1993	78	126
	4-H	3.3	358	400	154	267	1993	101	163
	4-M	2.5	416	370	135	235	1993	90	144
	4-L	2.0	416	330	104	188	1993	71	117
P-4 Inarajan Dam	4-H M,L; 5-H	6.9	254	603	224	397	1997	109	178
P-5 Ugum Dam	all 3,4,5	9.0	100	538	132	203	1997	95	138

appears to include chlorination equipment, but no standby power.

The O&M costs for wells include labor, power, chlorine, and other chemical costs, and are generally significant, but are not covered in the Master Plan. Labor should cost $4000/yr/well and chlorine $3000/yr/well, based on standard dosages and 1 man-hour/day/well. The pumping energy for a well providing 200 gpm (0.29 mgd) can be given by:

$$C = 11.41\ QHP/e$$

where

C = energy cost, $/yr

Q = flow, mgd

H = head, ft

P = price of power, ¢/kwhr

e = efficiency

The head at the well is generally 100 psi and the depth to groundwater averages 170 ft, so the head required, H , is 170 + 2.31 (100) or 401 ft. The price of energy is taken as 6 and 11.9¢/kwhr, and well pumps can be assumed to have a wire-to-water efficiency of 0.50. This gives energy cost as:

$$\begin{aligned} C &= 11.41(0.29)(401)(6)/0.5 \\ &= 15{,}922 \text{ say } \$16{,}000\text{/yr/well for } 6¢\text{/kwhr} \\ &= \$31{,}600\text{/yr/well for } 11.9¢\text{/kwhr} \end{aligned}$$

The total O&M cost is, therefore, approximately $23,000/yr/well at 6¢/kwhr, or $38,600/yr/well at 11.9¢/kwhr.

The flow from "new wells" (i.e., built after 1985) for each plan is given in Figure 3-17. These data were taken from Figures 3-2 through 3-16. The flow from new wells (Q) at any time (t) can be represented by a set of straight lines. For example, for plan 3-M there is a period of construction, followed by a 20-year period of no construction immediately after Ugum Dam is completed, followed by a period of new construction once demand exceeds the capacity of the dam. This can be represented by

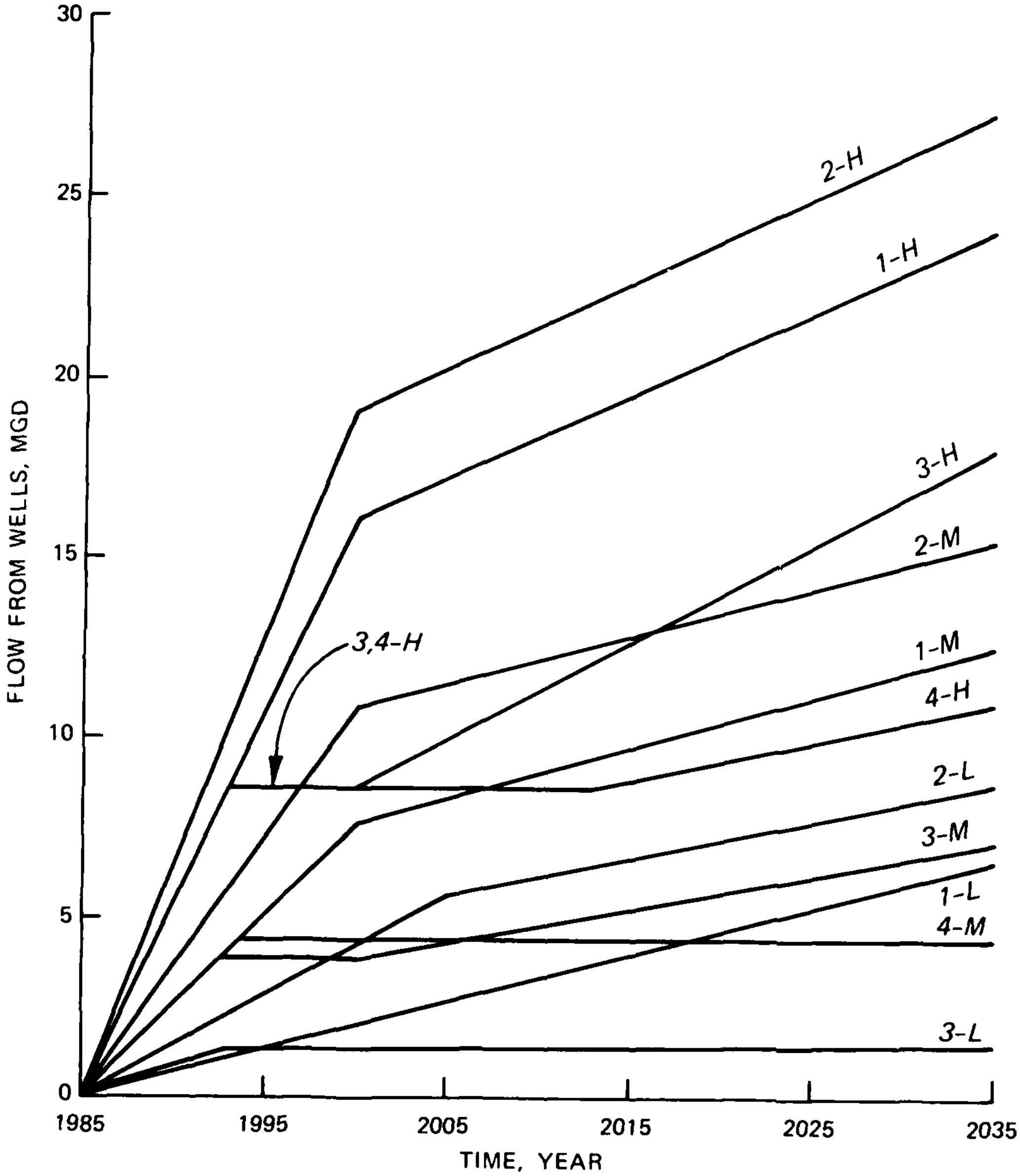

Figure 3-17. Additional Well Capacity Required
(No New Wells for 4-1 and All of 5)

$$Q(t) = \begin{cases} 0 + 0.475t & , \; 0 < t \leq 8 \\ 3.5 & , \; 8 < t \leq 28 \\ -0.8 + 0.159t & , \; 28 < t \leq 50 \end{cases}$$

Note that each piece of the function is represented by a line segment of the form

$$Q(t) = a + bt \quad , \; t_k < t \leq t_{k+1}$$

The values of a , b , t_k , t_{k+1} are given for each line segment in Table 3-7. The rate at which wells are constructed is represented by the b coefficient since it corresponds to:

$$\frac{dQ}{dt} = b, \text{ mgd/yr}$$

Since b is new well yield in million gallons per day per year, and each well yields 0.29 mgd (200 gpm), b/0.29 is the number of wells built per year (or 106/b is the average number of days between successive wells being brought on line).

The procedure for calculating the average annual cost of wells, given the function $Q(t)$ and the capital and O&M costs for wells, is described in Appendix C. The average annual cost for the wells required by each plan is given in Table 3-8. Note that O&M costs are consistently higher than capital costs.

<u>Purchase</u>

Some water must be purchased from the military for each alternative. In plan type 1, water will be purchased at roughly the same rate as at present. In plan types 2, 3, and 4, military sources will be used until a dam or sufficient wells can be constructed to make the PUAG capable of meeting all of its own needs. In plan type 5, military sources will be used to supplement the dams.

The quantity of military water required as a function of time is shown in Figure 3-18 for plan types 1 through 4 and Figure 3-19 for plan type 5. The coefficients of the line segments are shown in Table 3-9.

AD-A122 614 WATER SUPPLY ANALYSIS FOR THE GUAM COMPREHENSIVE STUDY (U) ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG MS ENVIRONMENTAL LAB T M WALSKI OCT 82

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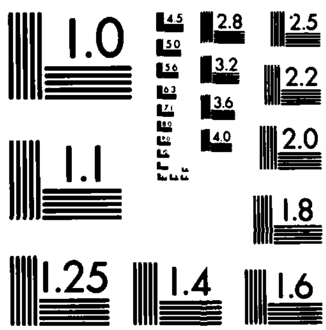


MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

Table 3-7

Coefficients for Well Equations

Plan	a	b	t_{k-1} < t <	t_k
1-H	0	1.07	0	15
	12.2	0.23	15	50
1-M	0	0.507	0	15
	5.7	0.137	15	50
1-L	0	0.13	0	50
2-H	0	1.27	0	15
	15.8	0.237	15	50
2-M	0	0.72	0	15
	8.5	0.137	15	50
2-L	0	0.373	0	15
	3.8	0.0971	15	50
3-H	0	1.06	0	8
	8.4	0	8	15
	6.8	0.22	15	50
3-M	0	0.475	0	8
	3.5	0	8	28
	-0.8	0.159	28	50
3-L	0	0.175	0	8
	1.5	0	28	50
4-H	0	1.1	0	8
	8.8	0	8	27
	0	0.217	27	50
4-M	0	0.575	0	8
	4.6	0	8	50

Table 3-8

Average Annual Cost for New Wells

Plan	Amortized Construction Cost (10^3\$/yr)	Amortized O&M Cost (10^3\$/yr)	Average Annual Cost (10^3\$/yr)
1-H	597	1200	1797
1-M	289	592	881
1-L	96	189	285
2-H	700	1440	2140
2-M	398	800	1198
2-L	212	416	628
3-H	410	893	1302
3-M	173	330	503
3-L	59	129	188
4-H	390	752	1142
4-M	194	403	597
4-L	-	-	-
5-H	-	-	-
5-M	-	-	-
5-L	-	-	-

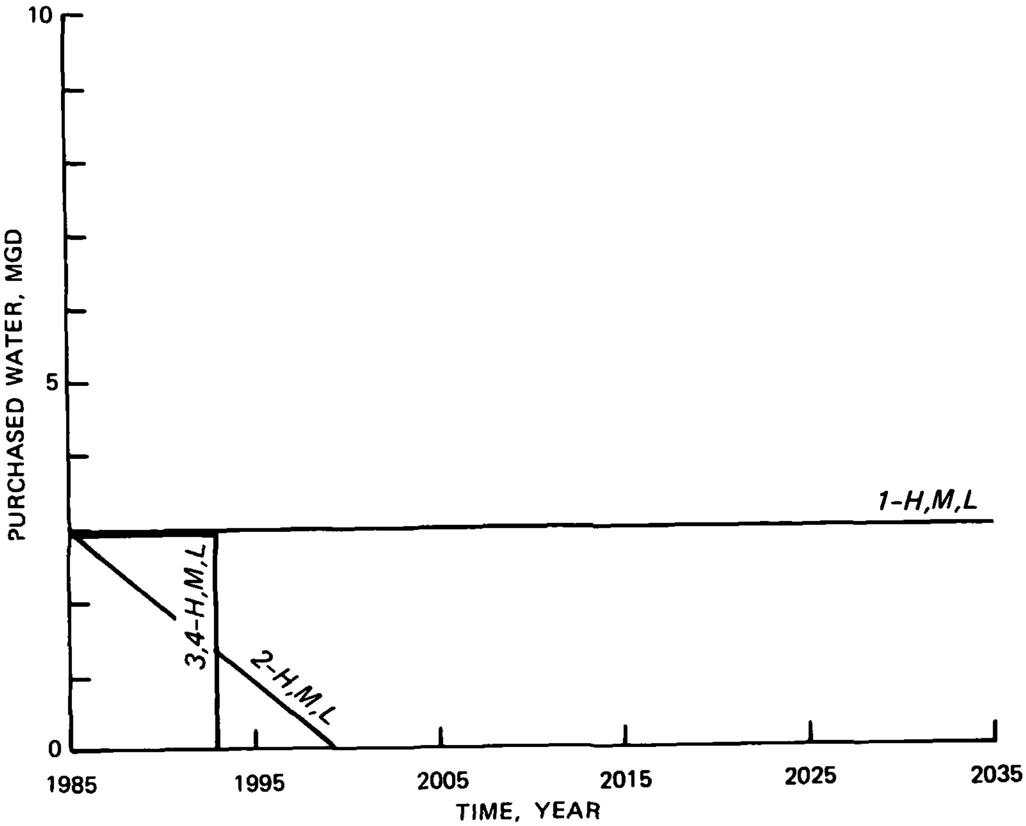


Figure 3-18. Purchase Requirements for Plans 1, 2, 3, and 4

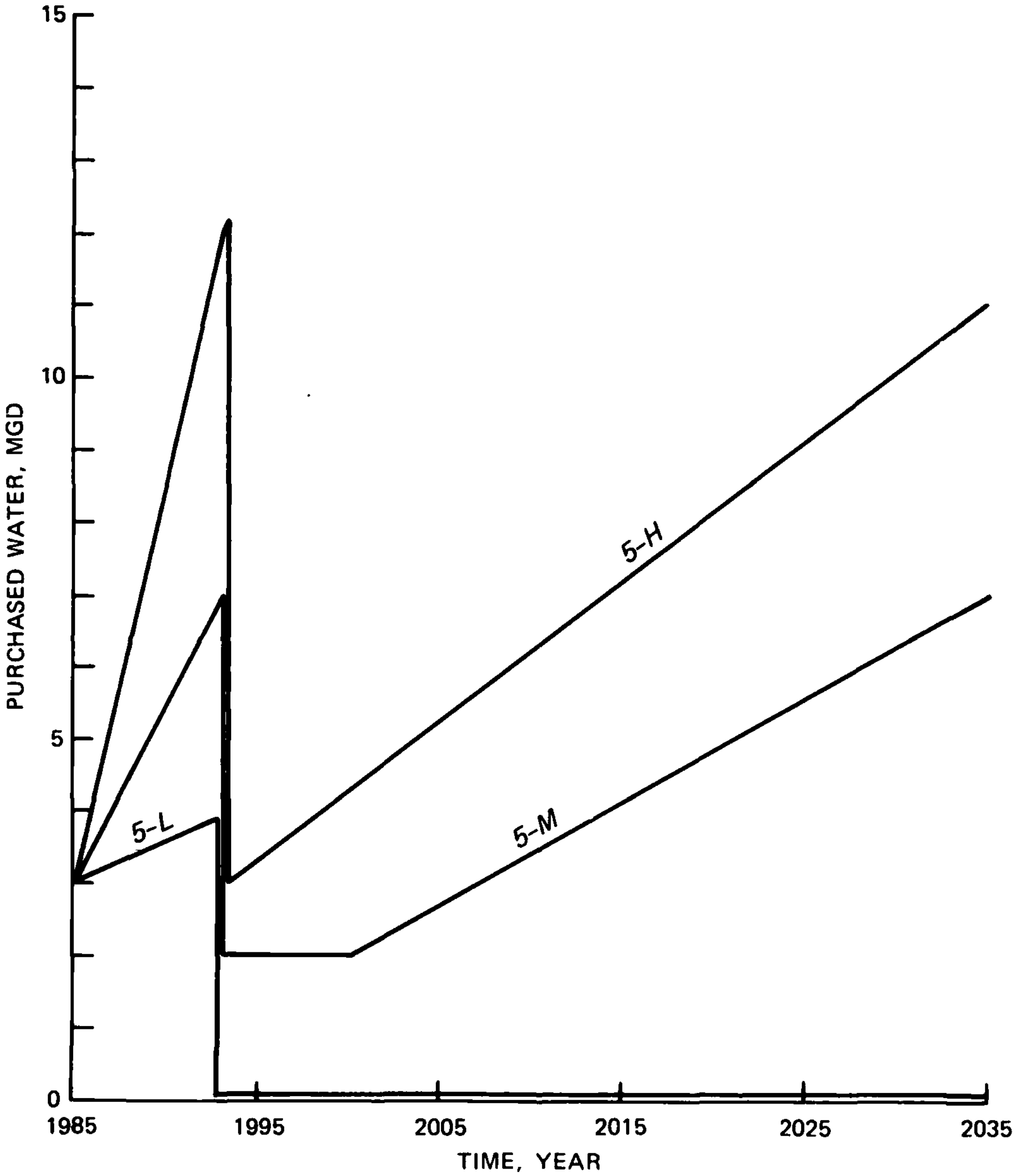


Figure 3-19. Purchase Requirements for Plan 5

Table 3-9

Coefficients for Purchase Equations, $Q = a + bt$

Plan	a	b	k-1	t_k
1-H, M, L	3.0	0	0	50
2-H, M, L	3.0	-0.20	0	15
3, 4-H, M, L	3.0	0	0	8
	0	0	8	50
5-H	3.0	1.125	0	8
	2.2	0.176	8	50
5-M	3.0	0.50	0	8
	2.0	0	8	15
	-0.2	0.136	15	50
5-L	3.0	0.125	0	8
	0	0	8	50

The average annual cost of water purchases for each alternative is presented in Table 3-10. They were calculated using the same formulas as the average annual cost of well O&M derived in Appendix C.

Table 3-10

Average Annual Cost to Purchase Water

Plan	Average Annual Cost (10^3\$/yr)
1-H, M, L	1220
2-H, M, L	510
3, 4-H, M, L	823
5-H	2445
5-M	1445
5-L	615

Miscellaneous

Numerous miscellaneous capital improvements were identified in the Master Plan. Most of these are required regardless of which plan is selected (e.g., security fencing at storage tanks). The only improvements that are significantly affected by the type of plan are the construction of typhoon-proof well housings (ABM-3) and the purchase of standby generators (ABM-2). Most of these will probably not be required in Plans 3, 4, and 5 since the dam source and pumping stations will have this type of protection and will be able to meet most of the island's needs during an emergency.

Miscellaneous improvements are estimated to cost $1,680,000 (ABM-3) and $705,000 (ABM-2). For plans 3, 4, and 5, the cost will be about $200,000; therefore, the additional cost to provide protection and backup power to wells, instead of a single surface water source, is $2,185,000. This construction project is to take place in, or about, 1988; therefore, the present worth may be estimated to be $1,752,000 and the average annual cost is $137,000.

4. Comparison of Alternative Plans

Introduction

Costs for the individual facilities developed in the previous section are combined in this section to determine the total average annual cost for each alternative. This is followed by a discussion of some other considerations not accounted for in the cost estimates. Procedures for calculating the foregone cost of conservation are then presented.

Cost Summary

Using descriptions of the facilities, which make up each alternative as given in Section 1, and cost estimates from Section 3, the average annual cost of each alternative was determined. This information is presented in Table 4-1 for an energy cost of 6¢/kwhr and a filtration water treatment plant at the southeastern river dams. Table 4-2 is for an energy cost of 11.9¢/kwhr while Table 4-3 is for conventional treatment. Costs are shown as a function of average day water use in the year 2035 in Figure 4-1. Bar charts are presented in Figures 4-2 through 4-4 for the high, medium, and low use projections, respectively, to indicate the relative importance of well, dam, transmission, and purchase costs.

These figures and tables show that, for all use projections, plan type 2 is the least costly with plan type 1 slightly more expensive. This indicates that wells are the least costly supplies and that supplementing wells with purchased water is slightly more expensive than building more wells.

The bar charts indicate that it is the very large first cost of the dams that makes plans requiring them relatively unattractive from an economic viewpoint. The plans using both the Inarajan and Ugum Dams (i.e. 4 and 5 high) are the most costly.

The wells are very attractive economically because their construction can be delayed until they are needed and they can be added in small increments. For example, plan 2-H requires 93 wells to be built. Suppose these wells were all built in 1985 and operated continuously for 50 years. In that case, the amortized capital cost would be

Table 4-1

Summary of Average Annual Cost (10^3\$/yr) of Alternatives

Energy = 6¢/kwhr; Direct Filtration

	Well and Miscellaneous	Dam and Treatment	Transmission and Pump	Purchase	Total
1-H	1449	-	905	1220	3574
1-M	779	-	672	1220	2671
1-L	346	-	561	1220	2127
2-H	1695	-	1226	510	3431
2-M	1012	-	932	510	2454
2-L	597	-	801	510	1908
3-H	658	2193	1262	544	4657
3-M	370	2193	1042	544	4149
3-L	136	2193	938	544	3811
4-H	838	4085	974	544	6441
4-M	434	4085	925	544	5988
4-L	-	4085	896	544	5525
5-H	-	4085	796	2445	7326
5-M	-	2193	469	1445	4107
5-L	-	2193	481	615	3289

Table 4-2

Summary of Average Annual Cost (10^3\$/yr) of Alternatives

Energy = 11.9¢/kwhr; Direct Filtration

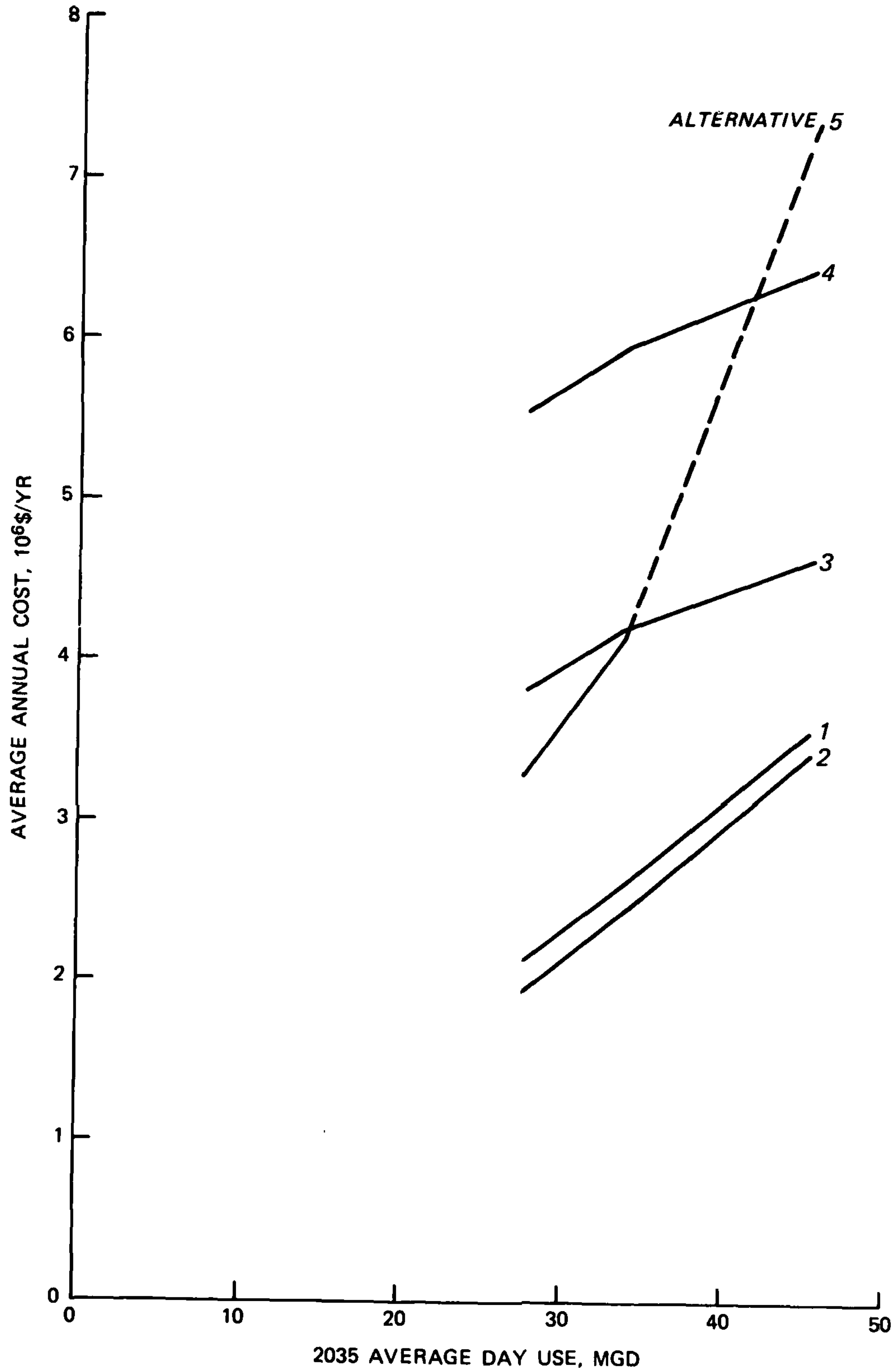
	Well and Miscellaneous	Dam and Treatment	Transmission and Pump	Purchase	Total
1-H	1934	-	975	1220	4129
1-M	1018	-	737	1220	2975
1-L	422	-	605	1220	2247
2-H	2277	-	1402	510	4189
2-M	1335	-	1035	510	2880
2-L	765	-	896	510	2171
3-H	1302	2193	1411	544	5450
3-M	508	2193	1199	544	4444
3-L	188	2193	1080	544	4005
4-H	1142	4085	1330	544	7101
4-M	597	4085	1212	544	6438
4-L	-	4085	1172	544	5801
5-H	-	4085	1031	2445	7561
5-M	-	2193	594	1445	4232
5-L	-	2193	623	615	3431

Table 4-3

Summary of Average Annual Cost (10^3\$/yr) of Alternatives

Energy = 11.9¢/kwhr; Conventional Treatment

	Well and Miscellaneous	Dam and Treatment	Transmission and Pump	Purchase	Total
1-H	1934	-	975	1220	4129
1-M	1018	-	737	1220	2975
1-L	422	-	605	1220	2247
2-H	2277	-	1402	510	4189
2-M	1335	-	1035	510	2880
2-L	765	-	896	510	2171
3-H	1302	2350	1411	544	5607
3-M	508	2350	1199	544	4601
3-L	188	2350	1080	544	4162
4-H	1142	4307	1330	544	7323
4-M	597	4307	1212	544	6660
4-L	-	4307	1172	544	6023
5-H	-	4307	1031	2445	7783
5-M	-	2350	594	1445	4389
5-L	-	2350	623	615	3588



igure 4-1. Average Annual Cost as Function of 2035 Use

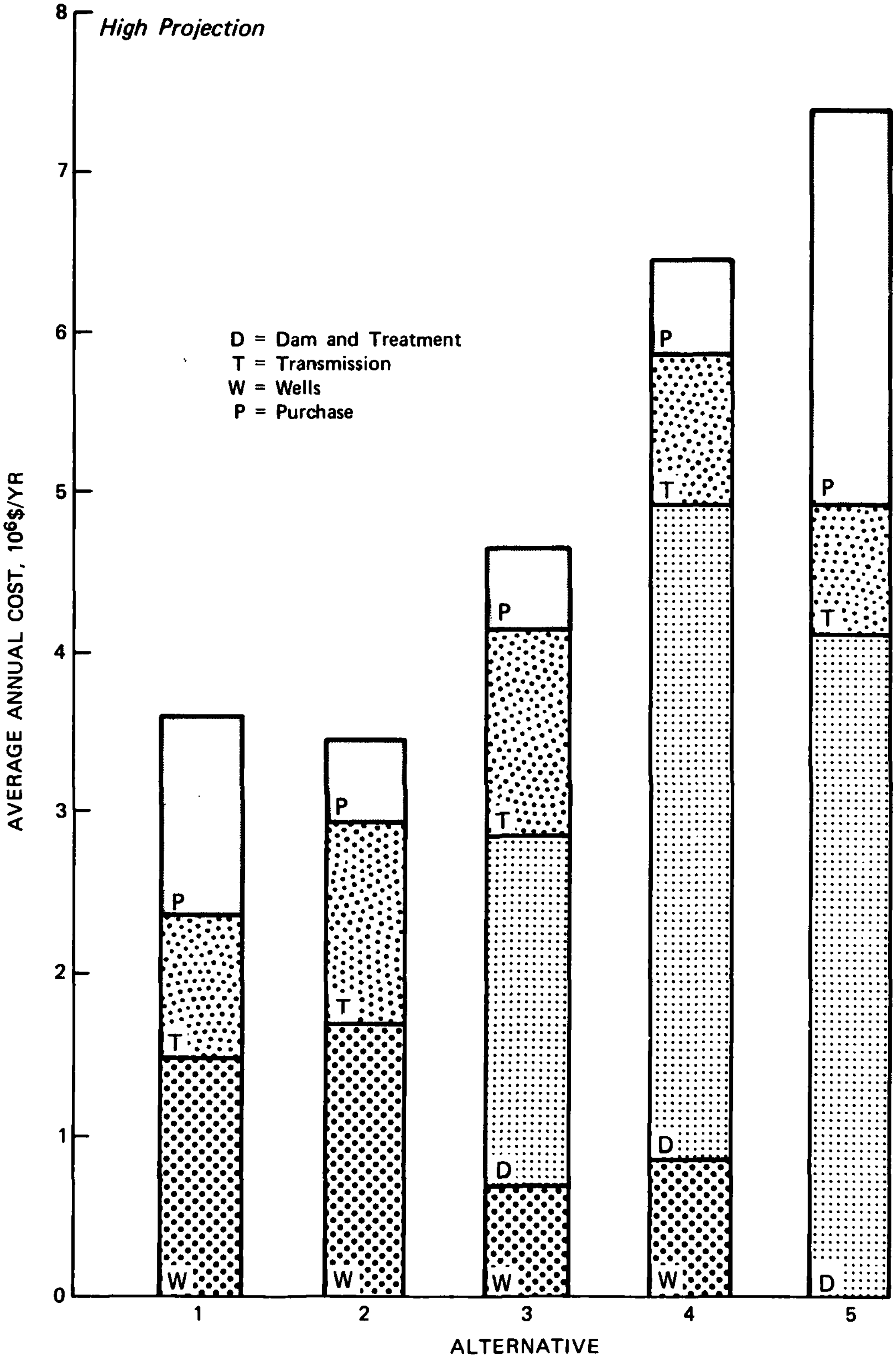


Figure 4-2. Cost Comparison for High Projection (For Energy = 6¢/kwhr and Filtration at Dams)

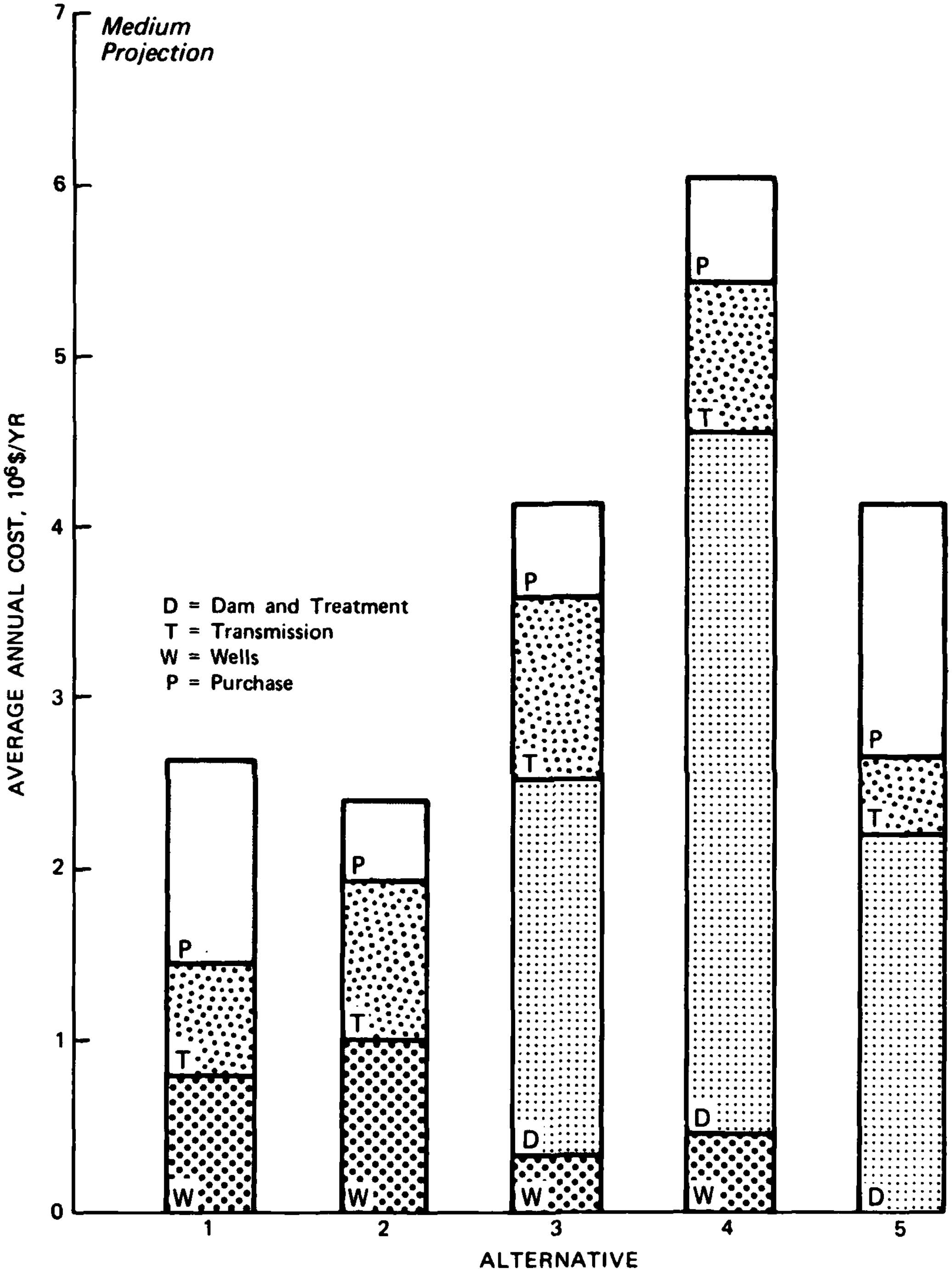


Figure 4-3. Cost Comparison for Medium Projection (For Energy = 6¢/kwhr and Filtration at Dams)

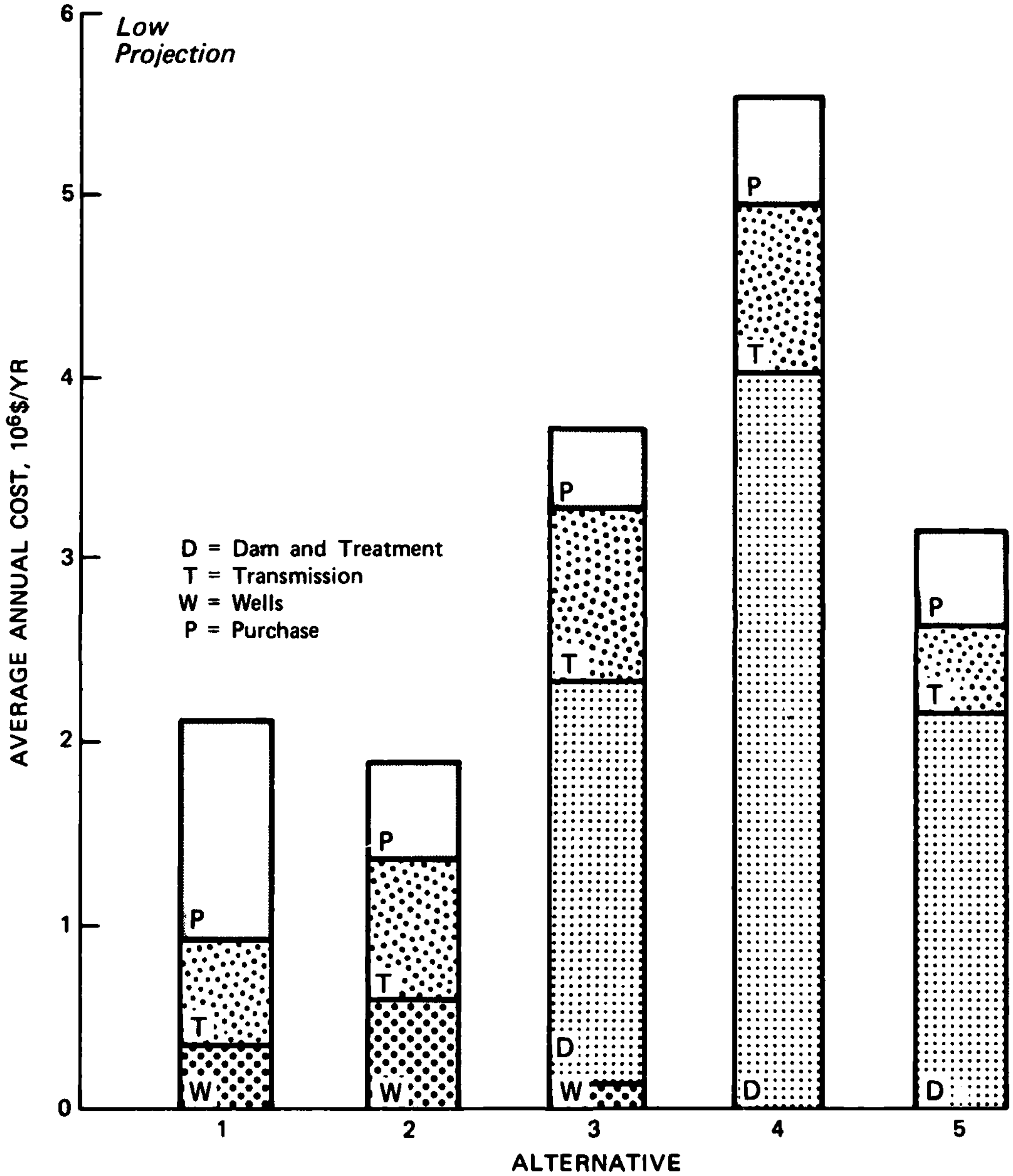


Figure 4-4. Cost Comparison for Low Projection (For Energy = 6¢/kwhr and Filtration at Dams)

$1,456,000/yr instead of $700,000/yr and O&M costs would be $2,139,000/yr instead of $858,000/yr. This means that alternative 2-H would be comparable in cost to alternative 3-H which uses a dam.

Similarly the dams become more attractive if their construction is delayed. For example, if the Ugum Dam construction is delayed by 10 years to 2003, the amortized capital cost is reduced by a factor of two. Of course, there would be a need for additional water in the intervening years, but, in general, the costs would be reduced by delaying dam construction.

Sensitivity to Energy
Cost and Level of Treatment

Table 4-1 and Figures 4-1 through 4-3 are based on energy costs of 6¢/kwhr. The cost to produce energy is actually 11.9¢/kwhr. If this price is used, the more energy intensive alternatives become less attractive. The cost of each alternative for an energy cost of 11.9¢/kwhr is shown in Table 4-2. The ranking of the alternatives does not change much between alternatives in Tables 4-1 and 4-2 but there is some relative change. For example, at 6¢/kwhr, alternative 5-M was 67 percent more expensive than 2-M. At 11.9¢/kwhr, it is 47 percent more expensive.

Another decision which can affect cost is the level of treatment provided at the dams. The Ugum River Report recommended conventional treatment (coagulation, flocculation, sedimentation, and filtration). The estimates given in Tables 4-1 and 4-2 are based on filtration only. Table 4-3 shows the costs for the case in which conventional treatment is used. The relative ranking of the alternatives remains the same, but the dams on southeastern rivers become slightly less attractive.

Water Quality

Water taken from the southeastern surface sources must be subjected to considerable treatment prior to use while groundwater taken from the northern lens can be disinfected and used directly (i.e. no treatment except chlorination). As a result, finished waters from the two sources may be quite different with respect to quality.

The treated surface water should be of generally superior quality, especially with respect to mineral content, hardness, and corrosivity (the water can be stabilized during the treatment process). Therefore, the higher quality surface water will require less additional treatment prior to special uses applications (e.g. boiler feed water, specialized cleaning operations, etc.). This will result in cost savings to consumers. An additional factor is that customer-owned appliances should be less subject to water quality related failures if the surface water is used.

Prevention of waterborne disease is always a primary concern in public water supply. In this regard, dependence on disinfection at individual well sites is questionable. Clearly, controlled disinfection at a centralized water treatment plant is more dependable and reliable than automated disinfection at a host of individual well sites.

The northern lens aquifer underlies a large developed area while the Ugum and Inarajan Dam drainage areas are relatively undeveloped. The aquifer is highly susceptible to contamination from chemical spills or illegal wastes discharge. Having a diversity of sources would enable the PUAG to shut down contaminated wells and use surface water if there were a problem with well contamination.

It is difficult to determine from the Master Plan whether water from the northern lens aquifer is scale forming or corrosive. A determination should be made of the stability of the water. If it is not stable, it will result in a low carrying capacity of water mains. The stability is easy to control at a single source, but is difficult to control with widely scattered well sources.

From the above discussion, it is clear that water from the southeastern dams would be of better quality than from the northern lens aquifer. Unfortunately, there is no way to assign a dollar value to these benefits, except for perhaps the extra cost to treat boiler feedwater. Nevertheless, improved drinking water quality should be listed as a benefit of the surface water sources. Providing treatment at each individual well comparable to that achieved at surface water treatment plants would be extremely expensive since economies of scale could not be realized at each well.

Well Capacity

Wells in this study were assumed to yield 200 gpm (0.29 mgd). However, with time, wells tend to lose capacity due to fouling or clogging of screens. Most of the existing wells on Guam are currently producing less than 200 gpm (Appendix D of the Master Plan).

Since the average annual cost of wells varies inversely with yield (Appendix C), costs can be adjusted to account for the lower yield by multiplying the cost in Table 3-8 by the inverse ratio of the yields. For example, if a yield of 160 gpm was used for alternative 2-M, the cost (in 10^3\$) would be

$$(\$875) \times \frac{200}{160} = \$1,094$$

In seismically active areas such as Guam, wells occasionally need to be abandoned because ground motion causes them to become inoperable. This could become a problem on Guam and might result in substantial well replacement costs. If an estimate can be made of the rate at which wells must be replaced, then these costs (if significant) should be added to the cost of well alternatives.

Aquifer Yield

At present, there remains some question as to (safe) groundwater yield. The Ugum River Report used 40 mgd as safe yield for public water supply. The Master Plan (pg 8-5) states that usable yield is likely to be in the range of 30 to 60 mgd.

The answer to the question of safe yield should be provided when the "Northern Guam Lens Study" is published. This study report will include the results of a major groundwater modeling study.

If the study indicates that a safe yield of 45 mgd (corresponding to the high use projection) for public water supply cannot be provided, then some adjustment must be made to the results of this report as plan 2-H and possibly 1-H may be infeasible. There are several alternatives.

The first alternative is to reduce water loss. At present, unaccounted for water is on the order of 30 percent of production (approx 5 mgd). This can be cut in half with a thorough water inventory and leak detection survey and control program.

If the shortfall is small, some minor sources, such as Agana Springs, can be developed to relieve the stress on the aquifer. Small surface water intakes on the Pago, Talofofo, and Inarajan Rivers may also be possible. Limited amounts of additional water may also be purchased from the Navy.

If the shortfall is large and conservation by reduction of unaccounted for water or demand management is not adequate, development of the southeastern rivers becomes a necessity. In that case, plan 3 is the most attractive alternative from an economic as well as a water quality standpoint. In such a case, it is economically desirable to delay construction of the dam as long as possible.

Energy Cost

Energy prices of 6 and 11.0¢/kwhr are used in this report. Unlike capital costs, which occur near the beginning of the study period, energy costs increase throughout the study period as flow increases. If the unit price of energy increases disproportionately with other prices (i.e. the opportunity price of energy is greater than 11.9¢/kwhr), then the cost of energy for each of the alternatives should increase. In order to calculate the cost of energy correctly, it is necessary to project the opportunity price of energy throughout the study period. This, of course, cannot be done with any great confidence. The evaluation section of POD projects that the price of fuel on Guam will increase by a factor of 2.15 in the years from 1982 to 2000.

Plans relying primarily on wells use considerably more energy than those without wells. Thus, in the face of rising energy costs, these plans become less attractive than more capital-intensive projects (i.e. dams).

Conservation Foregone Costs

An important measure of the benefits of water conservation is the foregone water supply cost (i.e. costs not incurred as a direct consequence of conservation). These can be further divided into short run (i.e. existing facilities not used) and long run (i.e. new facilities not built nor operated).

Using 45 mgd as the unrestricted water use in 2035, it is possible to use the data from Table 4-1 to determine a foregone cost function for each type of plan (the method used is described ETL 1110-2-259). These functions are shown in Figure 4-5. Care must be exercised in using these functions for plans involving dams (e.g. plan 5) because the points are connected by a straight line when actually they might

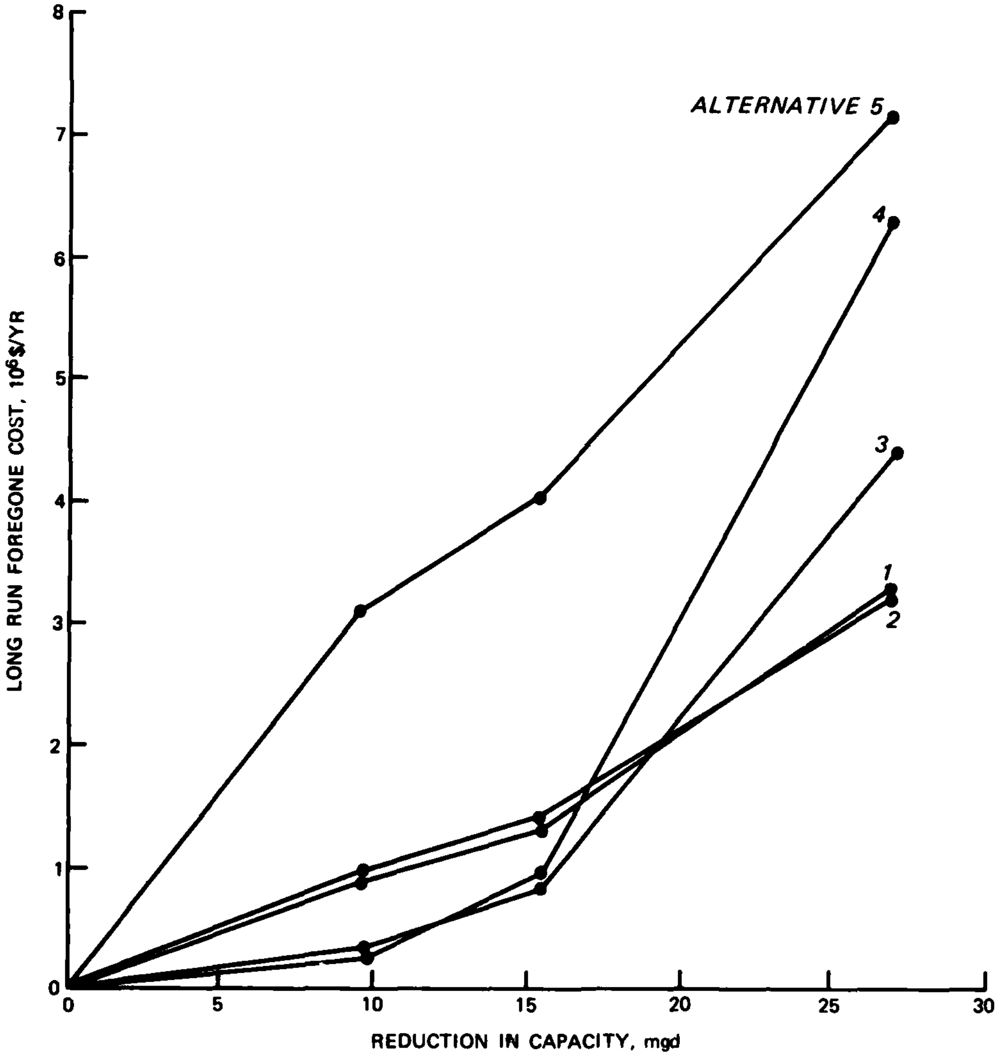


Figure 4-5. Smoothed Long Run Cost Functions

be better represented by functions with a break at the flow corresponding to a decision to build or not to build a dam as shown in Figure 4-6. This would require making cost estimates for a given use rate with and without the dam.

The short run foregone cost shows up primarily in savings in pumping energy at the wells or a reduction in water purchased. If measures affecting short run cost affect purchased water, the short run savings can be given as

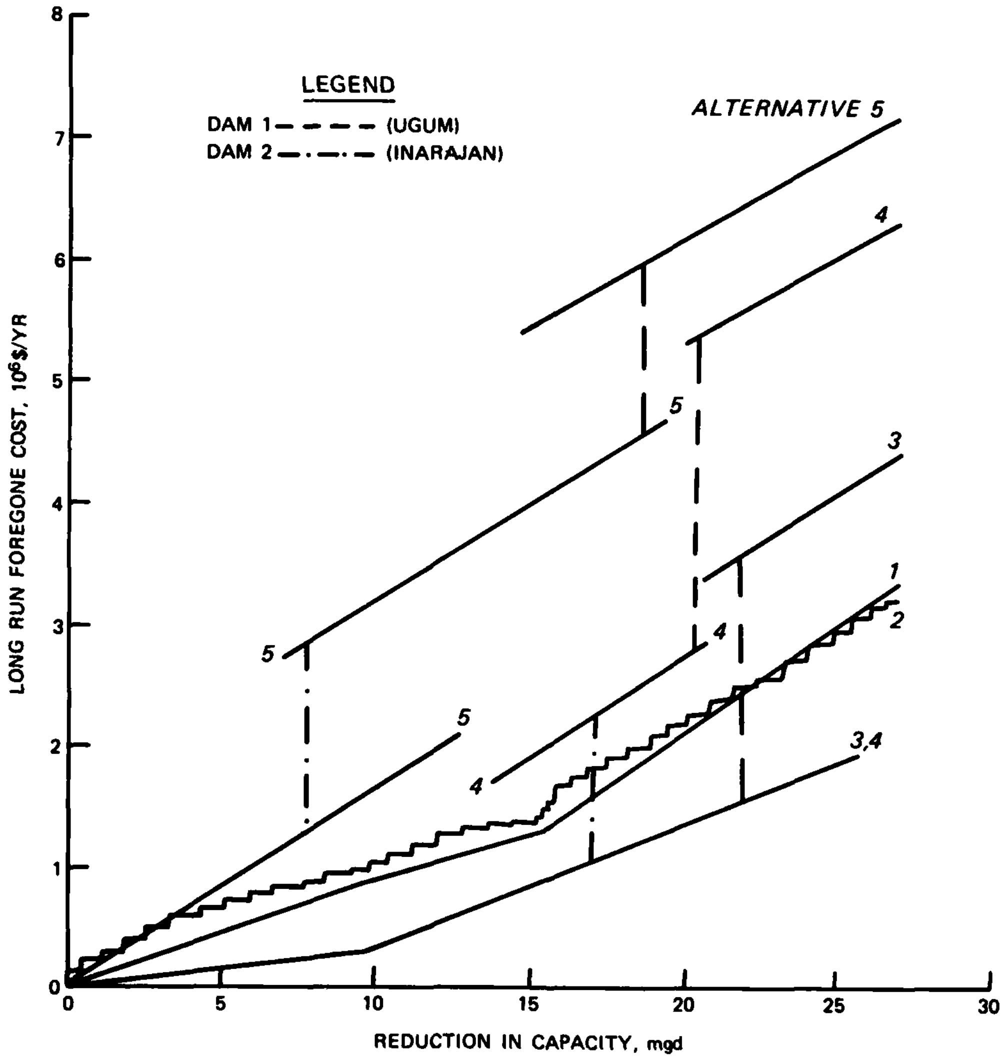


Figure 4-6. Actual Long Run Cost Functions

$$(\Delta Q)(\$1120/mg)\ t$$

where

Q = water use reduction, mgd

t = number of days water use is reduced, days

In the case of well water, the cost is

$$(\Delta Q)\ \frac{(23{,}000)\ (0.8)}{(0.29)\ (365)}\ t = 174\ (\Delta Q)t$$

based on \$23,000/yr O&M for each well

0.29 mgd yield per well

0.8 fraction of well O&M for energy

5. Summary

In Part II of this report, the facilities required for the five types of plans presented in Part I were identified. Preliminary designs for many of the facilities were available in the Master Plan and Ugum River Report. For those treatment and transmissions facilities not included in those documents, planning level designs were prepared and presented in this report.

Staging of construction was determined for each type of plan under three water use projections. Cost estimates, including both capital and O&M costs, were prepared for each major facility. The average annual cost of each alternative was then calculated.

In general, plans involving primarily development of groundwater proved to be more economical than those involving development of large dams, provided adequate groundwater is available.

Appendix A:

Proposed Capital Improvements Grouped into

5-Yr Construction Periods*

CONSTRUCTION PERIOD 1980 TO 1985

Supply Improvements (1980-85)

Service Area	Project	Description/Location	Number	Estimated 1980 Cost
A	AW-1	Construct first phase of well program along Routes 1, 3, and 9 and Y-Sengsong Road.	11	$2,200,000
B	BW-1	Construct wells within the area enclosed by Routes 4, 8, and 10.	13	2,600,000
	TOTAL SUPPLY IMPROVEMENTS			$4,800,000

Storage Improvements (1980-85)

Service Area	Project	Location	Capacity	Estimated 1980 Cost
A	AR-1	Site of the present Barrigada Reservoir.	3.0	$ 610,000
	AR-2	Site of the present Dededo Ground Reservoir.	2.0	505,000
	AR-3	Site of the present Dededo Ground Reservoir.	2.0	505,000
B	BR-3	Site of the present Mangilao Reservoir.	2.0	505,000
	BR-6	Site of the present Agana Heights Reservoir.	2.0	505,000
D	DR-1	West of Yona.	2.0	505,000
	TOTAL STORAGE IMPROVEMENTS			$3,135,000

* Barrett, Harris, and Associates (1979).

Transmission Main Improvements (1980-85)

Service Area	Project	Location	Size	Length (ft)	Estimated 1980 Cost
A	A-7	From the "normally closed" valve between Wells D-11 and D-6 south to the Dededo Ground Reservoir.	12"	4000	$ 200,000
	A-8	From the end of A-4, west along Route 1 to Dededo where connection is made to the existing 14" main.	16"	7500	465,000
	A-9	From Dededo, south to Latte Heights Subdivision	12"	7000	350,000
	A-10	From the Dededo Jr. High School, east along West Santa Monica to the end of A-6, then south along Y-Sengsong Rd. to Route 1.	16"	6000	372,000
B	B-15	From Bien Venida, northwest along Gibson Rd. to the Agana Heights Reservoir.	8"	3500	133,000
	B-16	South, from Bien Venida, along Gibson Rd. to Route 4, just north of Afami Rd.	14"	2500	125,000
	B-23	From the junction of the 8" line with the 12" line along Route 15 (near Mangilao) south past the Mangilao Reservoir and Washington High School, then west to Route 10.	16"	9000	558,000
D	D-18	From the new Yona Reservoir to Yona.	18"	10,000	700,000
	TOTAL TRANSMISSION MAIN IMPROVEMENTS				$2,903,000

Pressure Regulating Station Improvements (1980-85)

Service Area	Project	Estimated 1980 Cost
B	BPR-1	$ 15,000
	BPR-2	7,000
	BPR-4	1,000
	TOTAL PRESSURE REGULATING STATION IMPROVEMENTS	$ 23,000

Miscellaneous System Improvements (1980-85)

Service Area	Project	Description/Location	Estimated 1980 Cost
A	AM-3	Rehabilitate or dismantle and remove Dededo Elevated Reservoir.	$ 60,000
	ABM-1	Repair inoperable pump control valves at PUAG's existing 62 wells, including the replacement of parts as necessary.	255,000
	ABM-2	Construction of emergency standby generator hookups at 36 existing wells and the purchase of eighteen (18) portable standby generators.	705,000
	ABM-3	Construction of 15 of the proposed 25 emergency standby generators with typhoon proof buildings to serve a portion of the existing PUAG well supply.	1,575,000
	ABM-4	Construction of chlorination buildings to house chlorination equipment at thirty well stations.	185,000
	ABM-5	Sandblast and paint three 0.5 mg steel reservoirs and seven 1.0 mg steel reservoirs	575,000
	ABM-6	Preparation of a report to study the condition and usability of existing water storage reservoir level monitoring equipment and to indicate additional level monitoring equipment requirements.	25,000
	ABM-7	Install level monitoring and telemetry equipment at major water storage reservoirs.	510,000
	ABM-8	Provide security fencing at major water storage reservoirs.	125,000
B	BM-1	Miscellaneous site improvements at the Tumon Loop Reservoir.	204,000
	BM-2	Construct pressure sensing pump controls at Asan Spring and water level controls at Piti Reservoir.	20,000
D	DM-1	Construction of a new La Sa Fua raw water intake and construction of new Umatac Water Treatment Plant with a capacity of approximately 150 to 200 gpm.	518,000

Miscellaneous System Improvements (1980-85) (continued)

Service Area	Project	Description/Location	Estimated 1980 Cost
D	DM-3	Construction of the Ylig Water Treatment Plant and raw water intake facilities of approximately 350 50 gpm.	$ 1,495,000
	TOTAL MISCELLANEOUS SYSTEM IMPROVEMENTS		$ 6,252,000
	TOTAL WATER FACILITIES IMPROVEMENTS (1980-85)		$17,113,000

CONSTRUCTION PERIOD 1986-1990

Supply Improvements (1986-90

Service Area	Project	Location	Number	Estimated 1980 Cost
A	AW-1	Construct second phase of well program along Routes 1, 3, and 9 and Y-Sengsong Road	20	$ 4,000,000
	TOTAL SUPPLY IMPROVEMENTS			$ 4,000,000

Storage Improvements (1986-90)

Service Area	Project	Location	Capacity (mg)	Estimated 1980 Cost
A	AR-5	Site of the present Yigo Reservoir	2.0	$ 505,000
B	BR-1	Site of the present Tumon Loop Reservoir	2.0	505,000
	BR-2	Site of the present Tumon Reservoir	2.0	505,000
	TOTAL STORAGE IMPROVEMENTS			$1,515,000

Transmission Main Improvements (1986-90)

Service Area	Project	Location	Size	Length (ft)	Estimated 1980 Cost
A	A-4	From Yigo Reservoir south along Route 1 to the end of the existing 12" line at the Ypapao Subdivision entrance.	12"	22750	$1,138,000
	A-5	From the existing Y-Sengsong BPS north to the intersection of Route 3, then north along Route 3 and Route 9 to Route 1 at the Yigo Reservoir	16"	36000	2,232,000
	AB-2	From the intersection of A-5 with Route 3, south along Route 3, through the Liguan Terrace Subdivision area, west to Route 1, south past the Tumon Loop Reservoir to the intersection with the existing 14" water main.	24"	36750	3,602,000

Transmission Main Improvements (1986-90) (continued)

Service Area	Project	Location	Size	Length (ft)	Estimated 1980 Cost
A	AB-3	From the Dededo Ground Reservoir south to Route 1 then west and south along Route 1, parallel to the existing 14" line to the intersection with Route 1.	8"	15500	$ 589,000
B	B-1	From the Guam Reef Hotel, southeast along San Vitores Rd. to the Tumon Loop Reservoir	16"	4000	248,000
	B-2	From the junction of AB-2 and AB-3, southwesterly along Route 1 to the normally closed valve in the existing 14" line along Route 1.	20"	10500	861,000
	B-4	From Route 1 along Airport Road to Tumon Reservoir.	12"	2750	138,000
	B-13	From Piti, southwest along Route 1 to Route 6.	20"	2500	205,000
	B-18	From Coreana Rd. junction with Route 8 east along Route 8 to Canada Toto Road.	16"	2500	155,000
D	D-1	From Sanchez School, north along Route 2 to the Water Treatment Plant near the La Sa Fua River	6"	10800	346,000
	D-14	From Brigate Booster Pump Station No. 1 and No. 2 to the existing 6" line from the Ylig Water Treatment Plant.	12"	3200	240,000
	D-15	From Ylig Water Treatment Plant to Project D-14.	12"	3200	160,000
	D-16	From the junction of existing 12" and 6" lines near Ylig Bay to junction with Project D-18 and D-19.	12"	3200	160,000
	TOTAL TRANSMISSION MAIN IMPROVEMENTS				$10,074,000

Pressure Regulating Station Improvements (1986-90)

Service Area	Project	Estimated 1980 Cost
A	APR-1	$ 16,000
	APR-2	10,000
	APR-3	8,000
B	BPR-3	27,000
D	DPR-1	5,000
	TOTAL PRESSURE REGULATING STATION IMPROVEMENTS	$ 66,000

Miscellaneous System Improvements (1986-90)

Service Area	Project	Description/Location	Estimated 1980 Cost
A	ABM-3	Construction of the remaining 10 emergency standby generators with typhoon-proof buildings to serve a total of 35 existing PUAG wells.	$ 1,040,000
D	DM-4	Construction of Geus River Water Treatment Plant improvements with a capacity of 75 to 150 gpm.	400,000
	TOTAL MISCELLANEOUS SYSTEM IMPROVEMENTS		$ 1,440,000
	TOTAL WATER FACILITIES IMPROVEMENTS (1986-90)		$17,095,000

CONSTRUCTION PERIOD 1991 TO 1995

Supply Improvements (1991-95)

Service Area	Project	Description/Location	Number	Estimated 1980 Cost
A	AW-1	Construct third phase of well program along Routes 1, 3, and 9 and Y-Sengsong Road.	20	$4,000,000
	TOTAL SUPPLY IMPROVEMENTS			$4,000,000

Storage Improvements (1991-95)

Service Area	Project	Location	Capacity	Estimated 1980 Cost
B	BR-4	Site of the present Chaot Reservoir.	1.0	$ 400,000
	BR-5	Near the junction of Toto Road and Route 8.	2.0	505,000
C	CR-1	Pagachao Subdivision.	1.0	400,000
	TOTAL STORAGE IMPROVEMENTS			$1,305,000

Transmission Main Improvements (1991-95)

Service Area	Project	Location	Size	Length (ft)	Estimated 1980 Cost
A	A-3	East along Gayierno Rd. from Marine Dr., then south through Takano Subdivision to the Junction of AB-1 and A-1.	12"	7750	$ 388,000
	AB-1	From a point on Route 15, approximately 1 mile south of Gayierno Rd., south along Route 15 to Route 10 near the Mangilao Reservoir.	16"	48500	3,007,000
B	B-10	From the "normally closed" valve on Route 1, near Ypao Rd. southwest along Route 1 to Route 4.	18"	14500	1,015,000
	B-11	West along Route 1 from Route 4 to Asan.	20"	14500	1,189,000
	B-12	From Asan west along Route 1 to Piti.	16"	9500	589,000

Transmission Main Improvements (1991-95) (continued)

Service Area	Project	Location	Size	Length (ft)	Estimated 1980 Cost
B	B-17	From the junction of Routes 1 and 8, east along Route 8 to Careana Road.	12"	8000	$ 400,000
	B-22	From the junction of the existing 12" and 8" lines, approximately 2500 feet east of the Barrigada Reservoir south through Latte Heights to the Well M-2 area, then east past Well M-3, M-4, and M-8 to Route 15.	16"	8000	496,000
	BD-1	South along Route 4 from the junction of Routes 10 and 4, to the Pago Booster Pump Station.	16"	8500	527,000
C	C-1	From Route 2 at the Pagachao Subdivision entrance to the proposed reservoir in Pagachao Subdivision.	12"	3750	188,000
D	D-12	From the junction of Routes 4A and 17 northwesterly along Route 17 to the Cross Island Booster Pump Station.	18"	1000	700,000
	D-17	From the junction of the existing 12" and 6" lines near Ylig Bay north along Route 4 to Yona.	16"	5250	326,000
	D-19	From Yona to the Pago Booster Pump.	16"	5000	310,000
	TOTAL TRANSMISSION MAIN IMPROVEMENTS				$9,135,000

Booster Pump Station Improvements (1991-95)

Service Area	Project	Location	Capacity (gpm)	Estimated 1980 Cost
B	BPS-1	At the boundary between water Service Areas "A" and "B" near Latte Heights. Pumps water from the lower Dededo pressure zone to the higher Yigo pressure zone in Line B-22.	2000	$ 200,000
	BPS-2	Along Route 1 at west edge of Agana. Boosts pressure to allow flow into Piti Reservoir.	3350	265,000
	TOTAL BOOSTER PUMP STATION IMPROVEMENTS			$ 465,000

Pressure Regulating Station Improvements (1991-95)

Service Area	Project	Estimated 1980 Cost
B	BPR-6	$ 13,000
	TOTAL PRESSURE REGULATING STATION IMPROVEMENTS	$ 13,000

Miscellaneous System Improvements (1991-95)

Service Area	Project	Description/Location	Estimated 1980 Cost
A	AM-1	Construction of 8500 feet of 6" water main, 4500 feet of 8" water main, and a hydropneumatic booster pump station with fire pump in the Route 15-Mount Santa Rosa area.	$ 710,000
	AM-2	Abandon existing 4" water main along Gayierno Rd. and Route 1 in Yigo and construct water service reconnections as required.	88,000
	AM-4	Construct 4500 feet of 6" water main, 3500 feet of 12" water main, and two pressure regulating stations in the Harmon Village Area. Dismantle and remove existing steel reservoir.	615,000
C	CM-1	Replace water service laterals in Santa Rosa (Hyundai) subdivision with non-corrosive water service laterals.	450,000
D	DM-2	Construction of Laelae (Piga) Springs improvements and water treatment plant with capacity of approximately 75 to 150 gpm.	523,000
	TOTAL MISCELLANEOUS SYSTEM IMPROVEMENTS		$ 2,386,000
	TOTAL WATER FACILITIES IMPROVEMENTS (1991-95)		$17,304,000

CONSTRUCTION PERIOD 1996 TO 2000

Supply Improvements (1996-2000)

Service Area	Project	Description/Location	Number	Estimated 1980 Cost
A	AW-1	Construct fourth phase of well program along Routes 1, 3, and 9 and Y-Sengsong Road.	20	$4,000,000
	TOTAL SUPPLY IMPROVEMENTS			$4,000,000

Storage Reservoir Improvements (1996-2000)

Service Area	Project	Location	Capacity (mg)	Estimated 1980 Cost
A	AR-4	Mt. Santa Rosa.	1.0	$ 400,000
B	BR-7	At the site of the present Piti Reservoir.	2.0	505,000
	BR-8	Near the existing 6" connection to the 14" Navy line east of Nimitz Hill.	0.2	308,000
D	DR-2	Route 17 west of Windward Hills.	0.2	308,000
	TOTAL STORAGE IMPROVEMENTS			$1,521,000

Transmission Main Improvements (1996-2000)

Service Area	Project	Location	Size	Length (ft)	Estimated 1980 Cost
A	A-1	From the intersection of Gayierno Road and Takano Subdivision entrance east along Route 15 approximately two miles to the point of connection with Project AB-1.	6"	7250	$ 276,000
A	A-2	From the site of the proposed reservoir at Mt. Santa Rosa south along Route 15 to Gayierno Rd. to the point of connection with A-1.	12"	4500	225,000
	A-6	From the existing Y-Sengsong BPS south along Y-Sengsong Rd. to Dededo (Kaiser Housing).	12"	14000	700,000

Transmission Main Improvements (1996-2000) (continued)

Service Area	Project	Location	Size	Length (ft)	Estimated 1980 Cost
	A-11	From the Dededo Jr. High School, west along West Santa Monica to the connection with AB-2.	12"	3250	$ 163,000
	A-12	From the Harmon Village system, south to the intersection of AB-2.	8"	2500	95,000
B	B-3	From the Guam Reef Hotel, southwesterly along San Vitores Rd. to the junction with the road traversing northwest from JFK High School.	16"	7500	465,000
	B-5	From the Seventh Day Adventist Clinic, south along Ypao Rd. to Mamis Street, then west along Mamis and Espirito Streets to Hospital Rd.	12"	5250	263,000
	B-6	From the termination of B-3, west along San Vitores Road to Hospital Road.	12"	6000	300,000
	B-7	From San Vitores Road, south along Hospital Rd. to the intersection with Farenholt Avenue.	16"	2500	155,000
	B-8	From Hospital Rd., west along Farenholt Avenue to the junction with Camp Watkins Road, then south to the intersection of Route 1.	12"	4000	200,000
	B-9	South along Hospital Rd. from Farenholt Avenue to Route 1.	8"	4000	152,000
	B-14	From the junction of Routes 1 and 6, southeast along Route 6 to Nimitz Drive.	8"	6000	228,000
	B-19	From the junction of Route 8 and Canada Toto Rd. east along Route 8 to the intersection with Route 10, then south along Route 10 to the inter-intersection with Route 15.	12"	16000	800,000
	B-20	From the junction of Dairy Rd. and Route 10 west along Dairy Road to the junction with Route 4.	12"	15000	750,000
	B-21	From the junction of the existing 10" and 12" lines near the Barrigada Heights Reservoir, west to Route 16, then north on Route 16 for approximately 3500 feet.	12"	6500	325,000

Transmission Main Improvements (1996-2000) (continued)

Service Area	Project	Location	Size	Length (ft)	Estimated 1980 Cost
B	B-24	From the junction of University Avenue and Route 10, southwest along Route 10 to the junction with Route 4.	12"	7500	$ 375,000
C	C-2	From Kinsella Avenue to Juan Guerrero Street.	8"	2000	76,000
	C-3	From the junction of the 12" line (from Santa Rita) along Juan Guerrero Street, Herrara Street, and Carbuillido Street to the existing 12" line.	12"	2750	138,000
	C-4	From Santa Rosa Subdivision (Hyundai) east to the junction with Route 5.	8"	5500	209,000
	C-5	From the junction of the existing 10" and 12" lines near the Fena Water Treatment Plant, north along Route 5, through Talisay, to Route 17, then east to the Sinifa Reservoir access Road.	12"	12250	613,000
	CD-1	From the Cross Island Booster Pump Station to the Sinifa Reservoir access road.	16"	10000	620,000
D	D-2	From the Water Treatment Plant near Laelae Spring to Route 4.	6"	6000	192,000
	D-3	From Sanchez School to the Umatac Subdivision Reservoir.	12"	2000	100,000
	D-4	From the Umatac Subdivision Reservoir, south along Route 2 to approximately the Bile River.	6"	6250	200,000
	D-5	From the Bile River, south along Route 2 to the Pigua River.	8"	1000	38,000
	D-6	From Martyrs Memorial School to the Merizo Reservoir.	12"	1000	50,000
	D-7	From the junction of the existing 6" and 12" lines, south of Agfayan Bay, north along Route 2 to the Malojloj Booster Pump Station.	12"	25750	1,288,000

Transmission Main Improvements (1996-2000) (continued)

Service Area	Project	Location	Size	Length (ft)	Estimated 1980 Cost
D	D-8	From the Inarajan Reservoir to Asagas.	6"	1000	$ 62,000
	D-9	From the junction of Routes 4A and 4, northwest along Route 4A to the existing 6" main at Talofofo.	6"	5500	176,000
	D-10	Along Route 4A from Talofofo to the Windward Hills Reservoir No. 2.	16"	8500	527,000
	D-11	Along Route 4A from the junction of Routes 4A and 17 to Project D-10.	12"	3400	170,000
	D-13	Along Route 17 from the junction of Routes 4A and 17 to the junction of Routes 17 and 4.	16"	13000	806,000
	TOTAL TRANSMISSION MAIN IMPROVEMENTS				$10,737,000

Booster Pump Station Improvements (1996-2000)

Service Area	Project	Location	Capacity (gpm)	Estimated 1980 Cost
A	APS-1	On Gayierno Road near Marianas Terrace Sub-division. Pumps water to the Mt. Santa Rosa area.	350	$ 83,000
B	BPS-3	Along Route 6, between Piti School and Nimitz Hill. Provides the pressure needed to serve Nimitz Hill and Nimitz Hill Estates.	175	75,000
	BPS-4	Along Route 6 east of Nimitz Hill Estates provides the pressure needed to serve Nimitz Hill and Nimitz Hill Estates.	25	25,000
D	DPS-1	At present site of Brigade Booster Pump Stations 1 and 2, along Route 17, west of Windward Hills. Pumps water to Wind-ward Hills.	3000	250,000
	DPS-2	Along Route 17 west of Windward Hills. Pumps water to Sinifa Reservoir.	1750	190,000
	DPS-3	Along Route 4 in the vicinity of Toguan Bay. Pumps water from Merizo to Umatac.	100	55,000
	TOTAL BOOSTER PUMP STATION IMPROVEMENTS			$ 678,000

Pressure Regulating Station Improvements (1996-2000)

Service Area	Project	Estimated 1980 Cost
A	APR-4	$ 3,000
B	BPR-5	2,000
C	CPR-1	9,000
	CPR-2	4,000
	CPR-3	7,000
	CPR-4	4,000
	CPR-5	7,000
D	DPR-2	1,000
	DPR-3	4,000
	DPR-4	2,000
	DPR-5	2,000
	TOTAL PRESSURE REGULATING STATION IMPROVEMENTS	$ 45,000
	TOTAL WATER FACILITIES IMPROVEMENTS (1996-2000)	$16,981,000

APPENDIX B: TYPICAL OUTPUT FROM MAPS PIPELINE ROUTINE

This appendix contains printouts from the MAPS pipeline module for two pipelines: (1) Ugum Dam to Malojloj and (2) Inarajan Dam to Malojloj. For each pipe, nine different pipe diameters which would result in reasonable velocities are investigated. For each pipe size, the head losses and requirements are determined and the cost is calculated. The head requirements are then used to size pumping equipment and to determine its capital and O&M cost. Finally, a table giving the average annual cost for each size is printed. From the final printout, the optimal pipe size is selected based on life-cycle costs.

For the Ugum pipeline, the 24-in. pipe is clearly the best. For the Inarajan pipeline, either a 20- or 24-in. pipe would cost about the same. A 24-in. pipe is selected because it requires the least pumping energy, and energy costs are more likely to increase more than other costs over the life of the project.

Note that the velocity at optimal pipe size is 4.4 ft/sec for the Ugum pipe and 3.4 ft/sec for the Inarajan pipe. In the Master Plan, 6 ft/sec is used as a rule-of-thumb for pipe sizing. As is shown in this appendix, the energy costs, in lines that are generally flowing at capacity, would be too great using that rule.

UGUM FOR 9.0 &6.9

PIPE LINE WITH FORCE MOD 20
AND PIPE MOD 20
DETAILED OUTPUT, SUMMARY OR END?
1 OUTPUT FOR FORCE MAIN NO 20

UGUM-MALOJLOJ (S-3)

MAXIMUM FLOW- STAGE 1	.900E+01	MGD
AVERAGE FLOW- STAGE 1	.900E+01	MGD
LENGTH	.120E+05	FT
LENGTH	.227E+01	MI
INITIAL ELEVATION	.270E+03	FT
INITIAL PRESSURE HEAD	0.	FT
FINAL ELEVATION	.340E+03	FT
FINAL PRESSURE HEAD	0.	FT
ROUGHNESS HEIGHT	.400E-03	FT
ALLOWABLE PRESSURE IN PIPE	.200E+03	FT
RECTANGULAR TRENCH		
DEPTH OF COVER	.300E+01	FT
DRY SOIL CONDITIONS		
TYPE OF PIPE		

DUCTILE IRON PIPE IS USED FOR ALL DIAMETERS

HYDRAULIC ANALYSIS AT PEAK FLOW (FIRST STAGE)
13.923 CFS 9.000 MGD

DIAM (IN)	VELOCITY (FPS)	VELOCITY HEAD (FT)	MINOR LOSSES (FT)	FRICTION LOSSES (FT)	HEAD REQUIRED (FT)
14.0	.130E+02	.264E+01	0.	.467E+03	.537E+03
16.0	.997E+01	.155E+01	0.	.236E+03	.326E+03
18.0	.788E+01	.965E+00	0.	.129E+03	.199E+03
20.0	.638E+01	.633E+00	0.	.757E+02	.146E+03
24.0	.443E+01	.305E+00	0.	.301E+02	.100E+03
30.0	.284E+01	.125E+00	0.	.979E+01	.798E+02
36.0	.197E+01	.603E-01	0.	.394E+01	.739E+02
42.0	.145E+01	.325E-01	0.	.183E+01	.718E+02
48.0	.111E+01	.191E-01	0.	.944E+00	.709E+02

NO SECOND STAGE

CONSTRUCTION YEAR-STAGE 1	1980	
INTEREST RATE	7.625	%
DESIGN LIFE	50	YEARS
ENR CONSTRUCTION INDEX	3200.0	
LAND COST	0.	$
CITY MULTIPLIER	1.500	
TERRAIN TYPE--		

DIAM (IN)	PIPE COSTS ($)	OTHER COSTS ($)	CONSTRUCTION COSTS ($)	OVERHEAD COSTS ($)	OPERATION & MAINT. ($/YR)
14.0	.3806E+06	.9612E+05	.4767E+06	.1192E+06	.1474E+04
16.0	.4598E+06	.1142E+06	.5739E+06	.1435E+06	.1689E+04
18.0	.5432E+06	.1332E+06	.6764E+06	.1691E+06	.1911E+04
20.0	.6305E+06	.1531E+06	.7837E+06	.1959E+06	.2140E+04
24.0	.8162E+06	.2034E+06	.1020E+07	.2549E+06	.2642E+04
30.0	.1119E+07	.2769E+06	.1396E+07	.3491E+06	.3421E+04
36.0	.1449E+07	.3570E+06	.1806E+07	.4515E+06	.4255E+04
42.0	.1802E+07	.4430E+06	.2245E+07	.5613E+06	.5140E+04
48.0	.2177E+07	.5346E+06	.2712E+07	.6780E+06	.6072E+04

FORCE MAIN COST SUMMARY
MOD NO. 20

DIAM (IN)	CAPITAL COST ($)	O&M COST ($/YR)	AVERAGE ANNUAL COST ($/YR)
14.0	.596E+06	.147E+04	.481E+05
16.0	.717E+06	.169E+04	.578E+05
18.0	.845E+06	.191E+04	.681E+05
20.0	.980E+06	.214E+04	.788E+05
24.0	.127E+07	.264E+04	.102E+06
30.0	.175E+07	.342E+04	.140E+06
36.0	.226E+07	.426E+04	.181E+06
42.0	.281E+07	.514E+04	.225E+06
48.0	.339E+07	.607E+04	.271E+06

1 OUTPUT FOR PUMP STATION NO. 20

UGUM RW PUMP (S-3)

MAXIMUM FLOW(STAGE 1)	.900E+01 MGD
AVERAGE FLOW(STAGE 1)	.900E+01 MGD
REQUIRED HEAD BASED ON FORCE MAIN MCD 20	
RAW OR TREATED WATER PUMPING	
YEAR BUILT	1980
DESIGN LIFE	50 YEARS
EFFICIENCY OF PUMP AND MOTOR	.600E+02 PERCENT
MAXIMUM HEAD PER STATION	.100E+04 FT
NO. OF STATIONS DETERMINED BY PROGRAM	
NO. PUMPS PER STATION-STAGE 1	2
NO WET WELL	
IMPROVED STRUCTURE	
DOWNTIME	0.0 PERCENT

ECONOMIC OUTPUT

INTEREST RATE	.763E+01 PERCENT
ENR INDEX	.320E+04
CITY MULTIPLIER	.150E+01
O&M WAGE	.100E+02 $/HR
COST OF ELECTRICITY	.600E-01 $/KWHR
COST OF LAND SITE IMPROVEMENT	0. $

COST OF STRUCTURE AND SWITCHYARD FOR SINGLE STATION
COST BASED ON 9.00 MGD, BUILT IN 1980

DIAM	NO. OF STATIONS	POWER CAPACITY (KVA)	STRUCTURE COSTS ($)	SWITCHYARD COSTS ($)
14.0	1	.134E+04	.213E+06	0.
16.0	1	.771E+03	.140E+06	0.
18.0	1	.511E+03	.103E+06	0.
20.0	1	.380E+03	.819E+05	0.
24.0	1	.269E+03	.629E+05	0.
30.0	1	.219E+03	.539E+05	0.
36.0	1	.205E+03	.512E+05	0.
42.0	1	.200E+03	.502E+05	0.
48.0	1	.198E+03	.498E+05	0.

COSTS FOR MECHANICAL AND ELECTRICAL EQUIPMENT FOR SINGLE STATION
COSTS FOR STAGE 1 BASED ON .900E+01 MGD, BUILT IN 1980

DIAM (IN)	HEAD PER STATION (FT)	MECHANIC COST ($)	ELECTRIC COST ($)	MISC COST ($)	CONSTRUCT COST ($)	OVERHEAD COST ($)
14.	.547E+03	.140E+06	.118E+06	.139E+06	.793E+06	.198E+06
16.	.316E+03	.112E+06	.910E+05	.139E+06	.628E+06	.157E+06
18.	.209E+03	.953E+05	.749E+05	.139E+06	.536E+06	.134E+06
20.	.156E+03	.847E+05	.652E+05	.139E+06	.482E+06	.121E+06
24.	.110E+03	.737E+05	.553E+05	.139E+06	.431E+06	.108E+06
30.	.898E+02	.679E+05	.503E+05	.139E+06	.405E+06	.101E+06
36.	.839E+02	.661E+05	.487E+05	.139E+06	.397E+06	.992E+05
42.	.818E+02	.655E+05	.481E+05	.139E+06	.394E+06	.985E+05
48.	.809E+02	.652E+05	.479E+05	.139E+06	.393E+06	.982E+05

OPERATION AND MAINTENANCE COSTS FOR SINGLE PUMP STATION
COSTS FOR STAGE 1 BASED ON .900E+01 MGD FROM 1980 TO 2030
SUPPLY COST .521E+04 $/YR
LABOR COST .136E+05 $/YR

DIAM (IN)	HEAD REQUIRED (FT)	POWER REQUIRED (KWHR/YR)	POWER COST ($/YR)	TOTAL O&M ($/YR)
14.0	.537E+03	.937E+07	.562E+06	.581E+06
16.0	.306E+03	.541E+07	.325E+06	.343E+06
18.0	.199E+03	.359E+07	.215E+06	.234E+06
20.0	.146E+03	.267E+07	.160E+06	.179E+06
24.0	.100E+03	.189E+07	.113E+06	.132E+06
30.0	.798E+02	.154E+07	.923E+05	.111E+06
36.0	.739E+02	.144E+07	.863E+05	.105E+06
42.0	.718E+02	.140E+07	.841E+05	.103E+06
48.0	.709E+02	.139E+07	.832E+05	.102E+06

1 PUMP STATION COST SUMMARY
MOD NO. 20

DIAM (IN)	NO. OF STATIONS	STAGE 1 CAPITAL COST ($)	STAGE 1 O&M COST ($/YR)	STAGE 2 CAPITAL COST ($)	STAGE 2 O&M COST ($/YR)	AVERAGE ANNUAL COST ($/YR)
14.0	1	.991E+06	.581E+06	0.	0.	.658E 06
16.0	1	.784E+06	.343E+06	0.	0.	.405E+06
18.0	1	.670E+06	.234E+06	0.	0.	.286E+06
20.0	1	.603E+06	.179E+06	0.	0.	.226E+06
24.0	1	.538E+06	.132E+06	0.	0.	.174E+06
30.0	1	.506E+06	.111E+06	0.	0.	.151E+06
36.0	1	.496E+06	.105E+06	0.	0.	.144E+06
42.0	1	.492E+06	.103E+06	0.	0.	.141E+06
48.0	1	.491E+06	.102E+06	0.	0.	.140E+06

PIPELINE COST SUMMARY
FORCE MAIN MOD 20
PUMP STATION MOD 20

DIAM (IN)	AMORTIZED CONSTRUCTION COST(PIPE) ($/YR)	O&M COST (PIPE) ($/YR)	AMORTIZED CONSTRUCTION COST(PUMP) ($/YR)	O&M COST (PUMP) ($/YR)	AVERAGE ANNUAL COST ($/YR)
14.0	.466E+05	.147E+04	.775E+05	.581E+06	.706E+06
16.0	.561E+05	.169E+04	.614E+05	.343E+06	.463E+06
18.0	.661E+05	.191E+04	.524E+05	.234E+06	.354E+06
20.0	.766E+05	.214E+04	.472E+05	.179E+06	.305E+06
24.0	.997E+05	.264E+04	.421E+05	.132E+06	.276E+06←
30.0	.137E+06	.342E+04	.396E+05	.111E+06	.291E+06
36.0	.177E+06	.426E+04	.388E+05	.105E+06	.325E+06
42.0	.220E+06	.514E+04	.385E+05	.103E+06	.366E+06
48.0	.265E+06	.607E+04	.384E+05	.102E+06	.412E+06

PIPE LINE WITH FORCE MOD 21
AND PIPE MOD 21
DETAILED OUTPUT, SUMMARY OR END?
1 OUTPUT FOR FORCE MAIN NO 21

INARAJAN-MALOJLOJ (T-9)

MAXIMUM FLOW- STAGE 1	.690E+01	MGD
AVERAGE FLOW- STAGE 1	.690E+01	MGD
LENGTH	.670E+04	FT
LENGTH	.127E+01	MI
INITIAL ELEVATION	.960E+02	FT
INITIAL PRESSURE HEAD	0.	FT
FINAL ELEVATION	.340E+03	FT
FINAL PRESSURE HEAD	0.	FT
ROUGHNESS HEIGHT	.400E-03	FT
ALLOWABLE PRESSURE IN PIPE	.200E+03	FT
RECTANGULAR TRENCH		
DEPTH OF COVER	.300E+01	FT
DRY SOIL CONDITIONS		
TYPE OF PIPE		
DUCTILE IRON PIPE IS USED FOR ALL DIAMETERS		

HYDRAULIC ANALYSIS AT PEAK FLOW (FIRST STAGE)
10.674 CFS 6.900 MGD

DIAM (IN)	VELOCITY (FPS)	VELOCITY HEAD (FT)	MINOR LOSSES (FT)	FRICTION LOSSES (FT)	HEAD REQUIRED (FT)
12.0	.136E+02	.287E+01	0.	.341E+03	.585E+03
14.0	.999E+01	.155E+01	0.	.155E+03	.399E+03
16.0	.764E+01	.908E+00	0.	.785E+02	.323E+03
18.0	.604E+01	.567E+00	0.	.432E+02	.287E+03
20.0	.489E+01	.372E+00	0.	.253E+02	.269E+03
24.0	.340E+01	.179E+00	0.	.101E+02	.254E+03
30.0	.217E+01	.735E-01	0.	.330E+01	.247E+03
36.0	.151E+01	.354E-01	0.	.133E+01	.245E+03
42.0	.111E+01	.191E-01	0.	.621E+00	.245E+03

NO SECOND STAGE

CONSTRUCTION YEAR-STAGE 1	1980	
INTEREST RATE	7.625	%
DESIGN LIFE	50	YEARS
ENR CONSTRUCTION INDEX	3200.0	
LAND COST	0.	$
CITY MULTIPLIER	1.500	
TERRAIN TYPE--		

DIAM (IN)	PIPE COSTS ($)	OTHER COSTS ($)	CONSTRUCTION COSTS ($)	OVERHEAD COSTS ($)	OPERATION & MAINT. ($/YR.)
12.0	.2012E+06	.5022E+05	.2514E+06	.6285E+05	.8281E+03
14.0	.2125E+06	.5366E+05	.2662E+06	.6654E+05	.8232E+03
16.0	.2567E+06	.6375E+05	.3205E+06	.8011E+05	.9428E+03
18.0	.3033E+06	.7438E+05	.3777E+06	.9441E+05	.1067E+04
20.0	.3521E+06	.8551E+05	.4376E+06	.1094E+06	.1195E+04
24.0	.4557E+06	.1136E+06	.5693E+06	.1423E+06	.1475E+04
30.0	.6250E+06	.1546E+06	.7796E+06	.1949E+06	.1910E+04
36.0	.8090E+06	.1993E+06	.1008E+07	.2521E+06	.2376E+04
42.0	.1006E+07	.2473E+06	.1254E+07	.3134E+06	.2870E+04

FORCE MAIN COST SUMMARY
MOD NO. 21

DIAM (IN)	CAPITAL COST ($)	O&M COST ($/YR)	AVERAGE ANNUAL COST ($/YR)
12.0	.314E+06	.828E+03	.254E+05
14.0	.333E+06	.823E+03	.269E+05
16.0	.401E+06	.943E+03	.323E+05
18.0	.472E+06	.107E+04	.380E+05
20.0	.547E+06	.120E+04	.440E+05
24.0	.712E+06	.147E+04	.571E+05
30.0	.975E+06	.191E+04	.782E+05
36.0	.126E+07	.238E+04	.101E+06
42.0	.157E+07	.287E+04	.125E+06

1 OUTPUT FOR PUMP STATION NO. 21

INARAJAN PUMP (P-4)

MAXIMUM FLOW(STAGE 1)	.690E+01 MGD
AVERAGE FLOW(STAGE 1)	.690E+01 MGD
REQUIRED HEAD BASED ON FORCE MAIN MOD 21	
RAW OR TREATED WATER PUMPING	
YEAR BUILT	1980
DESIGN LIFE	50 YEARS
EFFICIENCY OF PUMP AND MOTOR	.620E+02 PERCENT
MAXIMUM HEAD PER STATION	.100E+04 FT
NO. OF STATIONS DETERMINED BY PROGRAM	
NO. PUMPS PER STATION-STAGE 1	2
NO WET WELL	
IMPROVED STRUCTURE	
DOWNTIME	0.0 PERCENT

ECONOMIC OUTPUT

INTEREST RATE	.763E+01 PERCENT
ENR INDEX	.320E+04
CITY MULTIPLIER	.150E+01
O&M WAGE	.100E+02 $/HR
COST OF ELECTRICITY	.600E-01 $/KWHR
COST OF LAND SITE IMPROVEMENT	0. $

COST OF STRUCTURE AND SWITCHYARD FOR SINGLE STATION
COST BASED ON 6.90 MGD, BUILT IN 1980

DIAM	NO. OF STATIONS	POWER CAPACITY (KVA)	STRUCTURE COSTS ($)	SWITCHYARD COSTS ($)
12.0	1	.112E+04	.175E+06	0.
14.0	1	.767E+03	.132E+06	0.
16.0	1	.623E+03	.112E+06	0.
18.0	1	.557E+03	.103E+06	0.
20.0	1	.523E+03	.985E+05	0.
24.0	1	.495E+03	.944E+05	0.
30.0	1	.482E+03	.925E+05	0.
36.0	1	.478E+03	.920E+05	0.
42.0	1	.477E+03	.918E+05	0.

COSTS FOR MECHANICAL AND ELECTRICAL EQUIPMENT FOR SINGLE STATION
COSTS FOR STAGE 1 BASED ON .690E+01 MGD, BUILT IN 1980

DIAM (IN)	HEAD PER STATION (FT)	MECHANIC COST ($)	ELECTRIC COST ($)	MISC COST ($)	CONSTRUCT COST ($)	OVERHEAD COST ($)
12.	.595E+03	.113E+06	.105E+06	.123E+06	.671E+06	.168E+06
14.	.409E+03	.972E+05	.880E+05	.123E+06	.572E+06	.143E+06
16.	.333E+03	.895E+05	.798E+05	.123E+06	.527E+06	.132E+06
18.	.297E+03	.855E+05	.757E+05	.123E+06	.504E+06	.126E+06
20.	.279E+03	.834E+05	.735E+05	.123E+06	.492E+06	.123E+06
24.	.264E+03	.816E+05	.716E+05	.123E+06	.482E+06	.121E+06
30.	.257E+03	.807E+05	.707E+05	.123E+06	.478E+06	.119E+06
36.	.255E+03	.805E+05	.705E+05	.123E+06	.476E+06	.119E 06
42.	.255E+03	.804E+05	.704E+05	.123E+06	.476E+06	.119E+06

OPERATION AND MAINTENANCE COSTS FOR SINGLE PUMP STATION
COSTS FOR STAGE 1 BASED ON .690E+01 MGD FROM 1980 TO 2030
SUPPLY COST .406E+04 $/YR
LABOR COST .116E+05 $/YR

DIAM (IN)	HEAD REQUIRED (FT)	POWER REQUIRED (KWHR/YR)	POWER COST ($/YR)	TOTAL O&M ($/YR)
12.0	.585E+03	.782E+07	.469E+06	.485E+06
14.0	.399E+03	.538E+07	.323E+06	.338E+06
16.0	.323E+03	.437E+07	.262E+06	.278E+06
18.0	.287E+03	.390E+07	.234E+06	.250E+06
20.0	.269E+03	.367E+07	.220E+06	.236E+06
24.0	.254E+03	.347E+07	.208E+06	.224E+06
30.0	.247E+03	.338E+07	.203E+06	.219E+06
36.0	.245E+03	.335E+07	.201E+06	.217E+06
42.0	.245E+03	.335E+07	.201E+06	.216E+06

1 PUMP STATION COST SUMMARY
MOD NO. 21

DIAM (IN)	NO. OF STATIONS	STAGE 1 CAPITAL COST ($)	STAGE 1 O&M COST ($/YR)	STAGE 2 CAPITAL COST ($)	STAGE 2 O&M COST ($/YR)	AVERAGE ANNUAL COST ($/YR)
12.0	1	.839E+06	.485E+06	0.	0.	.551E+06
14.0	1	.715E+06	.338E+06	0.	0.	.394E+06
16.0	1	.658E+06	.278E+06	0.	0.	.329E+06
18.0	1	.630E+06	.250E+06	0.	0.	.299E+06
20.0	1	.615E+06	.236E+06	0.	0.	.284E+06
24.0	1	.603E+06	.224E+06	0.	0.	.271E+06
30.0	1	.597E+06	.219E+06	0.	0.	.265E+06
36.0	1	.595E+06	.217E+06	0.	0.	.264E+06
42.0	1	.595E+06	.216E+06	0.	0.	.263E+06

PIPELINE COST SUMMARY
FORCE MAIN MOD 21
PUMP STATION MOD 21

DIAM (IN)	AMORTIZED CONSTRUCTION COST(PIPE) ($/YR)	O&M COST (PIPE) ($/YR)	AMORTIZED CONSTRUCTION COST(PUMP) ($/YR)	O&M COST (PUMP) ($/YR)	AVERAGE ANNUAL COST ($/YR)
12.0	.246E+05	.828E+03	.657E+05	.485E+06	.576E+06
14.0	.262E+05	.823E+03	.560E+05	.338E+06	.421E+06
16.0	.313E+05	.943E+03	.515E+05	.278E+06	.362E+06
18.0	.369E+05	.107E+04	.493E+05	.250E+06	.337E+06
20.0	.428E+05	.120E+04	.482E+05	.236E+06	.328E+06
24.0	.557E+05	.147E+04	.472E+05	.224E+06	.328E+06
30.0	.762E+05	.191E+04	.467E+05	.219E+06	.343E+06
36.0	.986E+05	.238E+04	.466E+05	.217E+06	.365E+06
42.0	.123E+06	.287E+04	.465E+05	.216E+06	.368E+06

APPENDIX C: CALCULATING AVERAGE ANNUAL COST OF GROUNDWATER AND PURCHASED WATER

In this appendix, formulas are derived for calculating the average annual cost for construction, and operation and maintenance (O&M) of wells, given construction and O&M costs of a single well; and purchase of water, given the unit price to purchase water. It is assumed that the required water yield as a function of time ($Q(t)$) can be represented by a series of straight line segments of the form

$$Q(t) = a + bt$$

for

$$t_{k-1} < t \leq t_k$$

The variables used in the development are defined below

- a,b = regression coefficients for water use segments
- A = cost to operate well or buy water, $\$/\Delta t$
- B = unit price for well O&M or purchased water, \$/yr/mgd
- C = capital cost of well, \$
- F = defined in text
- i = interest rate (0.07625)
- k = index on segments
- m = number of segments
- N = number of wells operating in year t
- PW = present worth
- Q = water use, mgd
- R = $-\ell n\ (1 + i)$
- t = time, years
- t_k = time at end of k-th segment, yr
- U = cost to operate one well one year, \$/yr

Capital Cost of Wells

If the number of new* wells existing at time t is N, the rate at which they are built in wells per year is dN/dt. Since each well yields approximately 0.29 mgd, N can be related to flow by

$$N = \frac{Q(t)}{0.29}$$

Since the flow can be given by $Q = a + by$

$$N = \frac{a + bt}{0.29}$$

and

$$\frac{dN}{dt} = \frac{b}{0.29}$$

The number of wells built in a single year ($\Delta t = 1$) is, therefore,

$$\Delta N = \frac{dN}{dt}\,\Delta t = \frac{b\Delta t}{0.29}$$

If a single well costs C dollars, the cost to build wells in a given year is

$$\text{Cost} = \frac{bC\Delta t}{0.29}$$

The present worth of this cost is

$$PW = \frac{bC\Delta t}{0.29(1 + i)^t}$$

where

i = interest rate

$$t = \begin{cases} 0 \text{ in } 1985 \\ 50 \text{ in } 2035 \end{cases}$$

---

* "New" means built after 1985.

The average annual cost is

$$AAC = \frac{crf\ bC\Delta t}{0.29(1+i)^t}$$

where crf = capital recovery factor

The above cost is for wells built in year t . Since wells can be built for every year in the study period,

$$AAC = \frac{crf\ C}{0.29} \sum_{j=0}^{50} \frac{b\Delta t}{(1+i)^j}$$

Since time is a continuous function, it is more convenient to write the above as

$$AAC = \frac{crf\ C}{0.29} \int_0^{50} \frac{bdt}{(1+i)^t}$$

Since there are several line segments (say m), the above integration must be performed separately for each segment. Therefore,

$$AAC = \frac{crf\ C}{0.29R} \sum_{k=1}^{m} \left[b_k \int_{t_{k-1}}^{t_k} \frac{dt}{(1+i)^t} \right]$$

where $R = -\ell n(1+i)$

Integrating yields

$$AAC = \frac{crf\ C}{0.29R} \sum_{k=1}^{m} b_k \left[\frac{1}{(1+i)^{t_k}} - \frac{1}{(1+i)^{t_{k-1}}} \right]$$

For this study,

$crf_{7\ 5/8,50} = 0.0782$

$C = \$200,000$

$$1 + i = 1.0735^*$$

$$R = 0.0709$$

b_k, t_k are given in Table 3-7

m depends on the number of segments

Therefore,

$$AAC = -760{,}663 \sum_{k=1}^{m} b_k \left(\frac{1}{1.0735^{t_k}} - \frac{1}{1.0735^{t_{k-1}}} \right)$$

O&M and Purchase Cost

For O&M and purchase cost, the procedure is similar, except that the total number of wells or volume of water purchased rather than the rate of demand increase is important.

The cost, A, to operate N wells for a year ($\Delta t = 1$) can be given by

$$A = NU\Delta t$$

where

N = number of wells

U = unit cost

Since each well yields 0.29 mgd and the flow in any year is given by $Q = a + bt$,

$$A = \frac{QU\Delta t}{0.29} = \frac{(a + bt)U\Delta t}{0.29}$$

The cost to purchase water for one year ($\Delta t = 1$) can be given by

$$A = QP(365)(1000)\Delta t$$
$$= (a + bt)P365{,}000\Delta t$$

where P = price of water, \$/1000 gal

---

* Note that an effective continuous interest rate of 7.35% is used which corresponds to a discrete rate of 7.625%. The capital recovery factor is the same as it would be for the discrete rate as it was outside of the integral.

The cost to operate wells or purchase water for time Δt can be given by

$$A = (a + bt)B\Delta t$$

where

$$B = \begin{cases} (U/0.29) & \text{for wells} \\ 365{,}000P & \text{for purchase} \end{cases}$$

B has units of \$/yr/mgd

The present worth of this cost can be given by

$$PW = \frac{(a + bt)B\Delta t}{(1 + i)^t}$$

The average annual cost over the study period for water used in time Δt is

$$AAC = \frac{crf(a + bt)B\Delta t}{(1 + i)^t}$$

Since flow is a continuous function of time, Δt can approach 0 to give

$$AAC = crf\ B \int_0^{50} \frac{(a + bt)\ dt}{(1 + i)^t}$$

Since the 50-year study period can be divided into m segments with different values for a and b, the integration must be done separately for each segment. Therefore,

$$AAC = crf\ B \sum_{k=1}^{m} \left[\int_{t_{k-1}}^{t_k} \frac{(a_k + b_k t)dt}{(1 + i)^t} \right]$$

Integration by parts yields

$$AAC = \frac{crf\ B}{R} \sum_{k=1}^{m} \left[\frac{\left(a_k + b_k t_k - \frac{b_k}{R}\right)}{(1 + i)^{t_k}} - \frac{\left(a_k + b_k t_{k-1} - \frac{b_k}{R}\right)}{(1 + i)^{t_{k-1}}} \right]$$

where $R = -\ell n(1 + i)$

For this study,

$$crf_{7\ 5/8,50} = 0.0782$$
$$1 + i = 1.0735$$
$$R = -0.0709$$

$$B^* = \begin{cases} 23{,}000/(0.29)/(1.07625) = 73{,}691 \text{ for well O\&M} \\ \frac{365{,}000}{1.07625}(1.2) = 379{,}837 \text{ for purchase} \end{cases}$$

a,b,t are given in Tables 3-7 and 3-9

n depends on number of segments

This yields

$$AAC_{well} = -87618F$$
$$AAC_{pur} = -418945F$$

where

$$F = \sum_{k=1}^{m} \left[\frac{a_k + b_k(t_k + 13.6)}{(1.0735)^{t_k}} - \frac{a_k + b_k(t_{k-1} + 13.6)}{(1.0735)^{t_{k-1}}} \right]$$

Computer Program

The following pages contain the computer programs used to determine average annual cost. Program WELL was used for construction cost while program WELLO was used for O&M and purchase costs. The subroutine SCAN is merely used to make data entry easy. It is possible to not require SCAN if a formatted read statement for A, B, and IT2 is used in statement 2.

* The 1.07625 in the formula for B is to correct B for the fact that costs accrue continuously but are accounted for at the end of the year.

```
 LIST,F=WELL
      PROGRAM WELL(INPUT,OUTPUT,TAPE5=INPUT.TAPE6=OUTPUT)
C CALCULATES AVERAGE ANNUAL COST OF WELL CONSTRUCTION
      DIMENSION VALUE(10),KLM(74)
      C=-760663.
      RINT=1.07625
 1     IT1=0
      IT2=0
      T=0
 2     READ(5,3)KLM
 3     FORMAT(74A1)
      CALL SCAN(NO,VALUE,74,KLM)
      IF(VALUE(1).LT.-1000)STOP
      IF(VALUE(1).LT.-100)GO TO 4
      A=VALUE(1)
      B=VALUE(2)
      IT1=IT2
      IT2=VALUE(3)
      Z1=RINT**(-IT1)
      Z2=RINT**(-IT2)
      T=T+B*(Z2-Z1)
      WRITE(6,5)B,IT1,IT2,T
 5      FORMAT(13H B,IT1,IT2,T ,F8.3,2I4,F10.3)
      GO TO 2
 4     AAC=C*T
      WRITE(6,6)AAC
 6     FORMAT(6H AAC= ,F10.0)
      GO TO 1
      END
```

```
LIST,F=WELLO
      PROGRAM WELLO(INPUT,OUTPUT,TAPE5=INPUT,TAPE6=OUTPUT)
C CALCULATES AVERAGE ANNUAL COST OF WELL CONSTRUCTION
      DIMENSION VALUE(10),KLM(74)
      C=-73691.
      RINT=1.0735
 1     IT1=0
      IT2=0
      T=0
 2     READ(5,3)KLM
 3     FORMAT(74A1)
      CALL SCAN(NO,VALUE,74,KLM)
      IF(VALUE(1).LT.-1000)STOP
      IF(VALUE(1).LT.-100)GO TO 4
      A=VALUE(1)
      B=VALUE(2)
      IT1=IT2
      IT2=VALUE(3)
      Z1=RINT**(-IT1)
      Z2=RINT**(-IT2)
      Y1=A+B*(IT1+13.6)
      Y2=A+B*(IT2+13.6)
      T=T+(Y2*Z2-Y1*Z1)
      WRITE(6,5)B,IT1,IT2,T
 5      FORMAT(13H B,IT1,IT2,T ,F8.3,2I4,F10.3)
      GO TO 2
 4     AAC=C*T
      WRITE(6,6)AAC
 6     FORMAT(6H AAC= ,F10.0)
      GO TO 1
      END
```

```
C SCAN

      SUBROUTINE SCAN(NO,VALUE,M7,KLM)
      DIMENSION VALUE(10),KLM(76),NUM(10)
      DATA IPOINT,IPLUS,MINUS/1H.,1H+,1H-/
      DATA NUM/1H0,1H1,1H2,1H3,1H4,1H5,1H6,1H7,1H8,1H9/
      K7=M7+1
      DO 1 I=1,10
1     VALUE(I)=0.
      NCOL=1
      N=1
      KPT=0
2     IF(KLM(NCOL).NE.MINUS)GO TO 4
3     SGN=-1.
      GO TO 5
4     IF(KLM(NCOL).NE.IPLUS)GO TO 6
7     SGN=1.
5     VALUE(N)=0.
      GO TO 8
6     IF(KLM(NCOL).NE.IPOINT)GO TO 9
10    KPT=1
      GO TO 7
9     K=0
      ICOMP=NUM(1)
11    IF(KLM(NCOL).EQ.ICOMP) GO TO 13
12    K=K+1
      ICOMP=NUM(K+1)
      IF(K-10)11,14,14
14    NCOL=NCOL+1
24    IF(NCOL-K7)2,16,16
16    NO=N-1
      RETURN
13    SGN=1.
      VALUE(N)=K
8     NCOL=NCOL+1
      IF(NCOL-K7)17,18,18
17    IF(KLM(NCOL).NE.IPOINT)GO TO 20
19    KPT=1
      GO TO 8
20    K=0
      ICOMP=NUM(1)
21    IF(KLM(NCOL).EQ.ICOMP)GO TO 23
22    K=K+1
      ICOMP=NUM(K+1)
      IF(K-10)21,18,18
18    VALUE(N)=VALUE(N)*SGN
      N=N+1
      KPT=0
      GO TO 24
23    IF(KPT)25,26,25
26    VALUE(N)=VALUE(N)*10.+K
      GO TO 8
25    VALUE(N)=VALUE(N)+K*10.**(-KPT)
      KPT=KPT+1
      GO TO 8
      END
```

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244 p. in various pagings ; ill. ; 27 cm. -- (Miscellaneous paper ; EL-82-5)
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