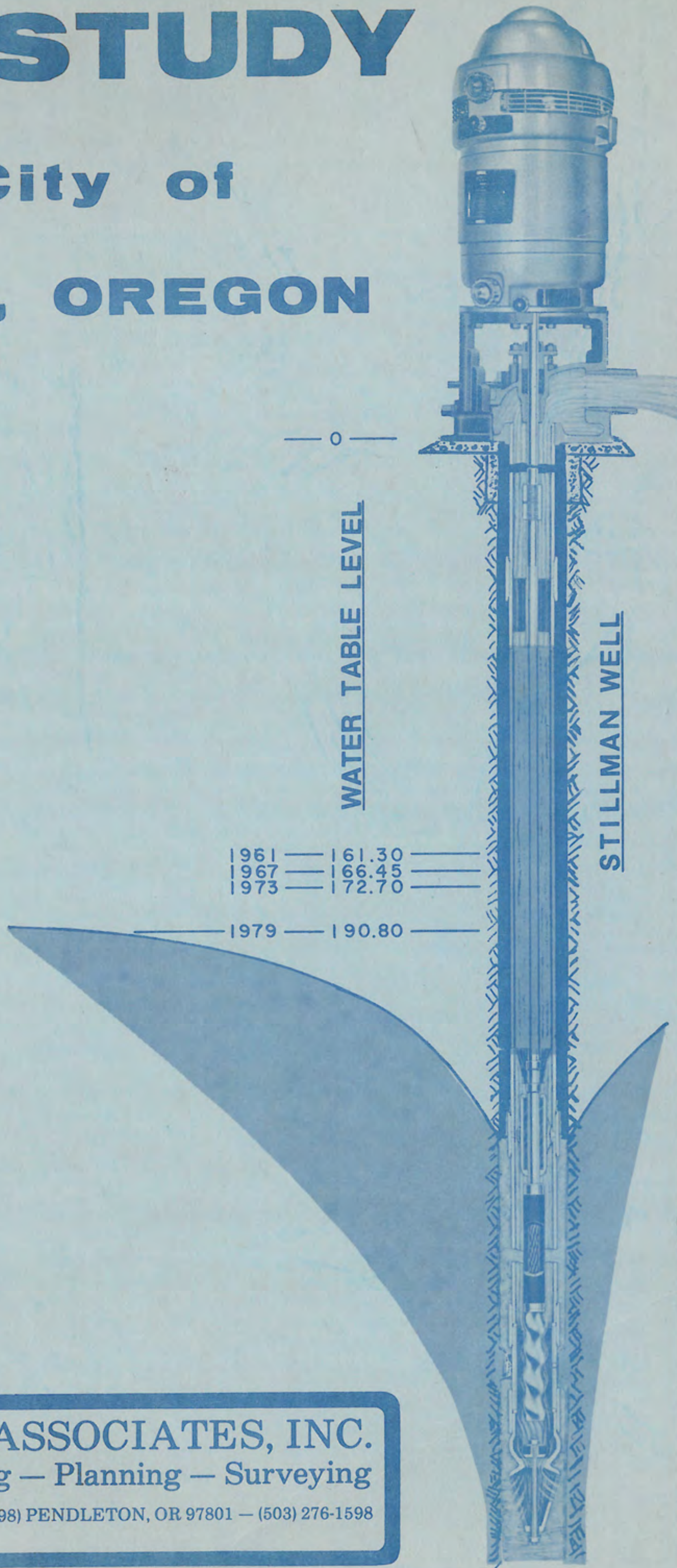
WATER STUDY

for the City of

PENDLETON, OREGON



WALLULIS & ASSOCIATES, INC.
Engineering — Planning — Surveying
(BOX 398) PENDLETON, OR 97801 — (503) 276-1598

September 1979

IMPACT OF IMPROVEMENTS ON WATER RATES

Present Water Revenue	15,000 Pop. With Category I	15,000 Pop. Increase Over Present	20,000 Pop. With Category I & II	20,000 Pop. Increase Over Present	22,500 Pop. With Category I, II, & III	22,500 Pop. Increase Over Present
27.6¢/1000 Gal.	47.72¢	72.8%	52.3¢	89.5%	74.03¢	168%
20.7¢/100 cu. ft.	35.79¢	72.8%	39.22¢	89.5%	55.53¢	168%
1968 Water Rates Adjusted For Inflation						
49.98¢/1000 Gal.	47.72¢	-4.72%	52.3¢	+4.64%	74.03¢	48%
37.48¢/100 cu. ft.	35.79¢	-4.72%	39.22¢	+4.64%	55.53¢	48%

Category I: Improvements would be adequate to service a design population of 16,500 without depletion of underlying groundwater.

Category II: Without these improvements at a population above 16,500 the underground water table should again start to experience a decline. Completion of components in this category should be adequate for a population of 20,000 persons.

Category III: These improvements are required for a population of 22,500 persons, prevent depletion of ground water and meet peak demands.

Note: Others withdrawing from the underground waters will effect above statements only if they are in the same aquifer as the City's.

WATER STUDY

For the City of

PENDLETON, OREGON

Elected Officials

Joe C. McLaughlin
Mayor

City Council

Andy Bellomo
Donovan Phillips
John Brenne
John E. Conroy

Rondi O'Gara
Doug Nichols
Robert Ramig
Donald L. Staebler

City Staff

Rudolph R. Enbysk, Manager

Gerald Odman, Public Works Director
David Lorenzen, Engineer

Marjorie Johnson, Recorder
Dana Anderson, Attorney

First Draft September 1979
Final Draft November 1979

GLOSSARY OF SELECTED TERMS

Acre Foot (Ac. ft.) – One foot of water over one acre or 325,829 gallons

Aquifer – A formation, group of formations, or part of a formation that contains sufficient saturated permeable material to yield significant quantities of water to wells or springs. Generally in horizontal scoriaceous (honeycombed) zones between successive lava flows. Can also be vertical joints which occurred during cooling and contraction of the lava.

Confined Ground Water – Ground water that is under pressure significantly greater than atmospheric. In a well that taps a confined ground water body, the static water level is above the top of the aquifer.

CFS – Cubic feet per second (one cfs = 448.6 gpm).

Drawdown – The lowering of ground water level caused by pumping. It is the difference, generally, in feet or meters, between the static water level and the pumping water level in a well.

Evapotranspiration – Water withdrawn from a land area by evaporation from water surfaces and moist soil and by plant transpiration.

GPM – Gallons per minute (gpm).

Hydraulic Gradient – The change in static head per unit of distance in a given direction (slope). The direction generally is understood to be that of the maximum rate of decrease in head.

Intermittent (or seasonal) Stream – A stream that flows only at certain times of the year when it receives water from springs or from some surface source such as melting snow in mountainous areas.

MCL – Minimum contaminant level.

MGD (mgd) – Million gallons per day (one MGD = 694.44 gpm).

Perched Ground Water – Unconfined ground water separated from an underlying body of ground water by an unsaturated zone (dense impermeable material).

Perennial Stream – A stream that flows continuously.

ppm – part(s) per million; an equivalent expression is mg/l (millograms per liter). In this report the small l has been capitalized to distinguish it from the number 1.

Potentiometric Surface - A surface that represents the static head (static level). In an aquifer it is defined by the levels at which water stands in tightly cased wells after long periods of rest or recovery.

Runoff - That part of the precipitation that appears in surface streams.

Specific Capacity - The rate of discharge of water in a well (usually in gpm) divided by the drawdown of water level (usually in feet) within the well. It is an approximate index of the capability of an aquifer to transmit water.

Static Head - The height above a datum (mean sea level) of the surface of a column of water in a well. The static water level in a well represents the average head of the water-bearing materials open to the well bore.

Turbidity - Turbidity is an empirical measurement of light refracted from suspended or dissolved matter in water. Color also affects turbidity. The murkier the water the higher the turbidity reading.

Unconfined Ground Water - Ground water in an aquifer that has a water table.

Water Table - The water surface in an unconfined water body at which the pressure is atmospheric.

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APPENDIX

PHOTOGRAPHS, MAPS, CHARTS, TABLES

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Photographs

Tables

WALLULIS & ASSOCIATES, INC.

Engineering — Planning — Surveying

* BOX 398 PENDLETON, OR 97801 — (503) 276-1598
108 E. MAIN STREET, HERMISTON, OR 97838 — (503) 567-3331

(*) Address replies to this office

September 7, 1979

Honorable Mayor and City Council
City of Pendleton
P. O. Box 190
Pendleton, Oregon 97801

Re: Pendleton Water Study
78-107

Transmitted herewith are the results of our investigation of the Pendleton Water System. The information contained in this report was complete and ready for publication in April of this year. About that time, Jerry Odman, Director of Public Works, attended a Public Works meeting and received the impression from Don Gipe, Oregon Office of EPA, that the Thornhollow gravity water supply might not require treatment. The treatment of the gravity water supply has the most significant impact on future water rates. It was mutually decided to defer publication of this report until the treatment requirements of the source were established by EPA. On May 11, 1979, Mr. Odman and I attended a conference in Portland with the EPA staff concerning their position on this matter. We were advised at the conclusion of this conference that we would receive within one week a written reply from EPA formally stating their position. Though we continuously "prodded" EPA for several months, we did not receive the promised reply. Finally, our efforts extended to requesting Mayor Joe McLaughlin on August 7, 1979, to assist us in obtaining a response from EPA. This effort proved to be successful and EPA's formal response was received on August 16, 1979. Unfortunately, efforts to have EPA "soften" their position on requiring treatment of the gravity supply proved unfruitful.

During this interim period the City has already completed some of the improvements recommended in this report. These are the partial extension of the 16" waterline on N. W. 5th Street, easterly extension of the 12" waterline on S. E. Court Avenue, and correction of the bottleneck at S. W. 6th Street and S. W. Emigrant Avenue. Rather than amend the text and correct the previously prepared maps, we are acknowledging the construction of these improvements in this letter of transmittal. Some areas of the report were updated.

This study entailed researching several previous studies, reports, professional papers, and records. We have attempted to keep this study as concise as practical and yet provide sufficient data to document our findings. To facilitate your review of this study, we are providing a summary sheet for your notations on areas where you would like additional explanation or documentation. If you arrive at a rather extensive list, it may be beneficial to arrange for an individual conference.

As stated in the first paragraph of this letter, the text of this report was completed in April of this year. Estimated costs in this report were based on an Engineering News Record Cost Index numerical value of 2861 (December 1978), and were projected at an 8% inflationary rate to 1981. In view of current inflation rates these estimates should be revised prior to the scheduling of a bond election.

We accepted the challenge of preparing this report, fully realizing that a properly documented study could not be justified on the basis of the financial renumeration. In the preparation of this study we have attempted to repay the community at large for the many direct and indirect benefits our firm has received. We appreciate the confidence the City has shown in our firm by selecting us to prepare this study and hope that our efforts in this regard merit the continued support of the City of Pendleton.

Respectfully submitted,

Stanley G. Wallulis

Stanley G. Wallulis, P.E.
President

SGW:jm

CHAPTER I

INTRODUCTION

A. GENERAL

1. STUDY AUTHORIZATION AND SCOPE

The City of Pendleton authorized the firm of Wallulis and Associates, Incorporated to prepare an updated Comprehensive Water System Study on May 17, 1978. The purpose of the updated Plan was to analyze the existing water system's principal elements of sources, distribution, transmission and storage for present deficiencies and future growth. Alternative solutions were to be studied and cost estimates prepared for the most effective solution for each principal element. The investigation was to also include recommended revenue sources and possible grant sources.

2. COMMUNITY DESCRIPTION

The City of Pendleton is the County Seat for Umatilla County, Oregon, and is situated along the banks of the Umatilla River at the junction of U.S. Highway 395 and Interstate 80 North. The economy of the County and the City is based primarily on agriculture, agri-industry, livestock, timber and wood related industries. Pendleton, as the largest city in Eastern Oregon, serves as a regional retail trade center, maintains a Class V "continental" airport served by United Airlines, home of Blue Mountain Community College, the hub for transportation and distribution of goods, several State and federal regional offices, Harris Pine Mills, Pendleton Woolen Mills, Iglehart Operations Division of General Foods Corporation, Prowler Industries, Golden West Mobile Homes, Eastern Oregon Hospital and Training Center, and Hill Meat Company.

B. WATER SYSTEM GENERAL DESCRIPTION

1. SUPPLY SYSTEM

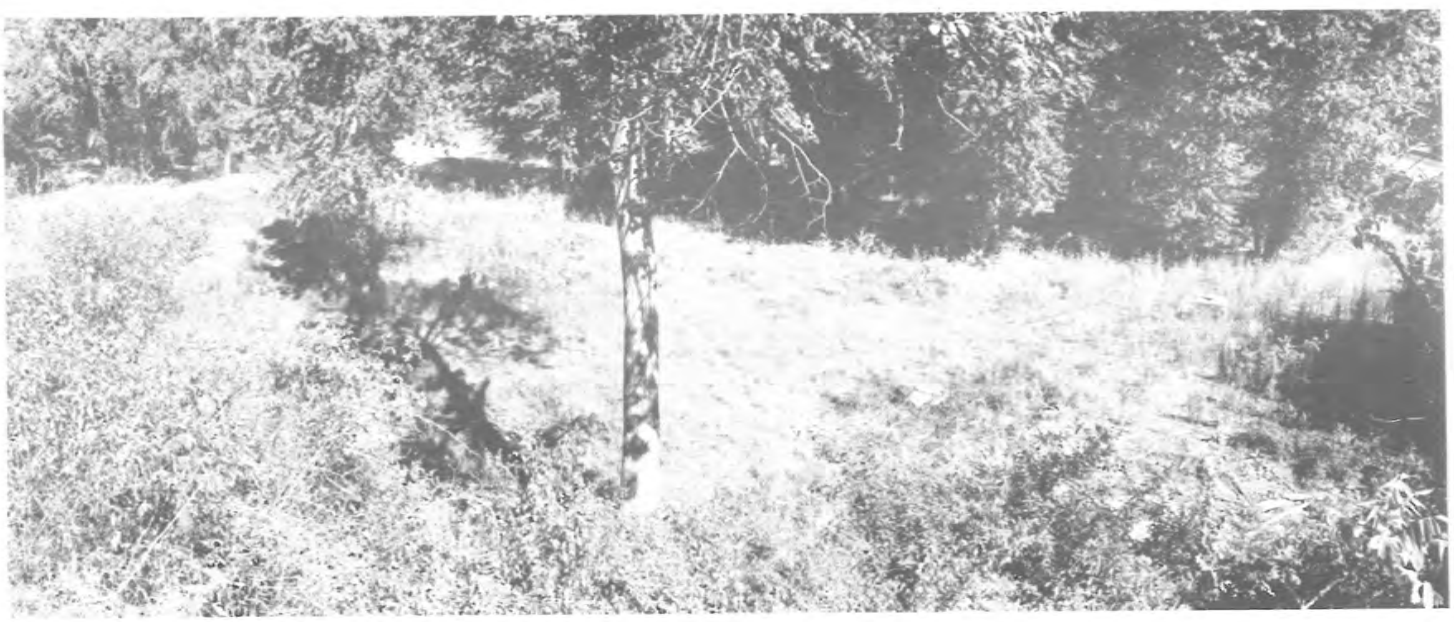
In the early 1900's the City's population was slightly below 5,000 persons and the area served by the City water system was concentrated in the valley floor. After 1910 the City began to feel the impact of additional growth and provided the impetus to seek a better quality and larger quantity of water for the community. Construction of the present gravity supply system was initiated in 1913 and in a few years 16.37 miles were constructed; subsequently the line has been extended to the uppermost springs for a total of some 22 miles in length. In 1917 the North and South Hill reservoirs were constructed and replaced a single open reservoir that was located approximately two blocks below the present South Hill Reservoir.

WEIR HOUSE
BELOW
WENIX SPRINGS

WEIR

TURBIDIMETER IN
WEIR HOUSE

NORTH & SOUTH WENIX SPRINGS



NORTH & MIDDLE SIMON SPRINGS

SOUTH SIMON SPRINGS

NORTH, MIDDLE & SOUTH CHAPLISH SPRINGS

LONGHAIR SPRINGS

VALVE HOUSE INTERIOR - WEST RESERVOIR SOUTH SIDE EXTERIOR

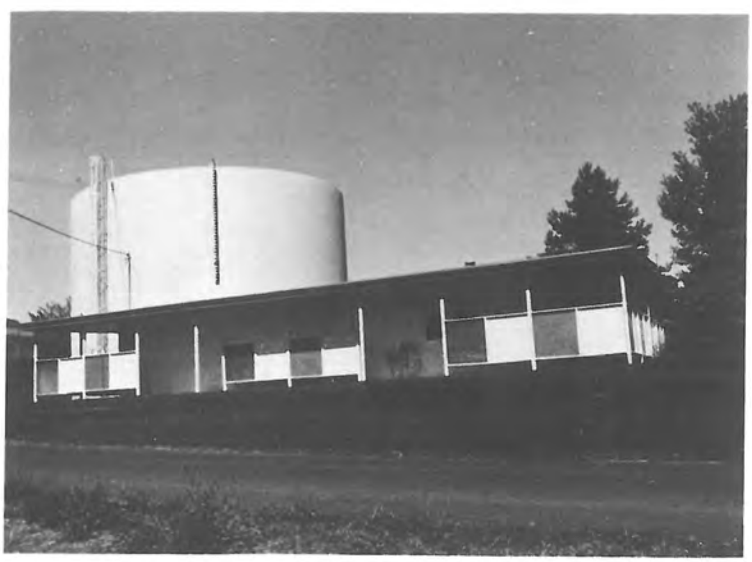
SOUTH HILL RESERVOIR

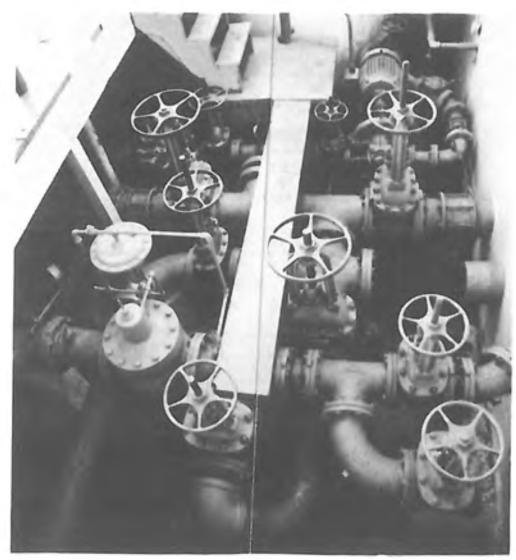
NORTH HILL RESERVOIR

AIRPORT RESERVOIR

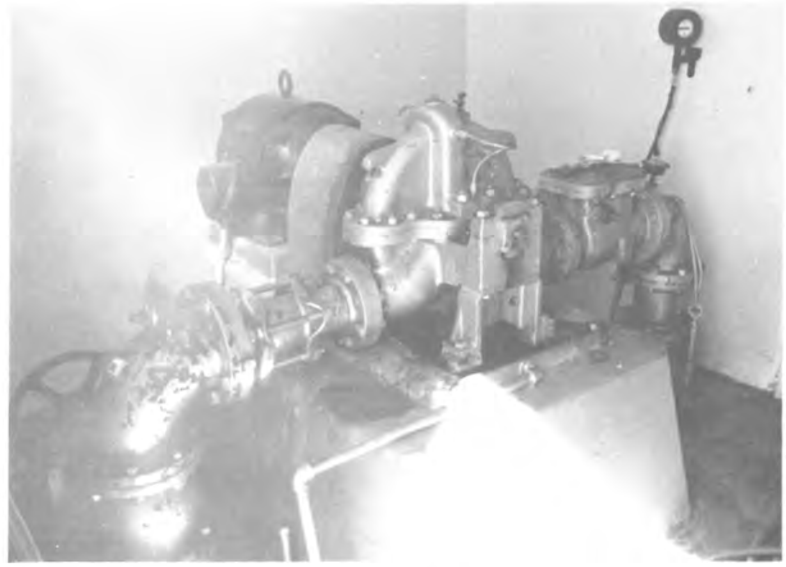
NORTH HILL HIGH LEVEL RESERVOIR

LOW LEVEL AIRPORT BOOSTER STATION



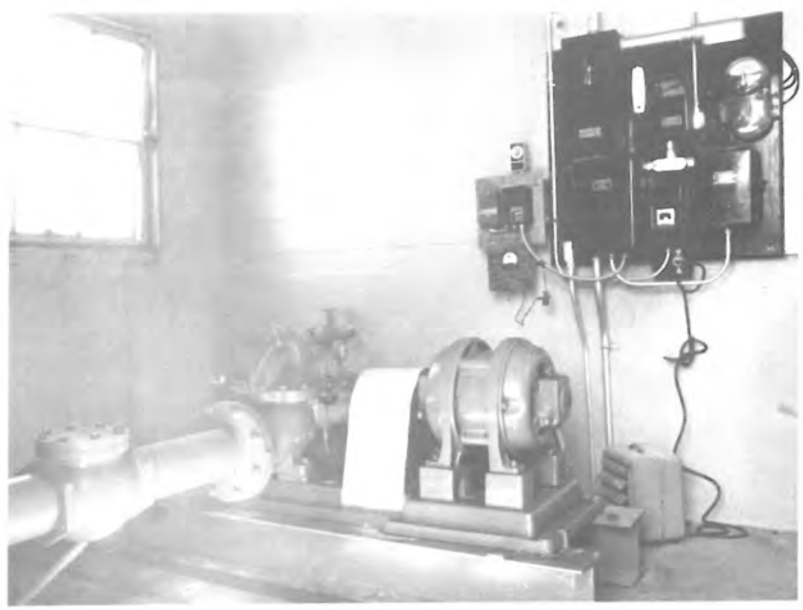


UPPER LEVEL AIRPORT BOOSTER STATION



BOOSTER STATION TO NORTH HILL HIGH LEVEL RESERVOIR
N. W. 12TH STREET & N. W. GILLIAM AVENUE

BOOSTER STATION TO
NORTH HILL HIGH LEVEL
RESERVOIR
LOCATED IN MANHOLE AT
SOUTHWEST CORNER OF
NORTH HILL RESERVOIR



BOOSTER STATION TO NORTH HILL LOW LEVEL RESERVOIR
N. W. 5TH STREET & N. W. HORN AVENUE

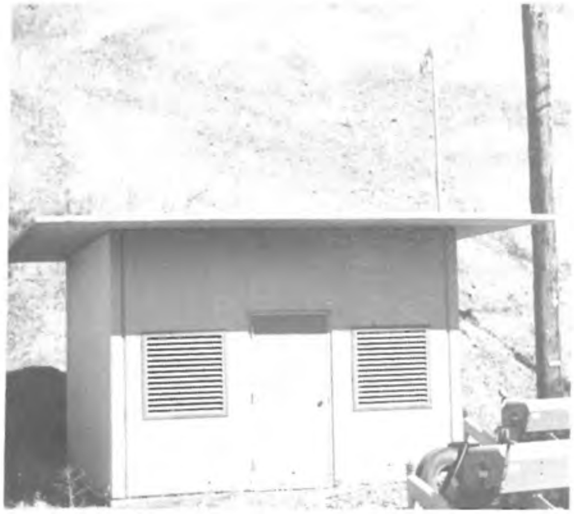
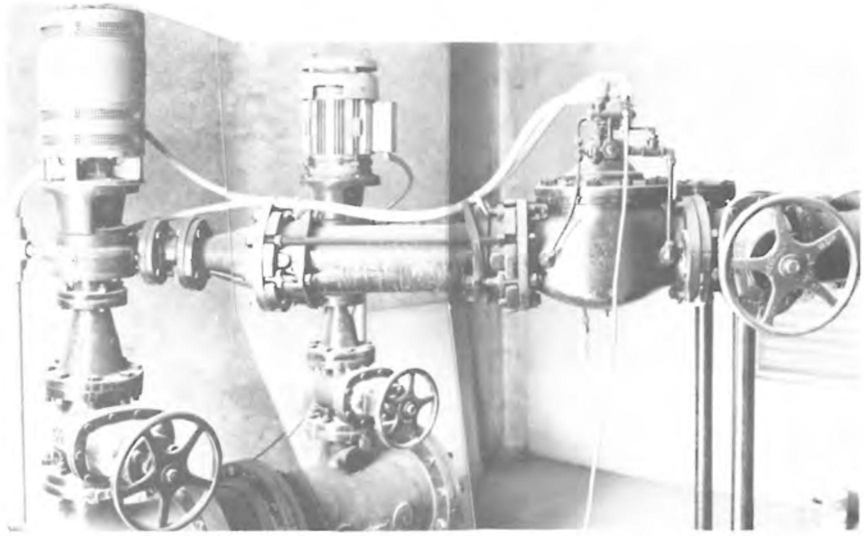
SOUTHWEST BOOSTER STATION FOR SOUTHWEST HIGH LEVEL SERVICE
S. W. 25TH STREET & S. W. LADOW

SOUTH BOOSTER STATION FOR SOUTH HIGH LEVEL SERVICE

S. E. 7TH STREET - SOUTH OF S. E. ISAAC AVENUE

SOUTHEAST BOOSTER STATION FOR SOUTHEAST HIGH LEVEL SERVICE

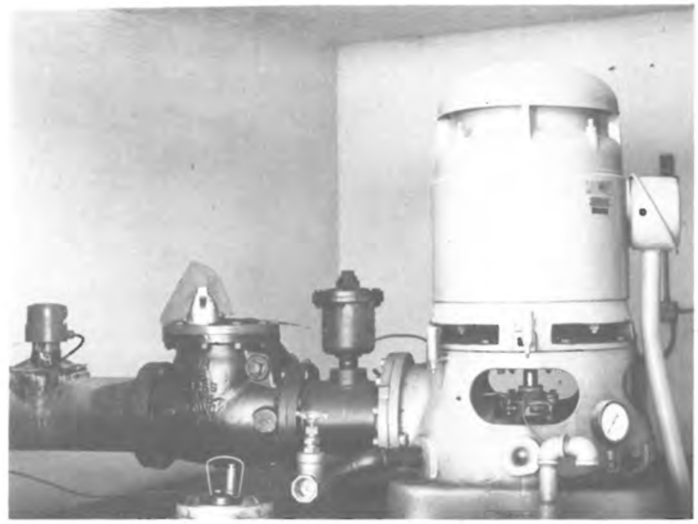
S. E. 20TH STREET SOUTH OF S. E. COURT

MT. HEBRON BOOSTER STATION FOR MT. HEBRON HIGH LEVEL SERVICE
U. S. HIGHWAY 11


WELL NO. 1
S. E. BYERS STREET WELL


WELL NO. 2
ROY RALEY PARK - ROUND-UP PARK WELL

WELL NO. 3
S. W. 21ST STREET WELL


WELL NO. 4
STATE HOSPITAL WELL

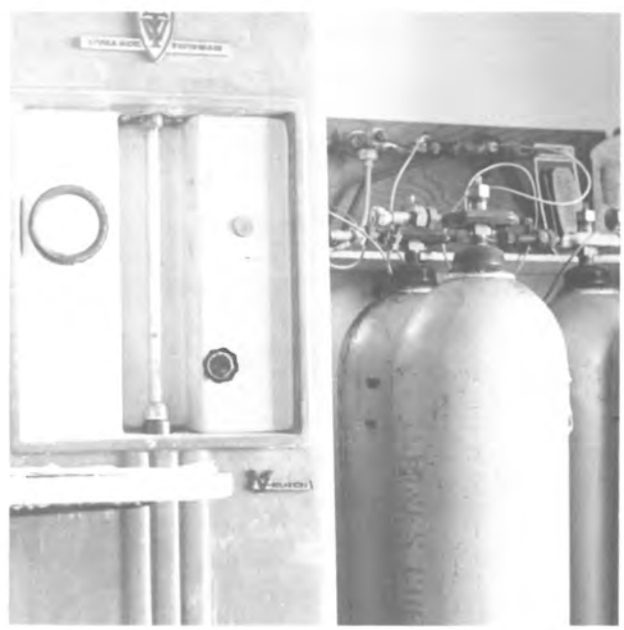

WELL NO. 5
STILLMAN WELL

WELL NO. 7 MISSION WELL

FLOURIDE TREATMENT HOLDING TANK (LEFT)
WELL NO. 7 BUILDING (CENTER)
CHEMICAL FEED BUILDING (RIGHT)

PUMP HOUSE

FLOURIDATION



CHLORINATION

CHEMICAL FEED BUILDING
WELL NO. 7 - MISSION WELL

The springs provided all of the City's water needs until 1948 when deep Wells No. 1 (S. E. Byers Avenue) and No. 2 (Round-up Park) were drilled to augment the gravity supply. Since 1948 an additional four deep well sources were added to the present supply system in 1952 (S. W. 21st Street); 1955 (State Hospital); 1960 (Stillman) and 1968 (Mission Well).

2. WATER SERVICE TO THE AIRPORT

In the late 1930's (approximately 1938-1939), a new transmission line was laid from the South Hill Reservoir to provide service to the airport. This line was completed in the early 1940's along with a booster pump station and two wooden reservoirs. The wood reservoirs have been subsequently replaced with a steel reservoir.

3. UPPER NORTH HILL SERVICE AREA

In the late 1940's continued growth of the community resulted in residential developments at higher elevations on the North Hill, requiring the construction of a reservoir to service the upper level and the installation of a booster pump station at N. W. 12th Street and N. W. Gilliam Avenue and later a booster pump at the North Hill reservoir.

4. MT. HEBRON - RIVERSIDE

In 1951 the water system was expanded to include the Riverside and Mt. Hebron areas. The Mt. Hebron development required the installation of a small booster station, which was installed by the developer and maintained by the residents for several years. In 1973 this area was annexed and in 1974 the old booster station was replaced with a new pumping station at the base of Mt. Hebron just west of U.S. Highway 11 to Walla Walla, Washington.

5. UPPER SOUTH HILL SERVICE AREAS

In the early 1950's (1953-1954) growth on the South Hill required the installation of a booster pump station at S. W. 8th Street and Isaac Avenue. In 1959 an additional booster pumping station was installed at S. E. 6th Street and S. E. Isaac Avenue. In 1962 these two areas were connected with a 6" line. Because of increased growth around the south freeway interchange, this station was subsequently replaced with the larger booster pumping station in 1968 at S. E. 7th Street and S. E. Isaac Avenue.

6. S. E. UPPER LEVEL SERVICE AREA

In the mid-1950's, a small booster pumping station was installed in the vicinity of S. E. 20th Street and S. E. Court Avenue. This booster station services a few homes and tourist commercial facilities situated at the junction of Highway 11 and the Old Oregon Trail Highway to Mission.

7. S. W. UPPER LEVEL SERVICE AREA

In the early 1960's additional growth in the Southwest area of Young's Second Addition and the construction of the Community Hospital required the installation of an additional booster pumping station at S. W. 25th Street and S. W. Ladow Avenue in 1961.

8. PENDLETON WATER SYSTEM SUMMARY

The present system consists of the base system servicing the valley floor, the North high level system, the airport high level system, the Southwest high level system, the South high level system, the Southeast high level system, and the Mt. Hebron high level system.

9. UMATILLA INDIAN AGENCY SERVICE AREA

In addition to the City system proper defined above, the gravity line also services the Indian Agency and a few other individuals. Well No. 7, the Mission Well, was strategically selected to provide the water customers with good quality water when the spring sources become too turbid (murky) for domestic use.

C. ACKNOWLEDGMENTS

Historical, technical data and records, and general assistance provided by the City Public Works Director, Jerry Odman; City Planner, Edd Rhodes; Water Foreman, Gene Harover; Mrs. Marge Johnson, City Recorder, were invaluable in the preparation of this report. Ongoing cooperation and conferences on the various alternatives considered resulted in the early elimination of several alternatives and permitted this Study to focus on the viable alternatives.

CHAPTER II

WATER SYSTEM REQUIREMENTS

A. POPULATION

Population records from 1890 to 1960 documented a continuous growth each decade, except for a stable period from 1900 to 1910 and a slight decrease from 1920 to 1930. The average increase in population over the seventy year time period was 170 persons per year. The fastest growth rate period was between 1930 and 1960 when the average yearly uniform increase in population was approximately 260 persons per year.

From 1960 to the present the population has fluctuated erratically. In 1960 the population was 14,434 persons and in 1970 fell to 13,197 persons. Since 1970 the population has rebounded at an average rate of 225 persons per year to a population of 15,000 (State census) as of July, 1978.

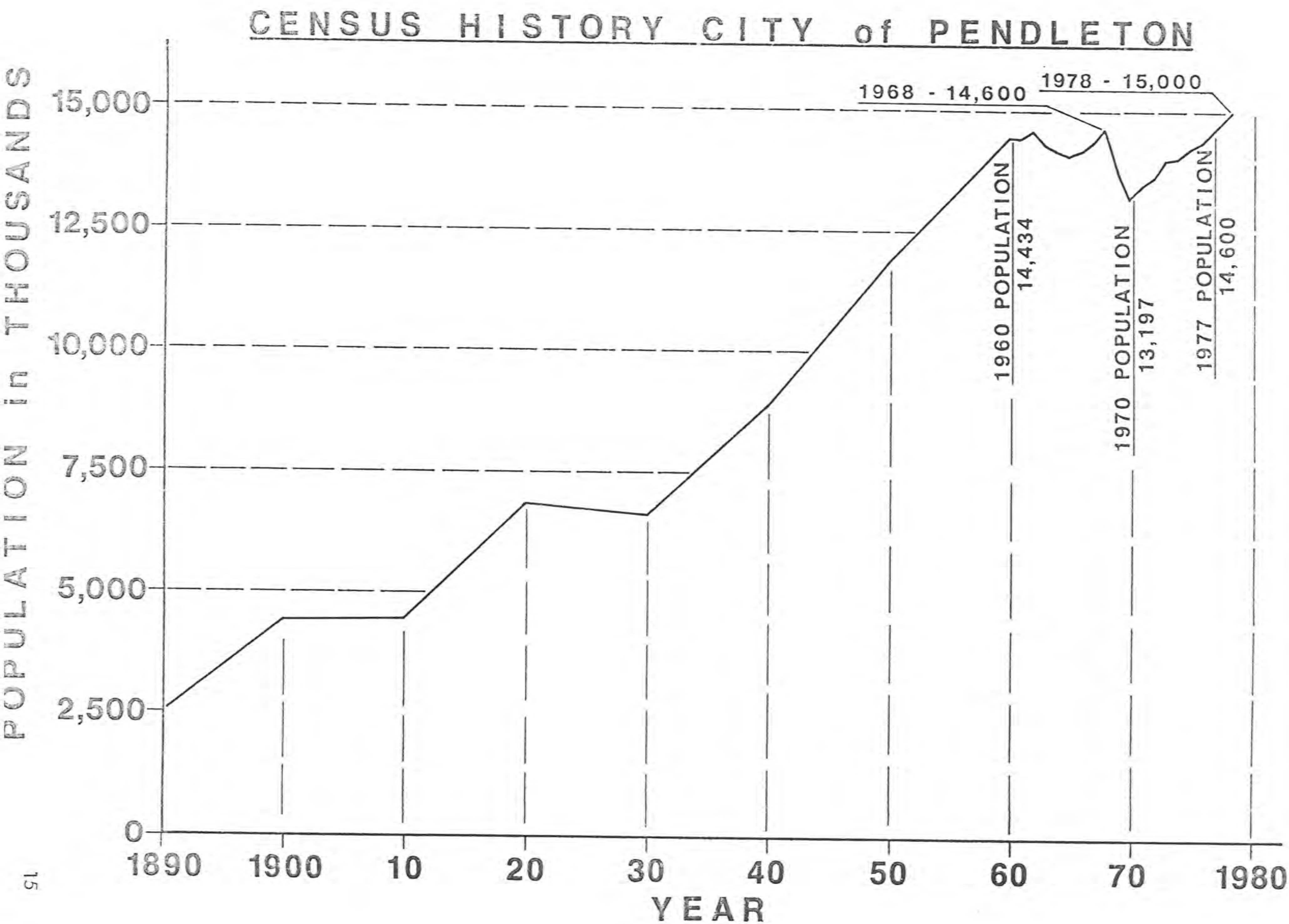
The reduction in the number of persons occupying the typical residential unit from 3.9 persons to 2.7 persons per unit is largely responsible for the construction of several new single-family residential homes and multi-family structures in the community.

Likewise, the changing trends in merchandising have dictated that commercial enterprises have adequate companion parking facilities. This, along with the construction of Interstate 80 North and the interchanges, have effectively segmented and dispersed the commercial and residential land uses in the community. Access corridors from the freeway to the downtown business district along Emigrant Avenue and Frazer Avenue has resulted in the conversion of former residential units to businesses.

The additional population gain, 566 persons since the 1960 census, has not by itself created a significant demand on the water supply. The dispersal of the population over a larger land area has resulted in additional water demand for irrigation of yards during the summer months when the gravity supply is at its lowest yields.

B. FUTURE GROWTH AREAS

Rather than attempt to project future population growth it was the consensus of the City staff that this Study should analyze the water system requirements for populations of 20,000 persons and 25,000 persons. The areas in which future growth are expected to occur are as follows:



CENSUS HISTORY CITY of PENDLETON
POPULATION in THOUSANDS
15,000
12,500
10,000
7,500
5,000
2,500
0
1968 - 14,600
1978 - 15,000
1960 POPULATION 14,434
1970 POPULATION 13,197
1977 POPULATION 14,600
1890
1900
10
20
30
40
50
60
70
1980
YEAR

AREA	First Growth Increment of 5,000 persons	Second Growth Increment of 10,000 persons
North	750	1,200
East	250	300
South	1,000	2,000
Southwest	3,000	6,500
TOTALS	5,000	10,000

The above divisions were based on recent development trends, topography, the Indian Reservation boundary to the East of the City and preservation of the best farm lands north of the City. These findings are consistent with a companion traffic study presently being finalized for the Southwest area. A map depicting the growth areas is shown on page 17 .

In 1957, a population survey of the community was made prior to the construction of the fire station at S. W. 10th Street and S. W. Court Avenue. It was determined at that time that the geographic population center was slightly west of where the present fire station was constructed in 1959.

In the intervening years the majority of the population growth has occurred in the Southwest area. The current geographical center of the City is estimated to be in the vicinity of S. W. 20th Street and S. W. Emigrant Avenue.

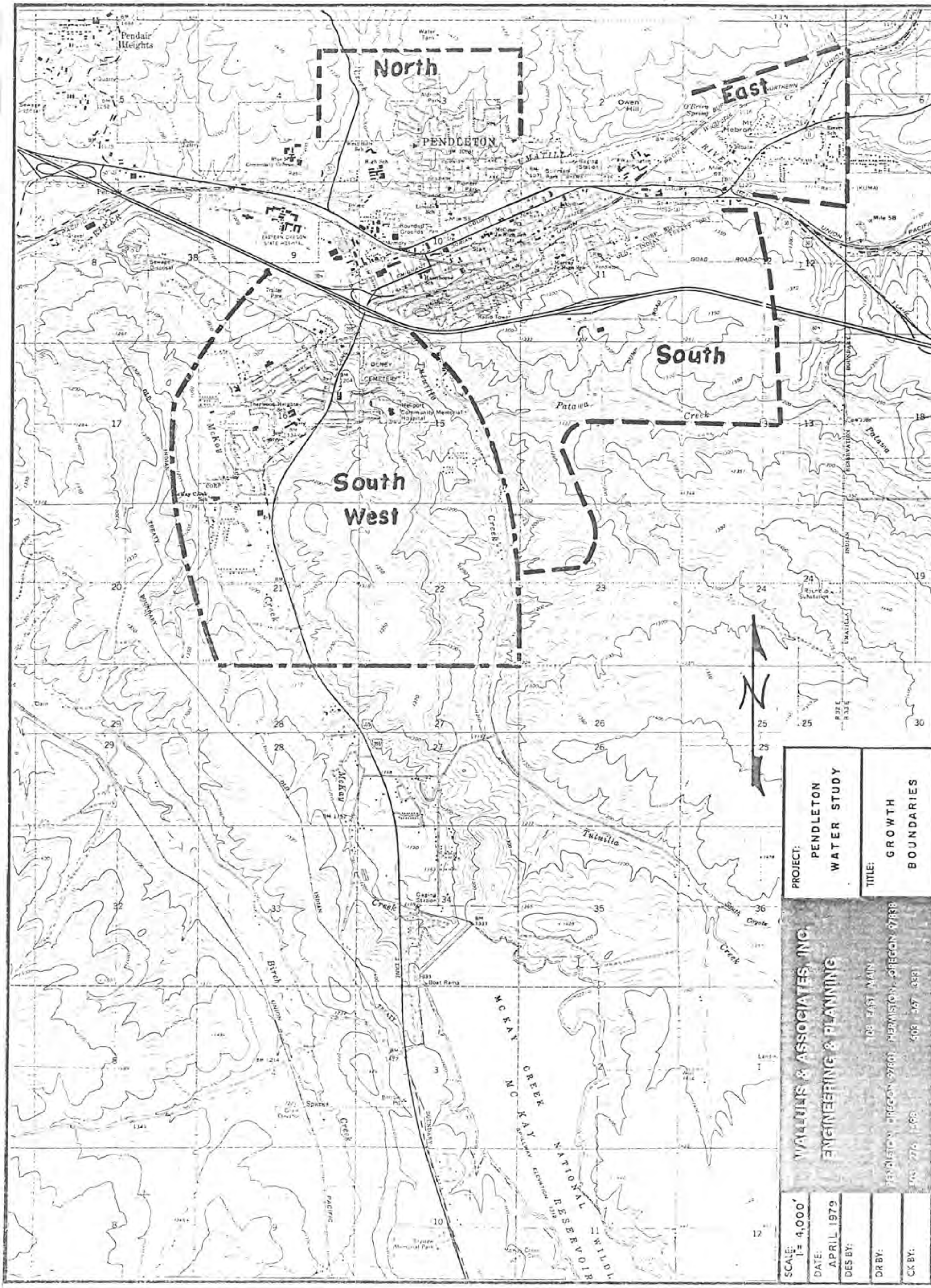
C. LARGE DEMAND USERS

Present large system demand users are the Eastern Oregon Hospital and Training Center, Harris Pine Mills, Pendleton Woolen Mills, Hill Meat Company, City Cemetery, City Park, schools, large food stores, hospitals, and large motel complexes. Residential units during the summer months collectively place the largest demand on the water system.

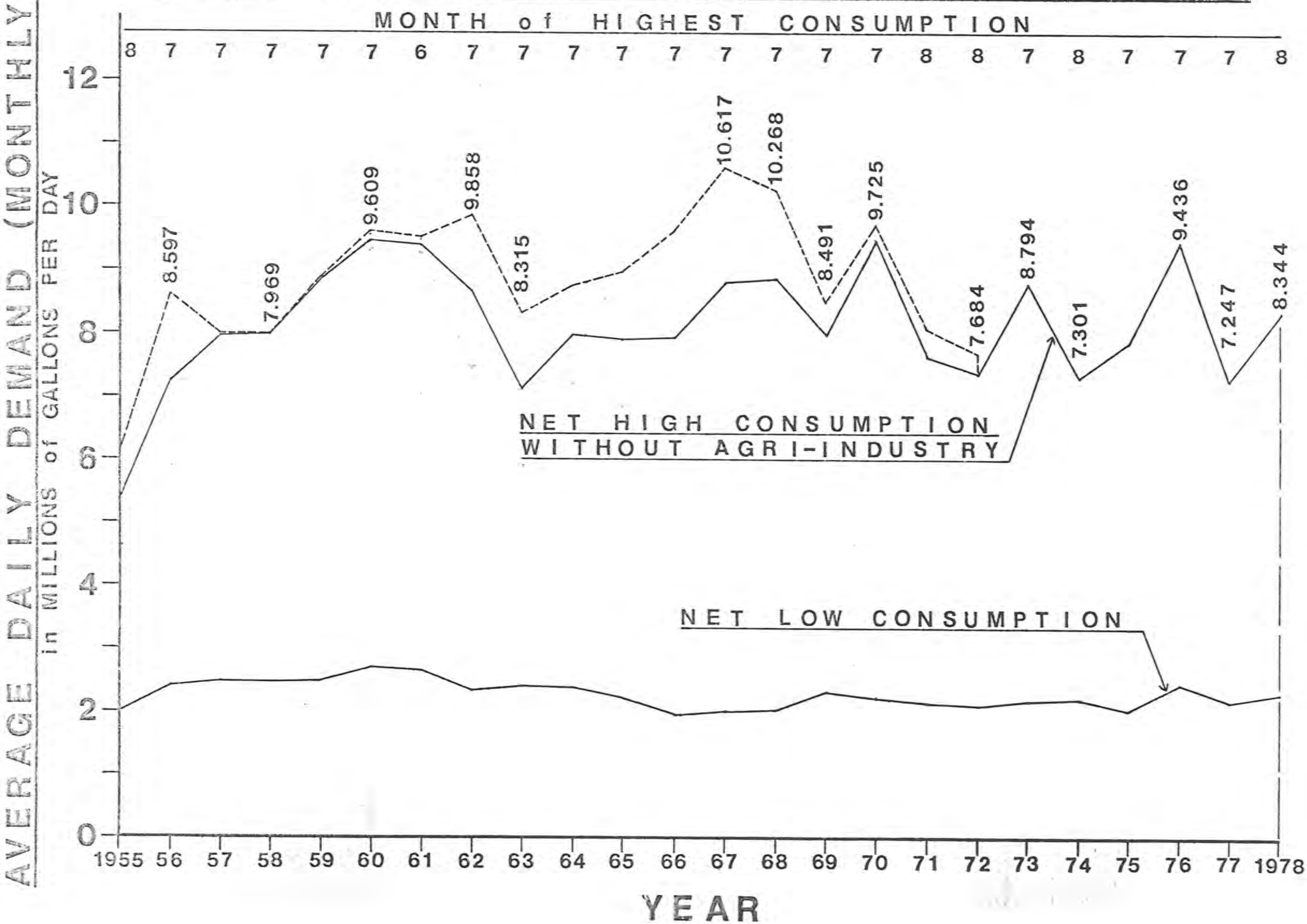
D. WATER CONSUMPTION HISTORY

The months of maximum and minimum water consumption were obtained from the City's records for the last 23 years and these results were plotted on a chart on page 18 .

From 1955 to 1968 Pendleton Frozen Foods and Smith Canning Company placed considerable demands on the water system during the summer months. In 1968 Pendleton Frozen Foods, the largest of the two water users, ceased their operations in Pendleton. From 1968 to 1972, Smith Canning Company gradually phased out their operations in Pendleton.


North
East
South
South West
PENDLETON
UMATILLA
Pendair Heights
Owen Hill
Mt Hebron
EASTERN OREGON STATE HOSPITAL
OLNEY CEMETERY
Patawa Creek
McKay Creek
Tutuilla
South Coyote Creek
Birch Creek
Boat Ramp
MC KAY CREEK NATIONAL WILDLIFE RESERVOIR
PROJECT: PENDLETON WATER STUDY
TITLE: GROWTH BOUNDARIES
WALLULIS & ASSOCIATES, INC.
ENGINEERING & PLANNING
108 EAST MAIN
PENDLETON, OREGON 97801 HERMISTON, OREGON 97838
SCALE: 1"= 4,000'
DATE: APRIL 1979
DES BY:
DR BY:
CK BY:

HISTORIC HIGH & LOW CONSUMPTIVE USE



Water consumption during summer months underwent a gradual reduction beginning in the mid-1950's when compact window-sized refrigerated air conditioners started to gain popularity. The gradual transition from water evaporative coolers to refrigerated air conditioning for residential units, offices, and commercial units has substantially reduced the per capita consumption of water that formerly went to waste.

Annual peak summer water demands have varied considerably from year to year. The irrigation demands respond to the climate conditions prevailing in each particular year and as such cause considerable variations in the annual summer demands. During growth periods when several new homes are built and yards are being established materially increases the normal summer irrigation demands.

The low consumptive usage platted on the graph does not necessarily reflect the amount of water that was actually placed to beneficial use. It has been the practice in years past to permit more water to flow from the Thornhollow Springs than the community actually needed. The surplus water received at the South Hill Reservoir was discharged to waste through an overflow pipe. As water meters are not read in the winter months, this historical data is of little benefit.

E. FIRE DEMANDS AND TOPOGRAPHY

In 1972 and 1974 the Insurance Services Office published a new "Guide for Determination of Required Fire Flow" and a new "Grading Schedule for Municipal Fire Protection".

1. RECOMMENDED FIRE FLOWS

For residential areas a short method is used to determine the recommended fire flows for single family and duplex units not exceeding two stories in height. Exposure distances given below is the open space between the nearest neighboring structure.

TABLE 1

RESIDENTIAL RECOMMENDED FIRE FLOWS
by the short method

Exposure Distance	Recommended Fire Flow in GPM
over 100 feet	500
31 feet to 100 feet	750 - 1,000
11 feet to 30 feet	1,000 - 1,500
10 feet or less	1,500 - 2,000*

*If buildings are continuous use 2,500 gpm and to the above where wood shingles could contribute to the spreading of fires, add 500 gpm.

The use of the structures that have either light or extra hazard occupancies may result in modification of the fire flow requirements in the Tables 2, 3, 4, and 5 that follow by as much as plus or minus 25%.

Factors that materially reduce the recommended fire flows are sprinkler protection (up to a 25% reduction), fire resistive or noncombustible construction and light hazard occupancy (up to a 50% reduction). Factors that materially increase the recommended fire flows are the exposure distance to adjacent structures and degree of risk (up to 75% increase); openings within the exposed building (shafts); occupancy of the building (up to 25% increase); and hillside locations where fire spreads more rapidly.

For the basis of comparison of the recommended fire flows for the various types of construction, we have prepared Table 2 for Wood Frame Construction, Table 3 for Ordinary Construction, Table 4 for Non-Combustible Construction and Table 5 for Fire Resistive Construction.

Wood frame is defined as a type of structural construction where wood is utilized for outside and inside bearing walls, roof and floor systems.

Ordinary construction is defined as a type of structural construction where masonry such as brick, concrete block or tilt-up concrete forms the exterior walls with wood joisted roofs such as built-up roofs and interior supports of consisting heavy timbers or wood joists interior. Non-load bearing internal walls can be constructed of wood with sheet rock covering.

Non-combustible construction is defined as a steel building constructed with unprotected metal for posts, beams, roof trusses, and an outer metal skin for the walls and roof. Non-load bearing internal walls can be constructed of wood with sheet rock covering.

Fire resistive construction is defined as a type of structural construction consisting of masonry walls and metal supports which have sprayed on fire resistive insulation. Roof structures would consist of fire protected metal trusses or concrete slabs or tee beams. Only limited use of non-bearing walls constructed of wood and sheet rock would be permitted.

TABLE 2

WOOD FRAME CONSTRUCTION

Ground Area Square Feet	Fire Flow Single Story in GPM	Fire Flow Two Story in GPM	Fire Flow Three Story inGPM
1,000	750	1,250	1,500
1,500	1,000	1,500	1,750
2,000	1,250	1,750	2,000
3,000	1,500	2,000	2,500
4,000	1,750	2,500	3,000
6,000	2,000	3,000	3,500
8,000	2,500	3,500	4,250
10,000	2,750	3,750	4,750
15,000	3,250	4,750	5,750
20,000	3,750	5,500	6,500
25,000	4,250	6,000	7,500
30,000	4,750	6,500	8,000
40,000	5,500	7,750	off chart
50,000	6,000	off chart	off chart
60,000	6,750	off chart	off chart
70,000	7,250	off chart	off chart
80,000	7,750	off chart	off chart

TABLE 3

ORDINARY CONSTRUCTION

Ground Area Square Feet	Fire Flow Single Story in GPM	Fire Flow Two Story in GPM	Fire Flow Three Story in GPM
1,000	500	750	1,000
1,500	500	1,000	1,250
2,000	750	1,250	1,500
3,000	1,000	1,500	1,750
4,000	1,250	1,500	2,000
6,000	1,500	2,000	2,500
8,000	1,500	2,250	2,750
10,000	1,750	2,500	3,250
15,000	2,250	3,000	3,750
20,000	2,500	3,500	4,500
25,000	2,750	4,000	5,000
30,000	3,000	4,500	5,500
40,000	3,500	5,000	6,250
50,000	4,000	5,750	7,000
60,000	4,500	6,250	7,750
70,000	4,750	6,750	off chart
80,000	5,000	7,250	off chart

TABLE 4

NON-COMBUSTIBLE CONSTRUCTION

Ground Area Square Feet	Fire Flow Single Story in GPM	Fire Flow Two Story in GPM	Fire Flow Three Story in GPM
1,000	500	750	750
1,500	500	750	1,000
2,000	750	1,000	1,250
3,000	750	1,000	1,250
4,000	1,000	1,250	1,500
6,000	1,000	1,500	2,000
8,000	1,250	1,750	2,250
10,000	1,500	2,000	2,500
15,000	1,750	2,500	3,000
20,000	2,000	3,000	3,500
25,000	2,250	3,250	4,000
30,000	2,500	3,500	4,250
40,000	3,000	4,000	5,000
50,000	3,250	4,500	5,500
60,000	3,500	5,000	off chart
70,000	3,750	5,500	off chart
80,000	4,000	5,750	off chart

TABLE 5

FIRE RESISTIVE CONSTRUCTION

Ground Area Square Feet	Fire Flow Single Story in GPM	Fire Flow Two Story in GPM	Fire Flow Three Story in GPM
1,000	500	500	500
1,500	500	500	750
2,000	500	750	750
3,000	500	750	1,000
4,000	750	1,000	1,250
6,000	750	1,250	1,500
8,000	1,000	1,250	1,750
10,000	1,000	1,500	1,750
15,000	1,250	1,750	2,250
20,000	1,500	2,250	2,750
25,000	1,750	2,500	3,000
30,000	1,750	2,750	3,250
40,000	2,250	3,000	3,750
50,000	2,500	3,500	4,250
60,000	2,750	3,750	4,500
70,000	2,750	4,000	5,000
80,000	3,000	4,250	5,250

2. FIRE FLOW DEFICIENT AREAS

Unfortunately, several large structures have been constructed on high ground where the present water system cannot supply the recommended fire flows. Existing structures in this category that presently cannot be served with the recommended fire flows coupled with other system demands are:

1. The Pendleton Community Hospital
2. Bi-Mart
3. Indian Hills Motel
4. S. E. 20th and Goodwin
5. Residences at N. E. corner of the City limits
6. Residences on North Skyline Drive
7. Blue Mountain Community College
8. Farmore Distributing Company
9. Hill's Furniture
10. Fire Station at Airport
11. Hill Meat Company
12. Apartment complexes along S. W. 28th Drive
13. Sherwood Heights School
14. K-Mart

New proposed structures that will fall into this category are the Kopper Kitchen and Motel Six to be built at the S. E. 3rd Interchange.

Based on a review of hydrant flow tests, some of which are up to eight-years old, there are other areas where deficient fire flows exist because of distribution system limitations. Some of the more typical locations are as follows:

1. Easterly portion of Riverside area
2. S. E. 25th and Court Avenue (Harvester)
3. St. Anthony Hospital
4. N. E. substation - N. E. Owens Hill
5. S. E. 8th and Byers Avenue
6. Industrial Park (Prowler - Golden West)
7. McKay Creek School
8. Oregon Trail Manor
9. Winter Motors
10. Pendleton High School

3. FIRE HYDRANT MAINTENANCE

In discussion on hydrant flow tests with Fire Chief Virgil Boyd, he stated that the manning requirements of the ambulance program required a shifting of manpower away from the hydrant testing program. This is unfortunate as the water distribution system has been gradually improved.

The older tests may not be truly reflective. Also the maintaining of current test records on fire hydrants documents their reliability and these records materially affect the fire rating given the community. The Insurance Services Office recommends that "Hydrants shall be inspected semi-annually and after use; inspection shall include operation at least once a year. Where freezing conditions occur, the semi-annual inspections shall be made in the spring and fall of each year; hydrants requiring special attention because of freezing shall be checked frequently during extended periods of severe cold."

CHAPTER III

SURFACE WATER SUPPLY

A. PRESENT AND FORMER SOURCES

In February, 1978, the United States Environmental Protection Agency (EPA) classified the springs which have been the backbone of the City water system as surface water supplies (see letter dated February 15, 1978 in Appendix A). The justification for classifying the springs as a surface source was based on the findings of a source evaluation conducted on January 31, 1978 by Mr. Bill Titus of the Oregon Operations Office of the EPA. The basis for arriving at the conclusion that the springs should be classified as a surface supply was as follows:

> "It is my opinion that the water originates from local rainfall and snowpack and is carried into the gravels of the valley floor by the well-developed local pattern of streams and creeks and by the Umatilla River itself. In support of this opinion, I list the following circumstantial evidence:
>
> 1) the flow from the source works is seasonally variable, the peaks correspond perfectly to local precipitation and snowmelt patterns.
>
> 2) the water is relatively soft, a general characteristic of surface water which has had little or no contact with underground rock formations.
>
> 3) the turbidity is seasonally variable, with peaks corresponding to peak runoff periods.
>
> Regardless of the ultimate source of the water, however, the flow from the ultimate source is being stored in the naturally-occurring fluvial gravel deposits which comprise the valley floor. These gravel deposits are not protected by an impervious layer (or even a restrictive layer) and are subject to contamination by surface runoff. Since protection from surface runoff is the essential criterion which distinguishes between ground water and surface water, it is my opinion that the source works in the Thornhollow area are surface water sources and it is my recommendation that they be classified as such.
>
> Insofar as the City's request for an adjustment to the turbidity MCL is concerned, I do not believe that the watershed which supplies the source works (the fluvial gravel deposits up-gradient from the source works and the streams which contribute flow to those deposits)

is protected to any reasonable degree from contamination. Since a protected watershed is an essential prerequisite for the MCL adjustment (see EPA DWPB Procedural Criteria 77-3), I do not feel that the City of Pendleton is eligible for the adjustment."

A complete copy of the report prepared by Mr. Titus has been reproduced and placed in the appendix. The official notice that the springs were classified as a surface source by the EPA was received on March 6, 1978 and the City was required to notify its customers that this source did not meet the Minimum Containment Levels (turbidity) mandated by the "Safe Drinking Water Act of December, 1974," Public Law 93-523. A letter was received from the Oregon Operations Office of the EPA dated April 11, 1978 that the City implement a remedial plan that would bring this source into compliance by January, 1981. The City subsequently responded that the stipulated date of January, 1981 was not a realistic date and the author of this report concurs with the City's position. A more reasonable time schedule is prepared under the recommendations section of this report.

An earlier study prepared in September, 1950, by Cornell, Howland, Hayes and Merryfield, stated in regard to the springs, "A large portion of these supplies, however, apparently consists principally of ground water drawn from gravel deposits adjacent to the Umatilla River. Investigations of these supplies in 1937 disclosed the fact that some of the sources in the infiltration systems delivered water identical in temperature to that of the Umatilla River. It is entirely possible a large portion of the Thornhollow supply is Umatilla River water which has been filtered through the gravel beds along its course."

1. UMATILLA RIVER AS A DIRECT SOURCE

a. Direct intake easterly of City of Pendleton

Prior to the construction of the gravity line to the springs, the City of Pendleton obtained its water supply by means of a direct river intake on the banks of the Umatilla in the vicinity of the easterly extension of S.E. Byers Avenue. This is the City's oldest water right which was issued on November 11, 1885 (Certificate No. 2604) for 2.0 cubic feet per second or 1.29 million gallons per day. The validity of this water right should be determined and, if still valid, could be beneficially utilized by requesting a modification in the diversion location.

b. Johnson Well

The Johnson Well was a former source situated along the banks of the Umatilla River approximately one and three-fourths miles downstream from the Weir house below Thornhollow Springs, with only a crude system of filtering the Umatilla River water. Pumps powered by a manually operated gasoline engine pumped the water through a transmission line to the gravity line to augment spring sources during summer low flows. This source supplied approximately 1.54 cubic feet per second or one million gallons per day. The use at this source was discontinued in the early 1950's because of the poor water quality (high turbidity).

No information could be found on the status of this water right and should be further researched for validity and possible re-use or for the basis of diversion at another location.

c. North Fork of the Umatilla River

This permit was granted on November 18, 1910 and was further granted by an act of the State Legislature (ORS 538.540) during the 1941 session. This water right was for 8.0 cubic feet per second, or 5.184 million gallons per day, subject to water rights existing on March 8, 1941, by the State Legislature. Permit No. 458 was issued but a certificate of right was never granted as this source was never developed. As this water right was granted by the legislature there is some question as to whether the standard rules and regulations apply. During low stream flow periods, water rights up up through 1905 are entitled to all of the available water. If this water right granted by the legislature could be construed to include storage rights for impoundment of surplus waters during the winter months, it would predate the recent water right for storage of surplus winter flows granted to the County Line Improvement District in the West end of the County. There is a notation, however, in the City records that the time limit for this permit expired on October 1, 1961.

2. WENIX SPRINGS

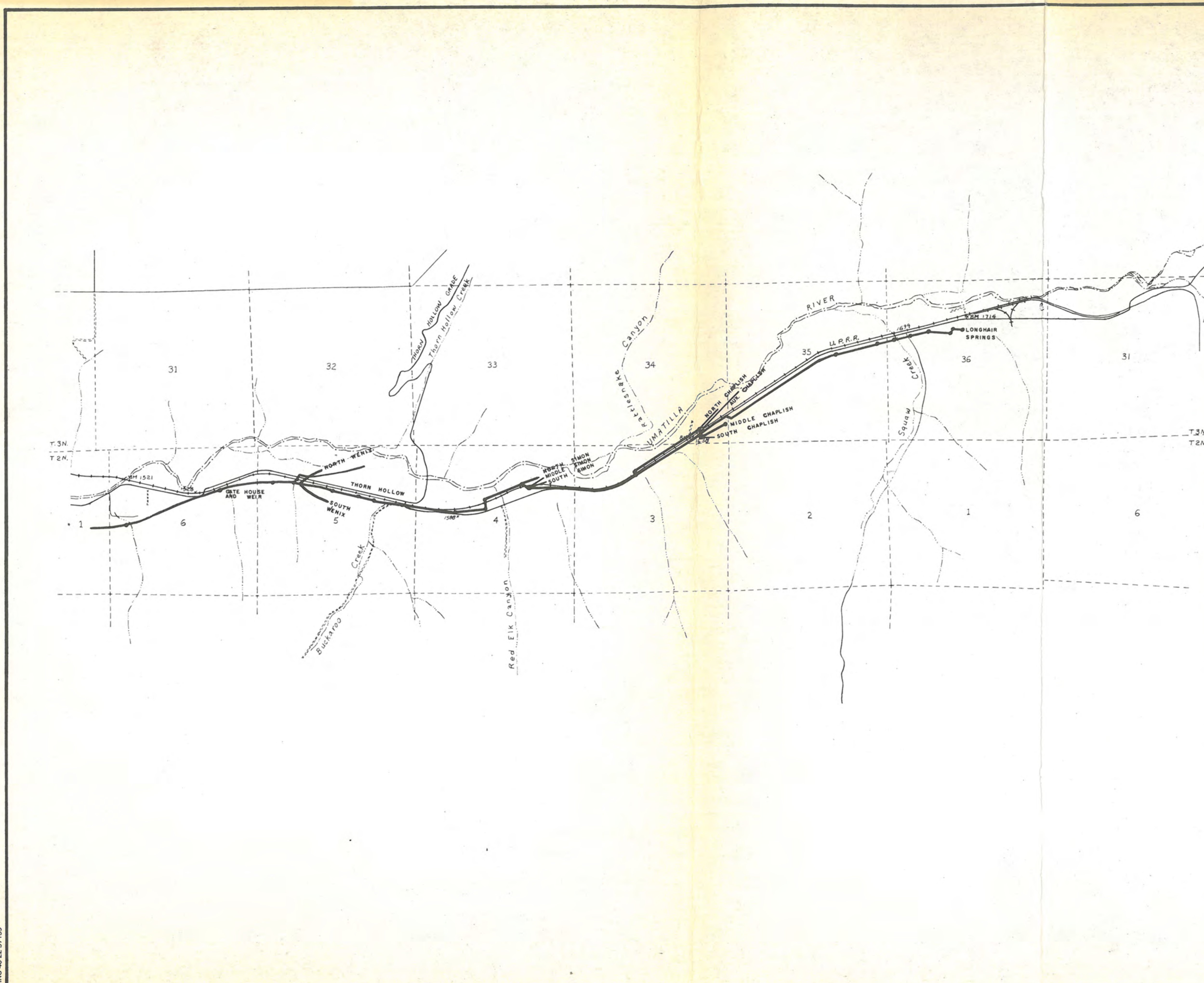
The priority date of this water right is November 28, 1910 (Certificate No. 3927) for 4.0 cubic feet per second or 2.58 million gallons per day. During summer months the yield from this spring and other upstream springs has fallen below the amount of this single water right.

3. CHAPLISH SPRINGS

The priority date of this water right is May 20, 1912 (Certificate No. 7993) for 3.0 cubic feet per second or 1.94 million gallons per day.

4. LONGHAIR AND SQUAW CREEK SPRINGS

The priority date of this water right is April 22, 1929 (Certificate No. 8051) for 2.0 cubic feet per second or 1.29 million gallons per day.



WALLULIS PROJECT NO: 78 - 107

AGENCY PROJECT NO:

PROJECT: PENDLETON WATER STUDY

WALLULIS & ASSOCIATES, INC.
ENGINEERING & PLANNING

108 EAST MAIN
PENDLETON, OREGON 97801 HERMISTON, OREGON 97838
503 · 276 · 1598 503 · 567 · 3331

SCALE: 1" = 3,000'
DATE: MAR. 1979
DES BY:
DR BY:
CK BY:

TITLE:
WATER
COLLECTOR
SYSTEM
for
CITY GRAVITY
SUPPLY

DRAWING: 30

SHEET OF

5. THREE SIMON SPRINGS

The priority date of this water right is also April 22, 1929 (Certificate No. 8052) for 2.7 cubic feet per second or 1.74 million gallons per day.

B. SURFACE WATER SUPPLIES CURRENTLY IN USE

For over the last twenty-five years, the surface water supply has consisted of the combined flow available from Wenix Springs, the Chaplish Spring, Longhair and Squaw Creek Springs, and the Three Simon Springs. The combined sources from these springs from 1953 through 1978 has supplied 69.8% of the City of Pendleton's water supply. For the time period covered between 1971 and 1978 inclusive, this percentage has been reduced to 64.4%. The amount of water supplied from surface waters and deep ground waters for the last twenty-five years is shown on page 32 and the percentage supplied from surface supplies and deep ground waters is shown on page 33.

On March 6, 1978, the United State Environmental Protection Agency mandated that the City springs comply with the minimum contaminant levels in respect to turbidity for surface water supplies. This has resulted in the partial curtailment of the spring sources and increased the amount of water withdrawn by our deep wells from the basalts. To meet the future required maximum approvable turbidity limit of one NTU (Nephelometer turbidity unit) will require this water to be either treated to acceptable turbidity limits or effectively reduce this major source to a very minor supply.

1. CORRELATION OF FLOWS: CITY SPRINGS AND UMATILLA RIVER

A study was made to determine if there was a correlation between Umatilla River flows measured at Cayuse and the yield from the City's springs as claimed by EPA. Records of spring flows measured at the Weir house were only available back to 1974 and the gaging station at Cayuse was only in existence from 1969 to 1974. The proximity of the Cayuse gaging station (1.7 miles east of Cayuse) to the spring sources upstream provided a good basis for comparing for only one year the collective yield of the springs to the flow of the Umatilla River. With the loss of the Cayuse gaging station, we decided to compare the prior annual flows at Cayuse to the recorded Umatilla River flows measured at Pendleton, less the measured flows at Wildhorse Creek for the years of 1969 to 1974. From the information developed in Tables 8 through 13 and the graphs on pages 40 to 48, we find that the Umatilla River flows measured at Pendleton less the measured flows of Wildhorse Creek very closely approximated the six-year history of river flows measured at Cayuse.

The months selected for comparison of Umatilla River flows and spring production were limited to those months of each year when there was a substantial reduction of flow taking place in the river. These months were focused on to investigate the ability of comparing the observed reduction of yield from the springs to the measured flows in the Umatilla River.

TABLE 6

WATER SOURCES

Annually in Millions of Gallons

Year	Gravity	Wells 1 – 5	Well No. 7	Total
1953	994.7	142.5	– 0 –	1,137.2
1954	1,018.6	162.4	– 0 –	1,181.0
1955	1,053.6	205.2	– 0 –	1,258.8
1956	1,136.2	305.5	– 0 –	1,441.7
1957	1,095.4	376.1	– 0 –	1,471.5
1958	1,119.9	352.8	– 0 –	1,472.7
1959	1,222.3	323.3	– 0 –	1,545.6
1960	1,246.0	481.4	– 0 –	1,727.4
1961	1,164.3	510.5	– 0 –	1,674.8
1962	1,089.8	486.6	– 0 –	1,576.4
1963	1,052.4	478.8	– 0 –	1,531.2
1964	1,110.0	369.9	– 0 –	1,479.9
1965	992.5	559.9	– 0 –	1,552.4
1966	973.4	553.0	– 0 –	1,526.4
1967	1,049.0	521.8	107.3	1,678.1
1968	1,025.8	528.9	136.6	1,691.3
1969	1,089.8	378.7	90.6	1,559.1
1970	1,162.7	402.5	64.8	1,630.0
1971	937.7	436.3	63.2	1,437.2
1972	1,032.4	381.8	113.8	1,528.0
1973	1,031.2	441.7	115.4	1,588.3
1974	1,086.5	366.2	87.9	1,540.6
1975	855.6	411.0	211.8	1,478.4
1976	854.4	501.5	104.0	1,459.9
1977	948.3	419.1	89.4	1,456.8
1978	1,019.1	455.8	83.4	1,558.3
Totals	27,361.6	10,553.2	1,268.2	39,183

Water Consumption History	Year	Annual Water Consumption
High Year	1960	1,727.4 million gallons or 5,301 acre feet
Low Year	1953	1,137.2 million gallons or 3,490 acre feet
Average Year	53–77	1,507 million gallons or 4,625 acre feet
Average Day	53–77	4.13 million gallons or 12.67 acre feet

Wells 1953 - 1977 = 29.985%
Springs 1953 - 1977 = 70.015%

TABLE 7

WATER SOURCES

By Percentage

Year	Springs	Wells	Total
1953	87.47	12.53	100
1954	86.25	13.75	100
1955	83.70	16.30	100
1956	78.81	21.19	100
1957	74.44	25.56	100
1958	76.04	23.96	100
1959	79.08	20.92	100
1960	72.13	27.87	100
1961	69.52	30.48	100
1962	69.13	30.87	100
1963	68.73	31.27	100
1964	75.01	24.99	100
1965	63.93	36.07	100
1966	63.77	36.23	100
1967	62.51	37.49	100
1968	60.65	39.35	100
1969	69.90	30.10	100
1970	71.33	28.67	100
1971	65.24	34.76	100
1972	67.57	32.43	100
1973	64.92	35.08	100
1974	70.52	29.48	100
1975	57.87	42.13	100
1976	58.52	41.48	100
1977	65.09	34.91	100
1978	65.40	34.60	100

For years 1971 - 1978 inclusive, deep wells averaged 35.55% of City supply.

TABLE 8

COMPARISON OF MONTHLY SEASONAL AND ANNUAL UMATILLA RIVER FLOWS
(IN ACRE FEET)

CAYUSE GAGE STATION VS PENDLETON STATION, MINUS ATHENA STATION

Year 1968 - 1969

Month	Cayuse	Pendleton	Pendleton/ Cayuse %	Losses & Ac. Ft. Withdrawn for Irrigation
October	5,400	5,230	96.85	
November	25,590	24,522	95.83	
December	31,110	31,839	102.34	
January	69,940	79,880	114.21	
February	24,300	28,630	117.82	
March	53,990	54,390	100.74	
April	117,900	123,640	104.87	
May	58,460	60,555	103.58	
SUB TOTAL	386,690	408,686	105.69	
*June	12,720	13,578	106.75	
SUB TOTAL	12,720	13,578	106.75	
July	5,620	5,182	92.21	438
August	2,740	1,780	64,96	960
September	2,750	2,040	74.18	710
SUB TOTAL	11,110	9,002	81.03	2,108
YEARLY TOTAL	410,520	431,266	105.05	2,108

*Transitional month.

TABLE 9

COMPARISON OF MONTHLY SEASONAL AND ANNUAL UMATILLA RIVER FLOWS
(IN ACRE FEET)

CAYUSE GAGE STATION VS PENDLETON STATION, MINUS ATHENA STATION

Year 1969 - 1970

Month	Cayuse	Pendleton	Pendleton/ Cayuse %	Losses & Ac. Ft. Withdrawn for Irrigation
October	3,830	3,700	96.61	
November	4,050	4,060	100.25	
December	12,760	12,612	98.84	
January	109,300	125,490	114.81	
February	57,110	53,300	93.33	
March	57,140	57,950	101.14	
April	60,990	63,670	104.39	
May	87,910	70,468	80.16	
SUB TOTAL	393,090	391,250	99.53	
*June	19,950	21,776	109.15	
SUB TOTAL	19,950	21,776	109.15	
July	4,470	3,714	83.09	756
August	2,670	1,890	70.79	780
September	3,110	2,849	91.61	261
SUB TOTAL	10,250	8,453	82.47	1,797
YEARLY TOTAL	423,290	421,479	99.57	1,797

*Transitional month.

TABLE 10

COMPARISON OF MONTHLY SEASONAL AND ANNUAL UMATILLA RIVER FLOWS
(IN ACRE FEET)

CAYUSE GAGE STATION VS PENDLETON STATION, MINUS ATHENA STATION

Year 19 70 - 19 71

Month	Cayuse	Pendleton	Pendleton/ Cayuse %	Losses & Ac. Ft. Withdrawn for Irrigation
October	5,140	5,468	106.38	
November	18,580	18,719	100.75	
December	20,280	22,529	111.09	
January	70,430	63,500	90.16	
February	48,690	46,600	95.18	
March	45,790	43,040	93.99	
April	60,240	55,260	91.73	
May	44,610	46,133	103.41	
SUB TOTAL	313,760	301,249	96.01	
*June	31,010	31,386	101.21	
SUB TOTAL	31,010	31,386	101.21	
July	5,730	5,182	90.44	548
August	2,980	2,227	74.73	753
September	3,420	3,000	87.72	420
SUB TOTAL	12,130	10,409	85.81	1,721
YEARLY TOTAL	356,900	343,044	96.12	1,721

*Transitional month.

TABLE 11

COMPARISON OF MONTHLY SEASONAL AND ANNUAL UMATILLA RIVER FLOWS (IN ACRE FEET)

CAYUSE GAGE STATION VS PENDLETON STATION, MINUS ATHENA STATION

Year 19 71 - 19 72

Month	Cayuse	Pendleton	Pendleton/ Cayuse %	Losses & Ac. Ft. Withdrawn for Irrigation
October	4,360	3,870	88.76	
November	20,450	20,073	98.16	
December	46,410	50,780	109.42	
January	46,110	50,690	109.33	
February	78,340	67,250	85.84	
March	167,900	159,610	95.06	
April	73,230	67,798	92.58	
May	76,680	71,950	93.83	
SUB TOTAL	513,480	492,021	95.82	
*June	20,320	19,749	97.19	
SUB TOTAL	20,320	19,749	97.19	
July	4,800	4,070	84.79	730
August	3,130	2,200	70.29	930
September	3,130	2,520	80.51	610
SUB TOTAL	11,060	8,790	79.48	2,270
YEARLY TOTAL	544,860	520,560	95.54	2,270

*Transitional month.

TABLE 12

COMPARISON OF MONTHLY SEASONAL AND ANNUAL UMATILLA RIVER FLOWS
(IN ACRE FEET)

CAYUSE GAGE STATION VS PENDLETON STATION, MINUS ATHENA STATION

Year 19 72 - 19 73

Month	Cayuse	Pendleton	Pendleton/ Cayuse %	Losses & Ac. Ft. Withdrawn for Irrigation
October	3,710	3,530	95.15	
November	5,760	5,660	98,26	
December	32,990	28,638	86.81	
January	31,870	29,237	91.74	
February	14,780	15,469	104.66	
March	28,070	27,720	98.75	
April	27,510	26,310	95.64	
May	16,600	16,475	99.25	
SUB TOTAL	161,290	153,039	94.88	
*June	4,730	3,990	84.36	
SUB TOTAL	4,730	3,990	84.36	
July	2,770	2,170	78.33	600
August	2,320	1,730	74.57	590
September	3,080	2,640	85.71	440
SUB TOTAL	8,170	6,540	80.05	1,630
YEARLY TOTAL	174,190	163,569	93.90	1,630

*Transitional month.

TABLE 13

COMPARISON OF MONTHLY SEASONAL AND ANNUAL UMATILLA RIVER FLOWS
(IN ACRE FEET)

CAYUSE GAGE STATION VS PENDLETON STATION, MINUS ATHENA STATION

Year 19 73 - 19 74

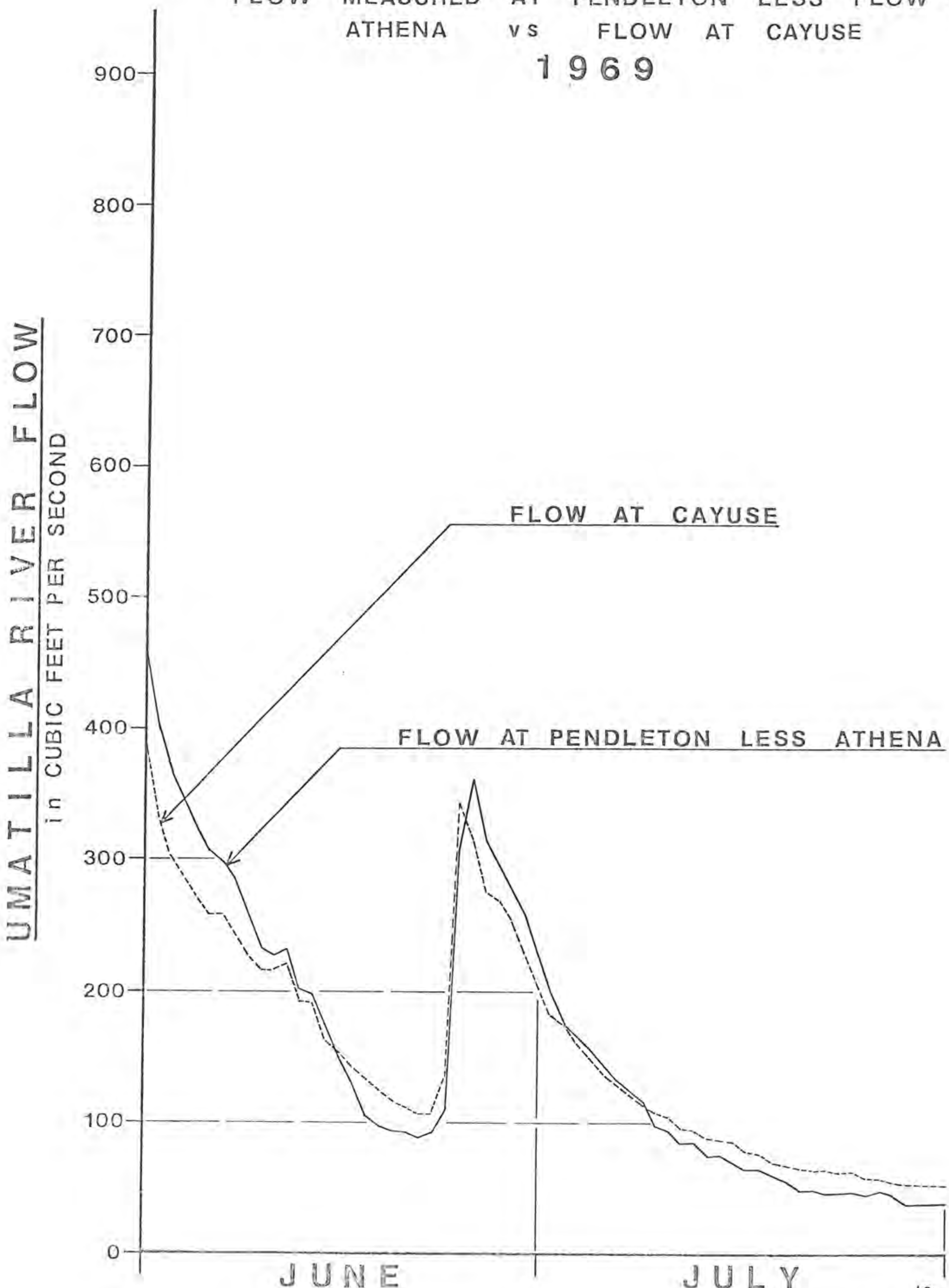
Month	Cayuse	Pendleton	Pendleton/ Cayuse %	Losses & Ac. Ft. Withdrawn for Irrigation
October	3,580	3,560	99.44	
November	46,820	41,566	88.88	
December	104,000	104,530	100.51	
January	85,590	80,610	94.18	
February	53,370	51,790	97.04	
March	75,680	78,280	103.44	
April	129,500	136,350	105.28	
May	94,660	94,739	100.08	
SUB TOTAL	593,200	591,425	99.70	
*June	52,230	52,993	101.46	
SUB TOTAL	52,230	52,993	101.46	
July	8,520	8,280	97.18	240
August	3,340	3,220	96.41	120
September	2,750	2,510	91.27	240
SUB TOTAL	14,610	14,010	95.89	600
YEARLY TOTAL	660,040	658,428	99.76	600

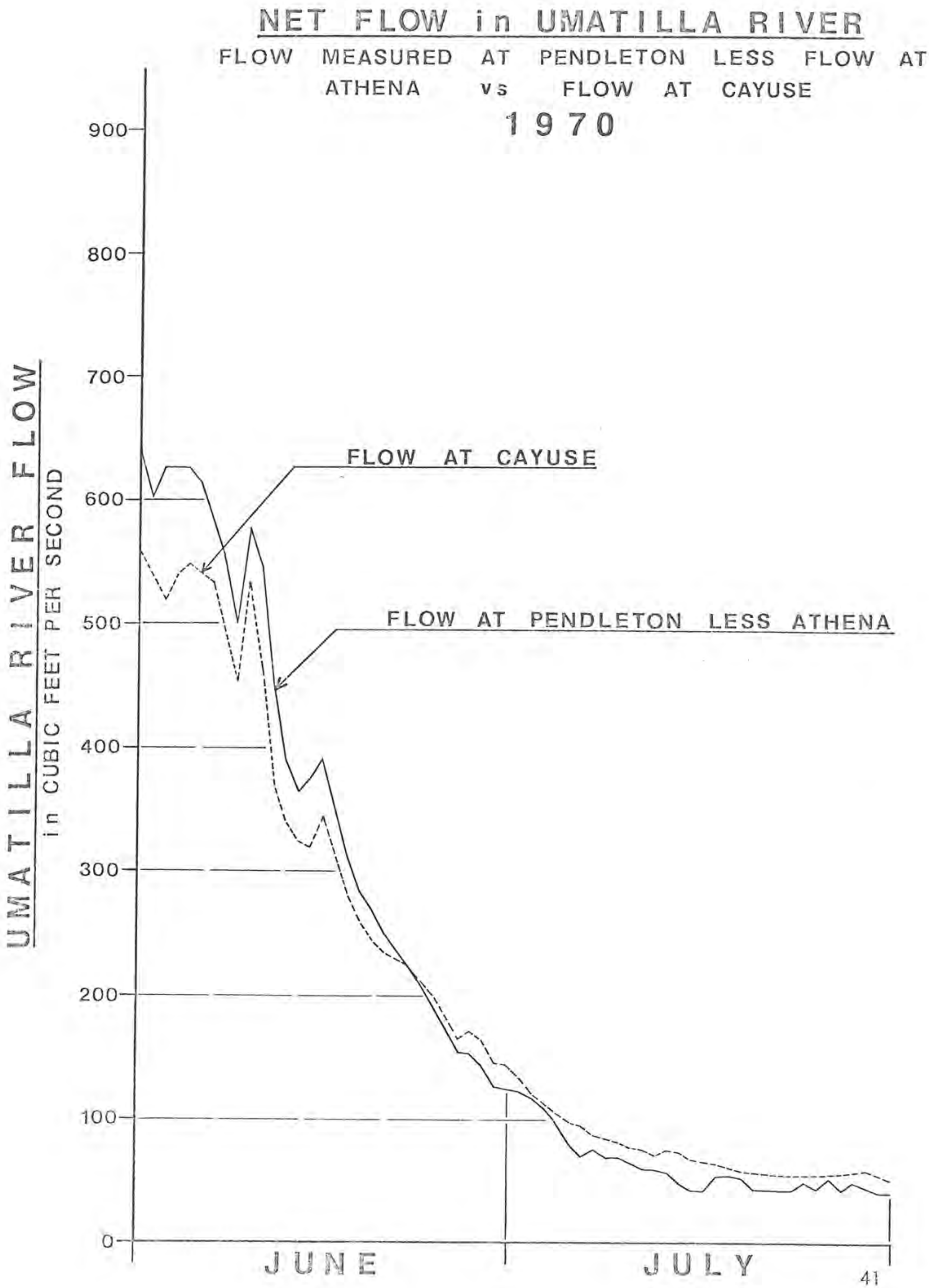
*Transitional month.

NET FLOW in UMATILLA RIVER

FLOW MEASURED AT PENDLETON LESS FLOW AT ATHENA vs FLOW AT CAYUSE

1969



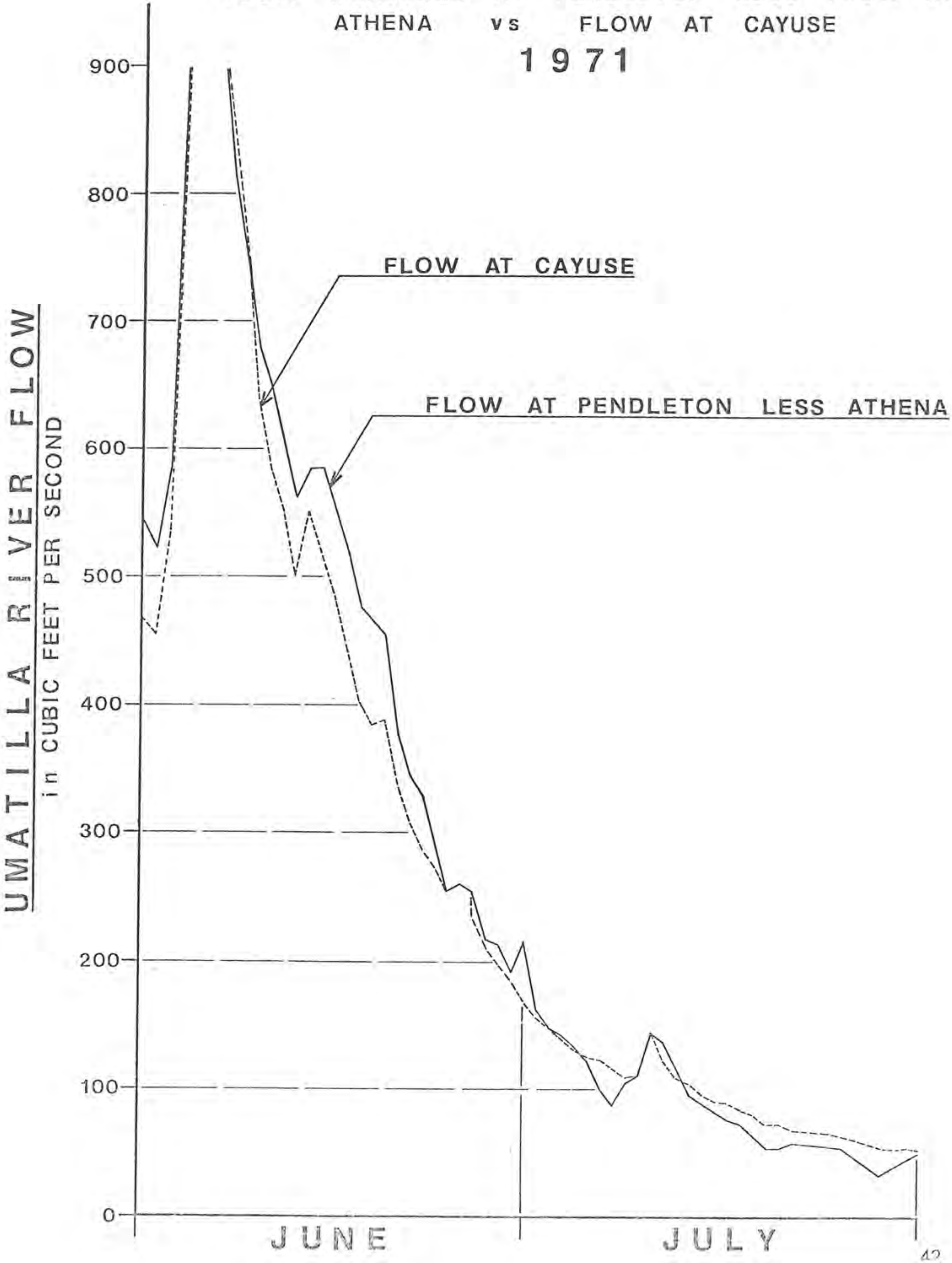


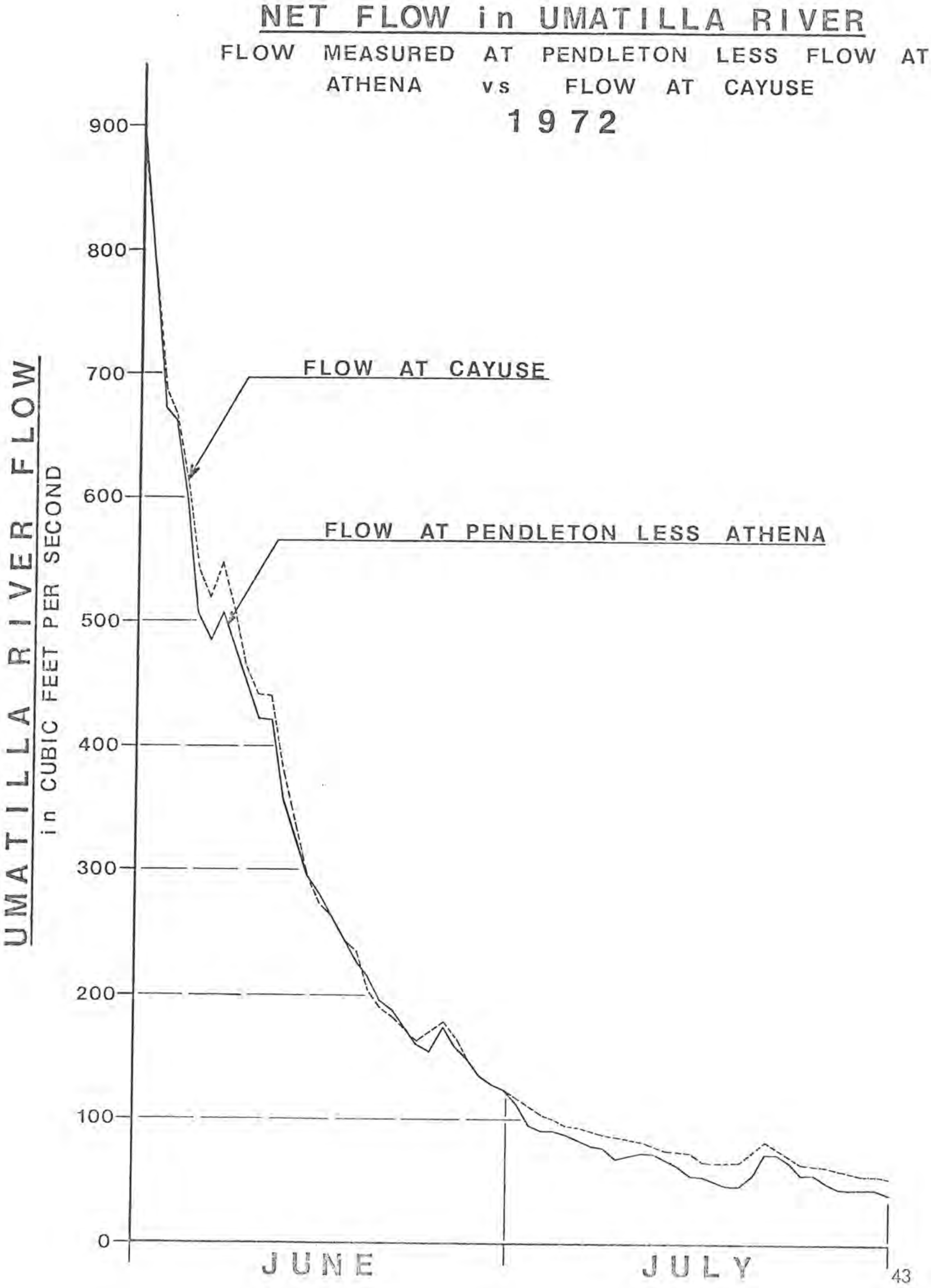
NET FLOW in UMATILLA RIVER
FLOW MEASURED AT PENDLETON LESS FLOW AT ATHENA vs FLOW AT CAYUSE
1970
UMATILLA RIVER FLOW
in CUBIC FEET PER SECOND
900
800
700
600
500
400
300
200
100
0
FLOW AT CAYUSE
FLOW AT PENDLETON LESS ATHENA
JUNE
JULY

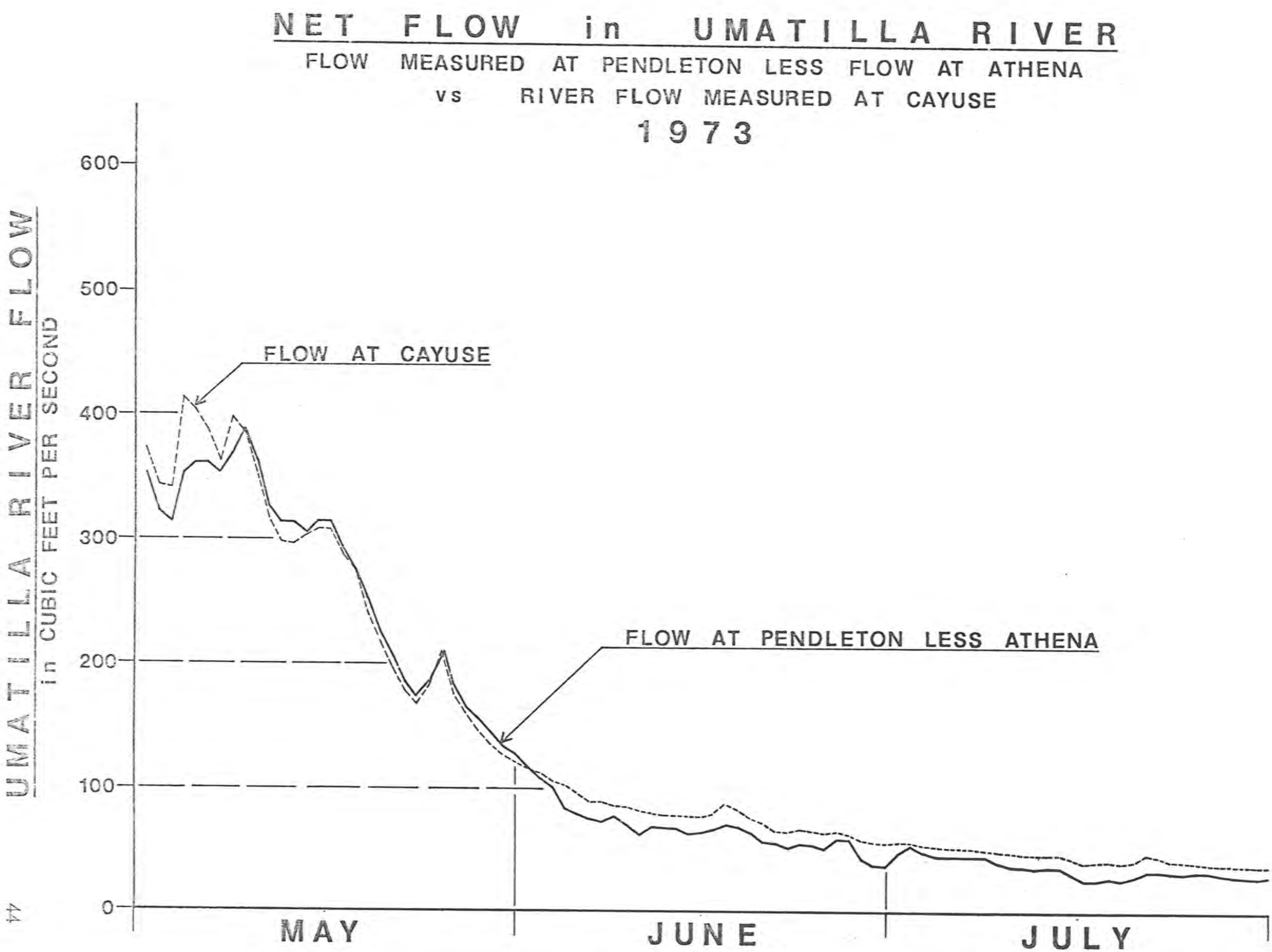
NET FLOW in UMATILLA RIVER

FLOW MEASURED AT PENDLETON LESS FLOW AT ATHENA vs FLOW AT CAYUSE

1971

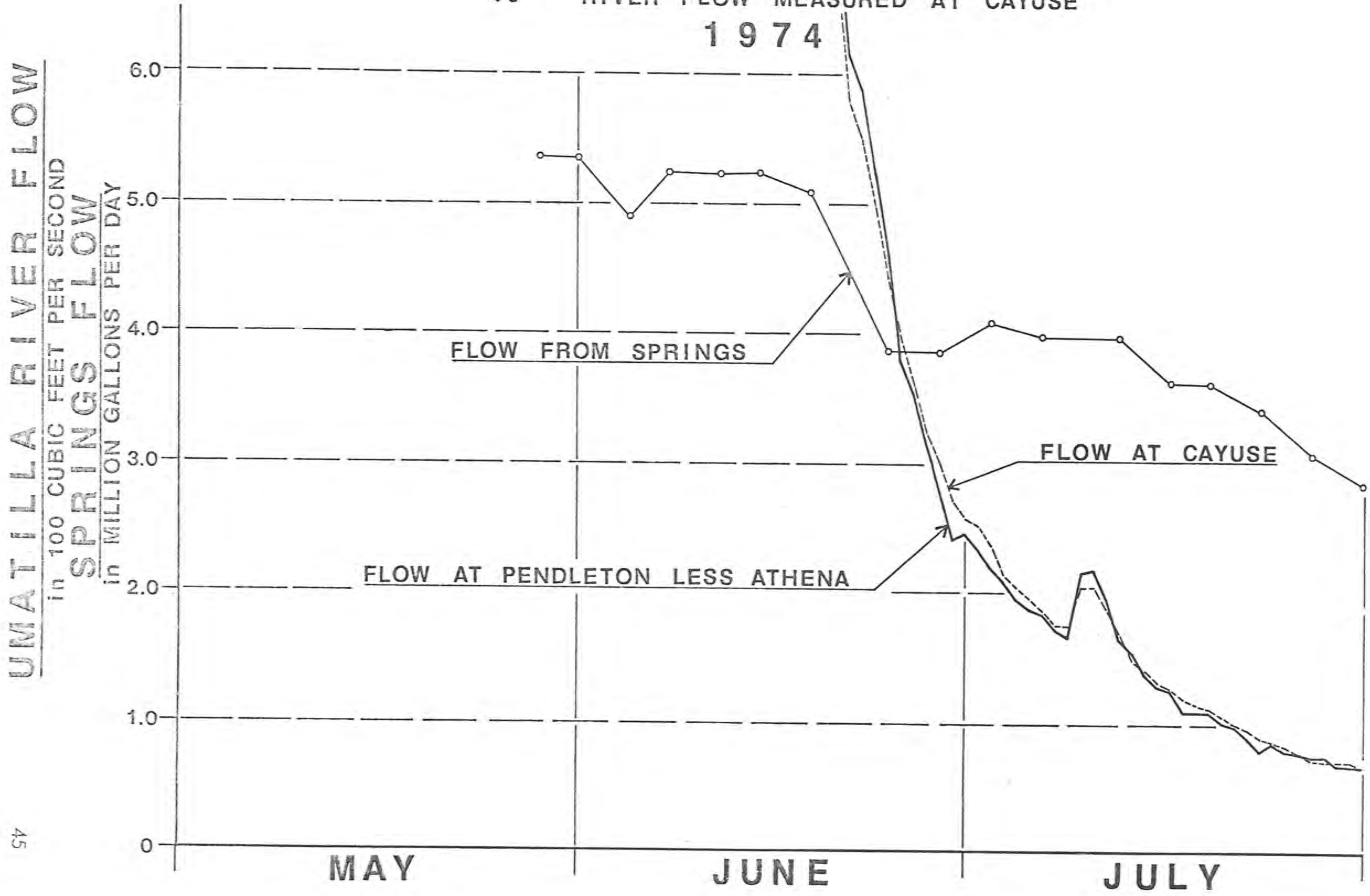


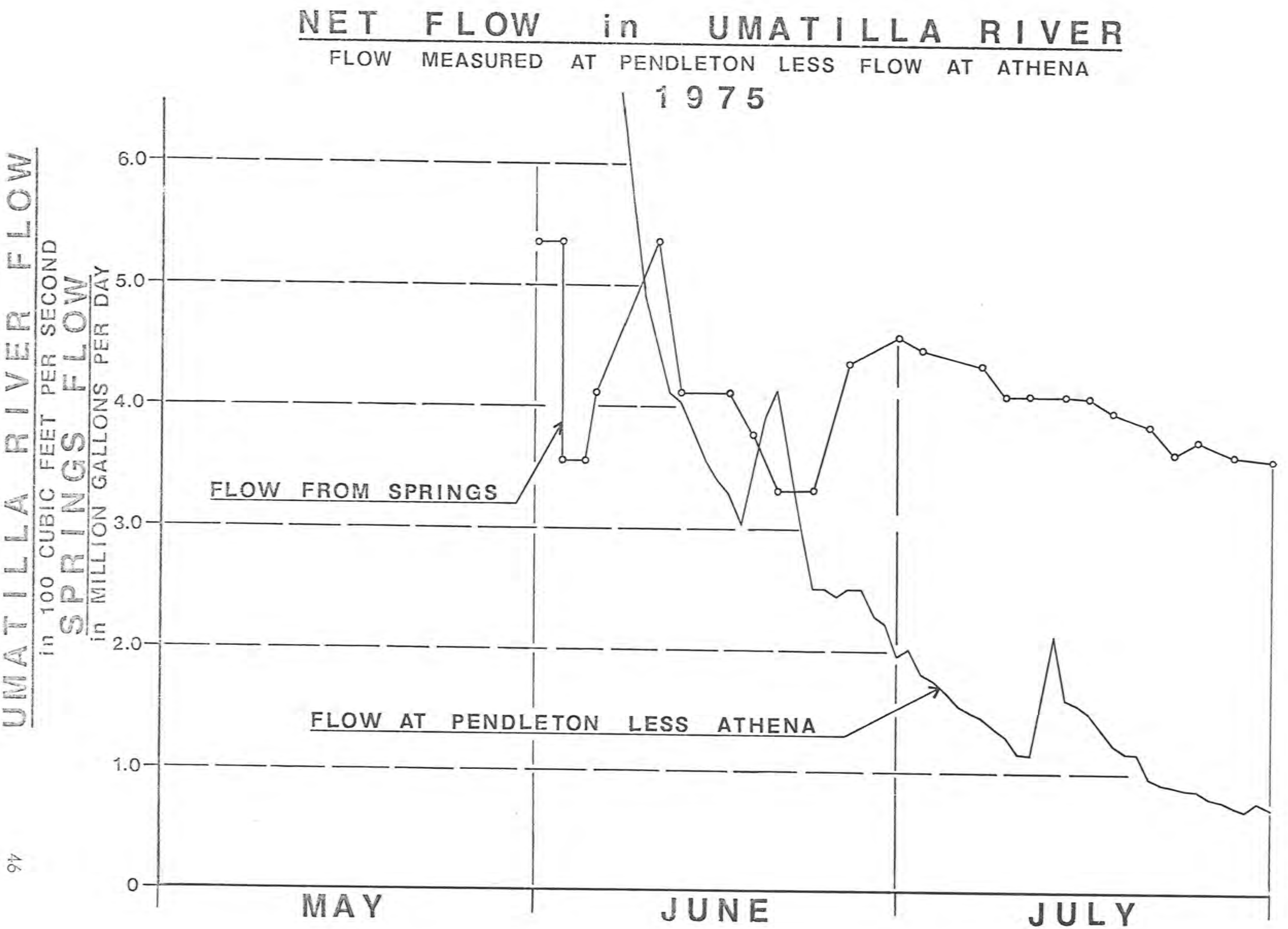

NET FLOW in UMATILLA RIVER
FLOW MEASURED AT PENDLETON LESS FLOW AT ATHENA vs FLOW AT CAYUSE
1972
UMATILLA RIVER FLOW
in CUBIC FEET PER SECOND
900
800
700
600
500
400
300
200
100
0
FLOW AT CAYUSE
FLOW AT PENDLETON LESS ATHENA
JUNE
JULY


NET FLOW in UMATILLA RIVER
FLOW MEASURED AT PENDLETON LESS FLOW AT ATHENA
vs RIVER FLOW MEASURED AT CAYUSE
1973
UMATILLA RIVER FLOW
in CUBIC FEET PER SECOND
600
500
400
300
200
100
0
FLOW AT CAYUSE
FLOW AT PENDLETON LESS ATHENA
MAY
JUNE
JULY

NET FLOW in UMATILLA RIVER

FLOW MEASURED AT PENDLETON LESS FLOW AT ATHENA
vs RIVER FLOW MEASURED AT CAYUSE
1974

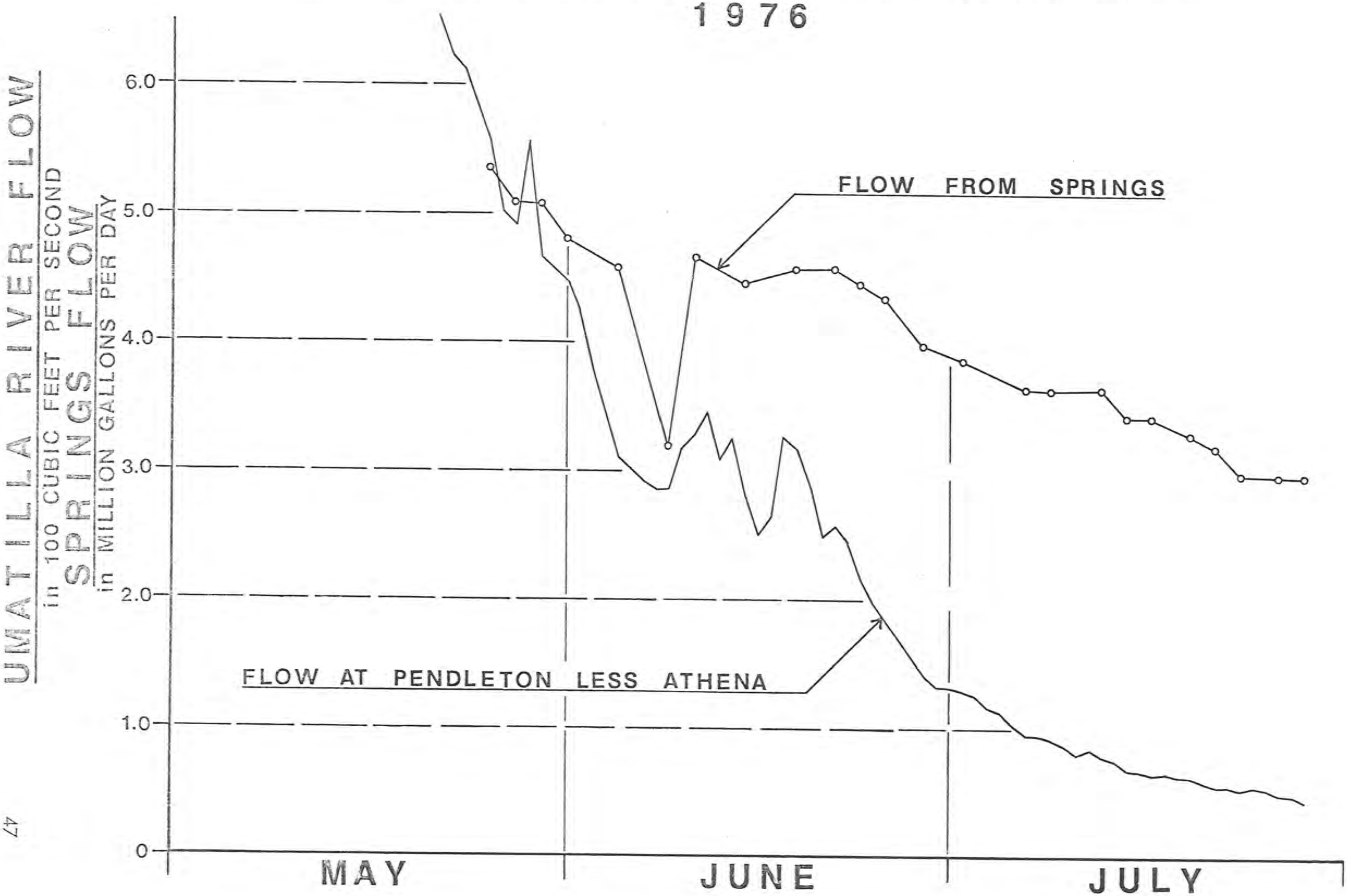



NET FLOW in UMATILLA RIVER
FLOW MEASURED AT PENDLETON LESS FLOW AT ATHENA
1975
UMATILLA RIVER FLOW
in 100 CUBIC FEET PER SECOND
SPRINGS FLOW
in MILLION GALLONS PER DAY
6.0
5.0
4.0
3.0
2.0
1.0
0
FLOW FROM SPRINGS
FLOW AT PENDLETON LESS ATHENA
MAY
JUNE
JULY

NET FLOW in UMATILLA RIVER

FLOW MEASURED AT PENDLETON LESS FLOW AT ATHENA

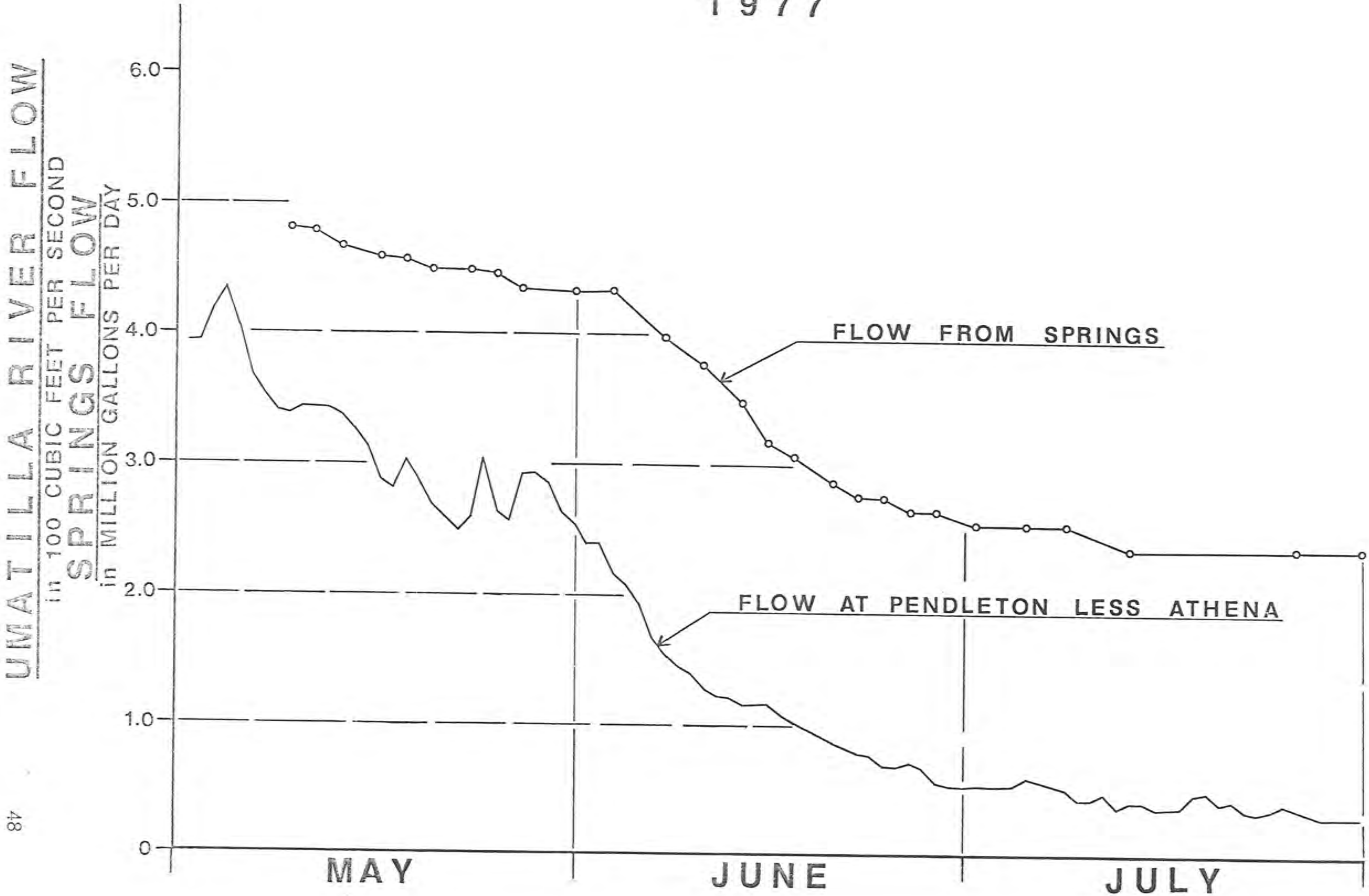
1976



NET FLOW in UMATILLA RIVER

FLOW MEASURED AT PENDLETON LESS FLOW AT ATHENA

1977



The City's springs during the spring months of each year are added successively in an upstream sequence to the supply system as the demand of the City dictates. In the years of 1975 through 1978 there were periods during most of the spring months when the spring sources were not used and the deep wells were utilized. The reasons for the use of wells during these spring periods could be because of high turbidity or maintenance on the spring collection systems.

We graphically plotted the river flows and the spring flows for the years 1974-1977 inclusive on graphs shown on pages 45 through 48. In order to plot the spring flows it was necessary to plot spring yields in terms of one millions of gallons per day to an equivalent scale of 100 cubic feet per second which is 64.63 million gallons per day. This scale distortion of 64.63 to 1.0 is in itself not important as what we were attempting to discern was if the two flows followed parallel tracks in the months of June and in July of each year. In most years, with some allowance for time lag, the tracking patterns showed similarity and the following information was obtained from these graphs.

TABLE 14

RECORDED RIVER FLOWS IN CUBIC FEET PER SECOND

	SELECTED SPRING FLOWS IN MGD				
YEAR	2.5	3.0	3.5	4.0	4.5
1974	N.A.	70	90	165	575
1975	N.A.	63	72	131	180-222
1976	N.A.	42-52	77	142	225-308
1977	42	95	118	152	267-280
1978					
Range	42	42-95	72-118	131-152	180-575

By reviewing Table 14 the spring flows and the Umatilla River flows do have similar diminishing yield patterns. There is some overlapping of recorded river flows when the springs are producing 3.0 and 3.5 MGD. Other than this overlap the spring production and river flows do have similar patterns as claimed by EPA.

C. OTHER SURFACE WATER SUPPLIES INVESTIGATED

1. McKAY RESERVOIR

The capacity of this reservoir is 73,800 acre feet which is equivalent to sixteen years of average annual water consumption by the City of Pendleton. Earlier studies indicated that it might be possible to purchase stored water from the McKay Reservoir from the Bureau of Reclamation. After several phone discussions and a personal visitation with Mr. Bob Brown, Project Superintendent of the Bureau of Reclamation's Central Snake Project Office in Boise, Idaho, we were informed that while there may have been surplus storage water available for municipal and industrial use several years ago that this opportunity is no longer present. To provide written documentation of the Bureau's position we have included copies of correspondence from our firm to the Bureau and their response in the Appendix. A recent news release in the East Oregonian, on March 24, 1979, stated that the Stanfield-Westland Irrigation District contracted for an additional 6,181 acre feet of unallocated water and that there is a balance of 5,819 acre feet of unallocated water remaining in McKay Reservoir. The true legal position on storage at McKay Reservoir should be further investigated.

2. MISSION RESERVOIR ON THE UMATILLA RIVER

For over a quarter of a century the construction of a dam in the general location of Mission has had various levels of support. The Umatilla Indian Reservation-Confederated Tribes have historically assumed a leadership role in opposing the construction of a dam in this location.

Without a unified local support including the backing of the Confederated Tribes of the Umatilla Indian Reservation, the prospects of the construction of this reservoir are very slight. The Environmental Impact Statement alone on a project of this magnitude could result in endless delays before any actual implementation of a plan could proceed.

The pursuit of this alternative without support of the Confederated Tribes of the Umatilla Indian Reservation would probably be a repeat of former futile exercises to construct this dam on the Umatilla River. As of this writing, there appears to be no justification for pursuing this alternative any further.

3. COLUMBIA RIVER

Several cities in Umatilla County experiencing a continuous lowering of their deep groundwater tables are looking towards the Columbia River as their eventual source of water. These cities include Hermiston, Umatilla, and Echo. A preliminary investigation disclosed that this alternative would require a minimum of twenty-five miles of pipeline, development of the source that may require treatment, and probably multiple pumping stations along any selected route.

A project of this magnitude could not be planned and completed within a time frame acceptable to EPA to improve the City of Pendleton's water quality. This alternaitve has therefore been removed from consideration in this Study, but should not be entirely discounted as a potential future source of supply for the City.

4. OTHER RESERVOIR SITES ON THE UMATILLA RIVER

In prior studies several potential sites on the upper reaches of the Umatilla River were considered. The specific reservoir sites were as follows:

a. North Fork of the Umatilla River
b. South Fork of the Umatilla River
c. North Fork of Meacham Creek
d. Lick Creek near the Umatilla River
e. Squaw Creek near the Umatilla River
f. Ryan Creek near the Umatilla River

The first four of the above impoundments were each determined to be of sufficient capacity to store up to 9,200 acre feet. Based on recent consumptive use records, any one of these four dams would provide adequate water for a population of 26,700 persons. The latter two reservoir impoundment sites were of somewhat less capacity and without any specific capacity stated.

Reservoirs on the upper reaches of the Umatilla River would probably gain more acceptance than downstream impoundments which would require the taking of agricultural farmable grounds out of production. This opinion is based on "Findings & Conclusions" Nos. 28, 29, 45, and 50 of the Umatilla River Basin Study, dated June, 1963. A copy of these findings and conclusions is included in the Appendix.

Costs estimated in earlier studies for upstream dams on the Umatilla River were based on summer releases into the stream channels to be picked up again from the river at a downstream location. Earlier cost estimates for upstream impoundments after adjustment to current costs are used later in this report. These earlier cost estimates were admittedly based on minimal data and are not represented to be accurate without additional detail study.

In the consideration of the concept of upstream storage impoundments, the possibility of the existing water right on the North Fork of the Umatilla River certainly needs re-evaluation as to the potential right to store surplus waters at the rate of 8.0 cubic feet per second, or 5.16 million gallons per day. Information in City records indicates that the permit for this water right may have expired in October, 1961. If this old water right could be determined to be valid it would have precedence over the recent water right recently granted for 75 cfs to the "County Line Improvement District" in the West end of the County. Presently the Westland Irrigation canal, which is utilized by the County Line Improvement District, can only carry approximately 30 cfs of the 75 cfs granted water right.

In water shortage areas, such as Umatilla County the documented validity of this water right on the North Fork may in itself outweigh the economics in consideration of alternative sites. Earlier studies prior to the granting of the water right to the County Line Improvement District indicated that with Cold Springs Reservoir as the only prior storage right that there remained ample surplus winter run-off from the watershed to fill a reservoir on the Upper Umatilla River to a capacity of 9,200 acre feet.

In discussions with the office staff of the Umatilla County Watermaster's office, we have been advised that the fish ladders at Three Mile Dam (above mouth of the Umatilla River) has a water right for 75 cfs. There is currently a proposal to abandon this dam and replace this water right with a conduit from the new powerhouse to be built at the McNary Dam. Even if this dam is eliminated the State Fish and Wildlife Department would probably insist on maintaining the minimum flow of 75 cfs for the migration of fish to the upper reaches of the Umatilla River. We have prepared Table 15 on page 53 which shows the amount of surplus water available for storage above a base flow of 75 cubic feet per second measured at the gage station near the City of Umatilla. This table demonstrates that there is ample water available for storage in the upper reaches of the Umatilla River.

We have likewise prepared Table 16 on page 54 to determine the amount of the surplus water available from the Umatilla River above a base flow of 150 cfs. This is prepared in the eventuality that the City cannot utilize the North Fork water right and that a minimum river flow of 150 cfs would be required to satisfy the needs of the County Line Improvement District and the Three Mile Dam rights. Table 16 shows the amount of surplus water available on a monthly basis for each year above the minimum required base flow of 150 cfs measured at the Umatilla River. Table 16 indicates that there is normally ample water available for upstream storage even with the larger minimum base flow of 150 cfs. The unknown factor that would have to be determined in either case would be the ability of any impoundment site to collect and store the surplus water at that particular location during the time periods when surplus flows would be occurring at the mouth of the Umatilla River. A review of the flow records at the Umatilla River gaging station above Gibbon indicates that surplus flows normally track surplus flows recorded at the river's mouth except for winter months when the snow pack remains at the higher elevations and melts at the lower elevations or during periods of precipitation when rain occurs at the lower elevations and snow occurs at the higher elevations. Water in any upstream impoundment could be stored in the early months of each year with the consent of earlier water right holders. The stored water could subsequently be released in the event a low run-off year infringed on these earlier water rights.

TABLE 15

UMATILLA RIVER MONTHLY FLOWS MEASURED NEAR UMATILLA, OREGON
ABOVE BASE FLOW OF 148.76 ACRE FEET PER DAY (75 CFS)

YEAR	OCTOBER	NOVEMBER	DECEMBER	JANUARY	FEBRUARY	MARCH	APRIL	MAY	JUNE	TOTAL YEARLY SURPLUS WATER
1955-56	0	18,768	73,229	79,029	39,916	100,089	77,448	63,499		451,978
1956-57	209	7,608	34,389	5,629	44,735	87,559	79,318	51,669		311,116
1957-58	5,269	7,608	37,539	42,449	102,835	38,099	177,438	55,629		466,866
1958-59	0	15,878	56,769	83,179	48,485	50,349	53,908	14,949		323,517
1959-60	15,729	19,938	8,379	15,419	32,136	64,989	33,648	39,999		230,237
1960-61	0	20,598	9,249	9,279	80,665	81,179	19,628	8,779		229,377
1961-62	0	4,858	15,289	29,999	15,135	38,809	48,578	22,559		175,227
1962-63	3,819	9,478	26,279	11,419	74,415	22,409	51,588	15,879		215,286
1963-64	0	5,078	6,169	12,699	17,656	22,039	51,218	18,619		133,478
1964-65	0	1,428	114,489	140,889	123,835	30,209	64,948	6,209		482,007
1965-66	0	778	2,349	4,069	4,695	34,359	12,748	0		58,998
1966-67	0	2,208	28,559	48,239	32,505	10,669	6,698	33,429		162,307
1967-68	0	2,348	29,349	14,509	60,296	5,559	0	0		112,061
1968-69	529	17,288	26,499	80,289	31,435	50,509	125,738	32,509		364,796
1969-70	4,459	3,878	7,479	117,189	56,255	63,389	51,648	46,969		351,266
1970-71	3,889	10,958	13,289	68,619	48,195	34,019	31,348	10,759	7,308	221,076
1971-72	1,249	12,678	46,089	51,689	91,336	221,489	56,388	44,279		525,197
1972-73	2,649	4,198	28,359	32,159	5,825	9,929	0	0		83,119
1973-74	0	31,398	115,189	122,189	70,895	82,679	162,138	72,549	13,138	657,037
1974-75	79	5,408	12,299	95,689	44,315	77,219	54,788	73,459		363,256
1975-76	2,679	9,338	84,049	90,139	42,466	43,599	109,838	35,029		417,137
1976-77	0	2,198	1,889	3,929	655	4,869	11,508	0		25,048
1977-78										
AVERAGE	1,843	9,723	35,326	52,668	48,577	53,364	58,207	29,399		289,107

* Additional Run-off in September - added 2,578 acre feet to total

TABLE 16

UMATILLA RIVER MONTHLY FLOWS MEASURED NEAR UMATILLA, OREGON
ABOVE BASE FLOW OF 297.52 ACRE FEET PER DAY (150 CFS)

YEAR	OCTOBER	NOVEMBER	DECEMBER	JANUARY	FEBRUARY	MARCH	APRIL	MAY	JUNE	TOTAL YEARLY SURPLUS WATER
1955-56	0	14,306	68,618	74,418	35,602	95,478	72,986	58,888		420,296
1956-57	0	3,146	29,778	1,018	40,570	82,948	74,856	47,058		279,374
1957-58	658	3,146	32,928	37,838	98,670	33,488	172,976	51,018		430,722
1958-59	0	11,416	52,158	78,568	44,320	45,738	49,446	10,338		291,894
1959-60	11,118	15,476	3,768	10,808	27,822	60,378	29,186	35,388		193,944
1960-61	0	16,136	4,638	4,668	76,500	76,568	15,166	4,168		197,844
1961-62	0	396	10,678	25,388	10,970	34,198	44,116	17,948		143,694
1962-63	0	5,016	21,668	6,808	70,250	17,798	47,126	11,268		179,934
1963-64	0	616	1,558	8,088	13,342	17,428	46,756	14,008		101,796
1964-65	0	0	109,878	136,278	119,670	25,598	60,486	1,598		453,508
1965-66	0	0	0	0	530	29,748	8,286	0		38,564
1966-67	0	0	23,948	43,628	28,340	6,058	2,236	28,818		133,028
1967-68	0	0	24,738	9,898	55,982	948	0	0		91,566
1968-69	0	12,826	21,888	75,678	27,270	45,898	121,276	27,898		332,734
1969-70	0	0	2,868	112,578	52,090	58,778	47,186	42,358		315,858
1970-71	0	6,496	8,678	64,008	44,030	29,408	26,886	6,148	2,846	188,500
1971-72	0	8,216	41,478	47,078	87,022	216,878	51,926	39,668		492,266
1972-73	0	0	23,748	27,548	1,660	5,318	0	0		58,274
1973-74	0	26,936	110,578	117,578	66,730	78,068	157,676	67,938	8,676	634,180
1974-75	0	946	7,688	91,078	40,150	72,608	50,326	68,848		331,644
1975-76	0	4,876	79,438	85,528	38,152	38,988	105,376	30,418		382,776
1976-77	0	0	0	0	0	258	7,046	0		7,304
1977-78										
AVERAGE										

5. CHENEY SPRINGS

The Cheney Springs are situated in a marshy area of the McKay Creek Valley floor to the east of McKay Creek, west of the sidehills and an existing sewer trunkline, approximately 2,000 feet upstream from the City's wastewater treatment plant. From an interview with Max Cheney, whose family has owned the property since 1900, and an onsite investigation, the following information was obtained:

1. The springs cover a considerable area where the groundwater surfaces and is channeled to a pond area where the water returns through the gravels to McKay Creek.

2. That spring flows increase and decrease with flow of McKay Creek. That during periods of irrigation at elevations above the springs the flows from the springs increased dramatically.

3. That during the summer of 1976 when there was no flow in McKay Creek the portion of the springs on his property dried up to the point that he had to dig a trench to gain access to the groundwater for his stock.

4. That the valley floor is underlain with gravels with the water table normally a few feet below the ground surface.

5. That he abandoned a shallow well at the foot of the hill to service his home because of concern of contamination.

6. That over the years of diverting water from McKay Creek he has found a shallow bedrock shelf in the bed of McKay Creek for approximately a quarter of a mile above the irrigated fields in which the springs occur. It is Mr. Cheney's opinion that this shelf forms a submerged dam forcing the shallow ground water to follow old stream beds to the east of McKay Creek and surface in his lower field.

7. That his irrigation rights are based on stream flows on McKay Creek that pre-date the construction of McKay Reservoir.

Based on this field investigation it would appear that these springs would be placed in the same category by EPA as a surface source and that this source would require treatment prior to use. Further, there are no old water rights on these springs that can be acquired. These springs have been an historical supplemental source of McKay Creek and as such have long ago been appropriated by downstream users.

CHAPTER IV

GROUNDWATER SUPPLY

A. PRESENT CITY WELLS - GENERAL

From February through September 1961 an investigation was conducted to determine if City Wells 1, 2, 3, 4, and 5 were all pumping from the same underground reservoir. At the First National Bank, which had an abandoned deep well, a float recorder was placed in the well to determine what, if any, responses would be recorded. As each of the five wells were operated an immediate response was noted on the float recorder even though considerable distances were involved as shown below. (See map on page 57)

Well No.	Well Name	Distrance from the First National Bank Well
1	Byers Street	5,200 feet
2	Round-up	2,900 feet
3	S. W. 21st Street	5,700 feet
4	State Hospital	7,000 feet
5	Stillman	1,500 feet

With large spacing between the City Wells this investigation demonstrated that a large underground reservoir of confined deepwater existed below the City. The outer boundaries of this underground reservoir had a minimum east-west dimension of 12,000 feet (2.27 miles), and a minimum north-south dimension of 5,000 (0.95 miles). Well No. 6 which is presently being utilized as a monitoring well, also immediately responds to pumping at any of the other City Wells. This information effectively extends the east-west dimension of the underground reservoir to a minimum of 14,100 feet (2.67 miles) and the north-south dimension to 6,300 feet (1.19 miles).

A hydraulic interconnection may or may not exist between Well No. 7 (Mission Well) and the five City wells that have been determined to be drawing from the common underground reservoir underlying the City.

1. City Wells No's. 1, 2, 3, 4, and 5

Records of the amounts withdrawn from the underground reservoir under the City for the years of 1948 to 1952 could not be found. From 1953 through 1978 the amount pumped by deep wells No. 1, 2, 3, 4, and 5 is 10,553,200 gallons (32.389 acre feet). This amount of water would be the equivalent of meeting all the water needs of the present City's population of 15,000 for seven years and four months.

AIRPORT
Pendair Heights
CITY WELL NO. 2
CITY WELL NO. 4
FIRST NATIONAL BANK WELL
CITY WELL NO. 5
MILTENBERGER WELL
CITY WELL NO. 1
PENDLETON
BROGOITTI WELL
SEWAGE PLANT WELL
HOSPITAL WELL
CITY WELL NO.3
CITY WELL NO. 6
GLENDALE / SUNSET WELL
12,000 FT
2.27 MILES
6,300 FT 1.19 MILES
22"
18"
30"
24"
WATER TREATMENT PLANT PROPOSED SITE 'A'
THORNHOLLOW 30"
WATER TREATMENT PLANT PROPOSED SITE 'B'
23"
GRAVITY
WATER
LINE
WATER TREATMENT PLANT PROPOSED SITE 'C'
CITY WELL NO. 7
HAGUEWOOD WELL
UMATILLA RIVER
UNION PACIFIC
INTERSTATE 80 NORTH
I-80 N
Mission
Patawa Creek
McKay Creek
1/2
MILES
0
2000
4000
FEET

We have prepared a chart on page 59 for the years 1959-1978 which are the only years that all of the data was available for wells No. 1, 2, 3, 4, and 5. During this interval of time the City has withdrawn 8.553 billion gallons (26,250 acre feet) of water from the great reservoir of deep ground waters underlying the City. The ground water withdrawn over this 19-year period would have provided all the water needed to supply the present City's population for six years. This chart shows that the water level of the underground reservoir has dropped 29.05 ft. in this 20 year period, for an average lowering of 1.45 feet/year. The rate of decline has, however, accelerated in the more recent years of 1975 through 1977 for an average lowering of 2.76 feet per year and 4.90 feet last year. It has been the experience of several communities that once the lowering of the underlying water table begins, the trend increases.

In Table 17 on page 60 we show the individual amount of annual production for all of the six City wells since 1967 when Well No. 7 (Mission) first came on line.

The graphic plot on page 59 shows that a fairly good correlation between the declining level of the water table level and the withdrawal rates can be made. This would seem to indicate that the underground reservoir is fairly uniform geometrically. The concern is that the slope of the line depicting the lowering of the underground water table has steepend upwardly consistently over the last seven years. Several other basins which have experienced a declining water table level also simulate the effect of withdrawing water from a bowl.

Wells 1 through 5 have been in a consistent overdrafting condition (mining or depletion of ground water) since they were first placed in service. Overdrafting is defined as when the annual withdrawl rate exceeds the amount of annual recharge.

As a part of this Study we had Well No. 4 (State Hospital) analyzed for the age of the water. The age dating was performed by Washington State University, utilizing the Carbon ^{14}C method. The results of this test indicated that the age of the water was approximately 2,570 years plus or minus 135 years, or that it may possibly contain a mix of modern waters (up to 34%) and older waters. Another factor that can effect the age dating process would be the contact of the water with limestone. With the large land surface area over the underground reservoir there is a possibility that precipitation could have penetrated through the soil profile and effected the age dating process. The results of this test, however, discount the possibility of the age of the water being contemporary or of recent age. The basis for arriving at this conclusion is a reference plane tied to the detonation of the hydrogen bomb.

GROUNDWATER WITHDRAWAL vs INCREASED DEPTH to STATIC DEEP GROUNDWATER LEVEL vs AGGREGATE RAINFALL

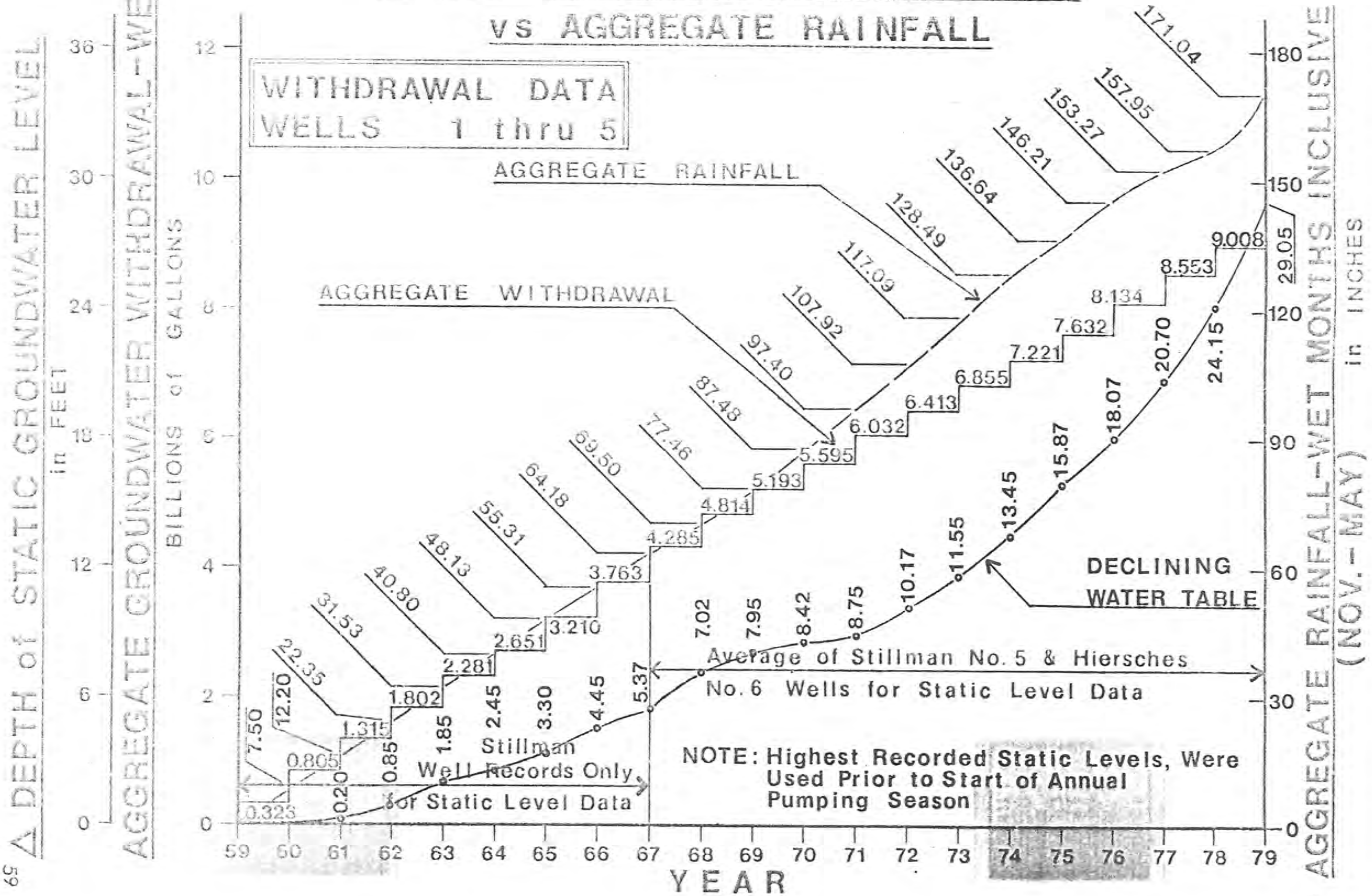


TABLE 17

HISTORIC PRODUCTION OF CITY DEEP WELLS

In Millions of Gallons

Year	Byers Ave. Well No. 1	Roundup Well No. 2	Tuituilla Well No. 3	Hospital Well No. 4	Stillman Well No. 5	Mission Well No. 7	Total
1978	35.04	114.93	92.04	141.40	72.03	83.4	539.2
1977	110.42	41.90	76.06	38.39	152.32	89.39	508.5
1976	2.04	34.09	58.23	111.66	219.88	103.96	605.5
1975	6.29	84.40	52.40	89.11	178.77	211.77	622.8
1974	80.28	- 0 -	36.35	75.86	173.66	87.85	454.1
1973	9.30	60.65	44.73	52.43	268.57	115.39	557.1
1972	64.93	0.44	42.57	82.56	191.32	113.78	495.6
1971	- 0 -	185.94	74.62	66.29	109.40	63.21	499.5
1970	142.74	8.87	63.24	23.94	163.73	64.78	467.3
1969	36.55	79.47	71.90	83.49	107.25	90.63	469.3
1968	201.37	76.74	58.22	115.16	77.42	136.56	665.5
1967	139.61	226.98	50.10	44.69	60.45	107.29	629.1

If this water would have been totally modern it would have a ^{14}C counting rate of 140% of the reference material of NBS Oxalic Acid. Although these waters are relatively young in relationship to other waters tested in the Columbia River basaltic flows, there is also a lack of historic data relating to earthquakes that may have taken place 2,500 years ago which may have induced either a shearing or subsidence of previously formed heavy gigantic lava blocks that were formed in the initial lava flows. Such a subsidence would cause a fault line where zones of shattered and broken rock form zones of low or zero permeability (in essence subsurface dams) entrapping and storing deep ground waters.

Upon review of the history of Wells No's. 1 through 5, as shown on the chart on page 59, the accelerated rate of decline first appeared in 1975 when average amounts of deep ground water were being withdrawn from the underground reservoir. The annual rainfall in the preceeding years prior to this acceleration in the dropping of the water table were years of average or better precipitation (see aggregate rainfall on chart on page 59). The annual aggregate rainfall shown on the chart on page 59 includes only precipitation from November 1 through May 31 of each year. The remainder of the precipitation is assumed to be lost to evaporation or transpiration by plant growth. It is also especially noteworthy that one of the largest rainfalls of record for this observed period occurred last year and the underlying reservoir experienced the largest recorded annual lowering of the water table. This supports the findings of the Carbon ^{14}C method that the deep ground waters are not modern and are not being annually replenished to any significant degree.

Another limiting factor on present well production capacity is that previous records state that the pump bowls for Wells No. 1 and 2 are presently at their lowest possible settings because of the well bore misalignments below these settings.

2. WELL NO. 6

The drilling of this well started in November, 1963 and the drilling and testing was completed in February of 1965. This well was drilled to a depth of 1,501 feet without developing a sufficient quantity of water to justify the installation of a well pump, motor and the construction of a well house.

Before deciding to abandon this well, several alternatives were tried to increase the yield of this well. These successive alternatives were tried:

a. The use of 100 gallons of an acid SC-200 was tried to dissolve clays that may be sealing off the aquifer that the bore hole penetrated.

b. Approximately 520 lbs. of dry ice was placed into the well which created a partial vacuum by the release of carbon dioxide gas. This resulted in the well discharging about 600-700 gallons of water. The purpose of this alternative was to create a surging effect and break down or enlarge the openings in the aquifers.

c. Compressed air was introduced into the well until the internal pressure had reached 80 psi. This lowered the water level in the well 185 feet. The air was then released and the cycle repreated nine more times. It was hoped that this surging action would increase the yield.

Presently, this Well is utilized only to monitor the water table elevation of the underground reservoir. Existing City wells vary considerably in their yield because of inconsistencies in which successive lava flows interbedded and overlapped the previous flow. Unfortunately, at this location the aquifers that were penetrated were not of sufficient porosity to develop a municipal well.

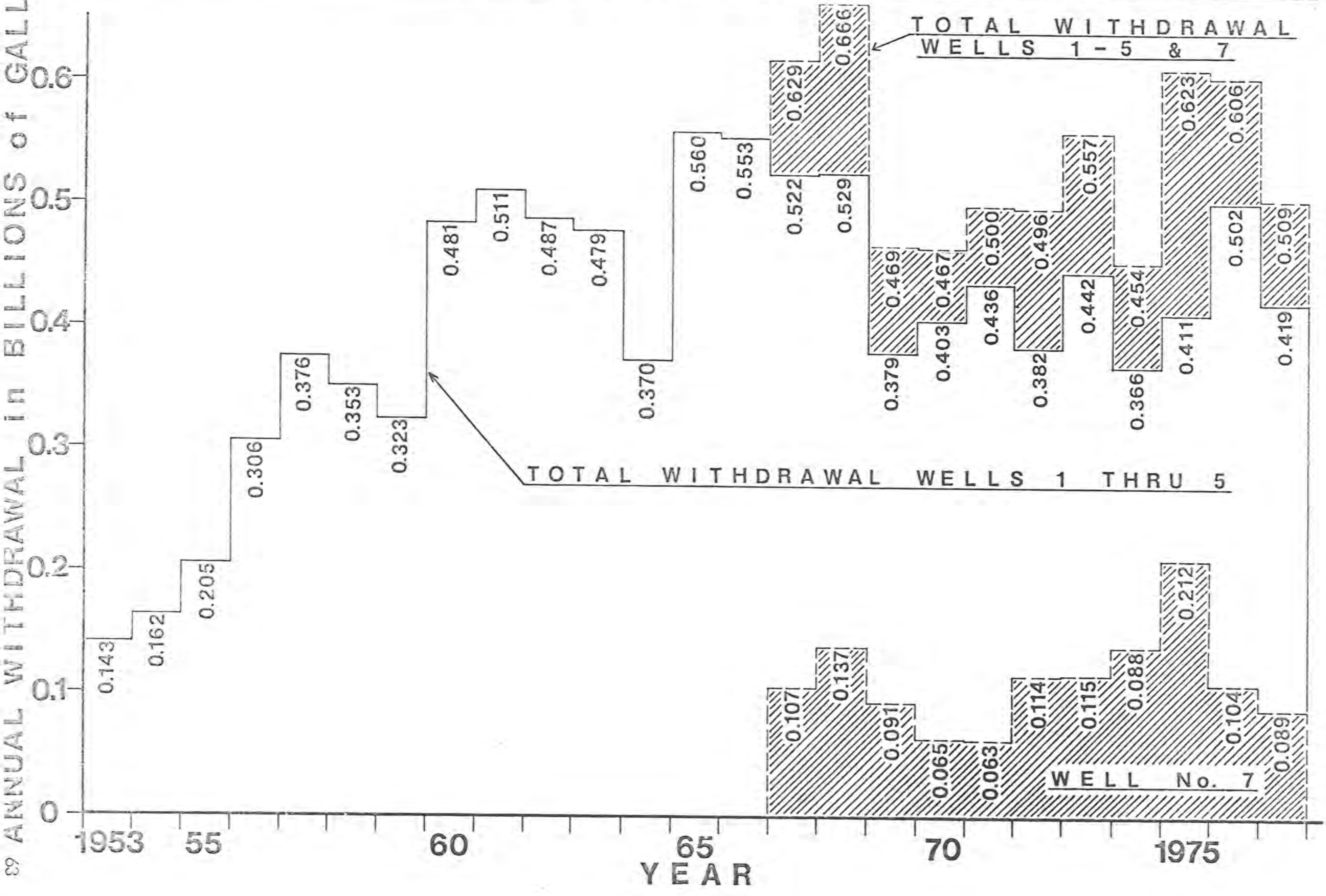
3. WELL NO. 7

The drilling of this Well was completed on April 27, 1966 and the pump and motor was installed and tested on January 20, 1967. The completed initial well consisted of a 50 horsepower motor and was cased with a 12" diameter casing to the 157 foot level, a 12" open rock hole to the 200 foot level, and an 8" open rock hole to the 452 foot level. The initial static water level was one foot below the surface. An eight inch column was installed to a depth of 185 feet, the bottom of the bowls and suction bell were set at the 190.7 foot level to which was attached a five foot length of six inch diameter suction pipe and one foot length of a six inch diameter conical strainer for a total depth of 196.7 feet.

The completed well tested at 835 gpm (1.2 MGD) initially and was designed to pump 800 gpm (1.15 MGD). The original well had a specific capacity (amount of water yield per foot of drawdown during pumping) of 5.71 gpm/ft when pumped at 1,000 gpm. The annual withdrawals of ground water from this well compared to City Wells No's. 1-5 is shown in Table 17 in millions of gallons for years 1967 through 1978 and graphically for years 1967 through 1977 on a chart on page 63.

By December, 1971, after five years of use, the water table had lowered approximately 29 feet and the yield of the well was reduced to 600 gpm. It was decided to deepen the well and enlarge the hole in an attempt to improve the yield from this well.

HISTORY of GROUNDWATER WITHDRAWAL FROM UNDERLYING BASALTIC AQUIFER and RESERVOIR



In November of 1971 a contract was let to bore out the well to a 12" diameter size to the 515 foot level and an 8" diameter hole to the 800 foot level. This work was completed in February of 1972. Based on tests at the completion of the well modification, the specific capacity of the well was 5.24 gpm/ft. This specific capacity was 8% less than the earlier test of April, 1966. The fact that the static level of the well remained unchanged during the re-drilling and deepening of the well coupled with the reduced specific capacity is sufficient documentation that no additional water producing aquifers were intercepted in the drilling process. The amount of reduced specific capacity can be attributed to the lowering of the water table, thereby reducing the pressure head to force the water through the water producing aquifers encountered in the initial drilling. As the water table continues to drop, this trend can be expected to continue. The deepening and enlarging of this well only effectively extended its useful life as a producing well. With a reported unstable layer encountered at the 430 foot level it has been recommended that the water level not exceed the 430 foot level during pumping.

By June of 1972 there had been installed in the reworked well a new 150 horsepower motor, 330 feet of 8" diameter column and shafting, 10 feet of pump bowl assembly, 10 feet of 8" diameter suction pipe, and a screen. The changes from the initial installation consisted of increasing the motor size by 100 horsepower and lowering of the suction intake at the bottom of the column from 195.7 to approximately 350 feet. This installation was designed to pump at the rate of 1400 gpm (2.02 MGD) and in future years after the water table had dropped, an additional 50 feet to be pumped at the rate of 1300 gpm (1.87 MGD).

Between June, 1972 and the fall of 1977 the water table of the well dropped an additional 26 feet with a yield production of approximately 850 gpm (1.22 MGD). During the summer of 1977 there was evidence that the water table had fallen below the suction pipe with the occasional pumping of air. In the summer of 1978 an additional 100 feet of column and shafting was added to the well. This places the bottom of the suction pipe 450 feet below the ground surface and 65 feet above the bottom of the 12" well bore hole reamed to the larger diameter in 1972. The current production from this well is approximately 800 gpm.

On June 22, 1972 the City directed a letter to the Oregon Water Resources Department that they may be receiving requests for large wells in the vicinity of City Well No. 7 (Mission), and requested notification when such applications were filed. The Department replied requesting the specific area of concern be identified by Section, Township and Range.

On July 5, 1977 the City drafted a letter to the Department identifying the specific area of concern on any new wells constructed in the vicinity of the Mission Well. On July 15, 1977 the City received a reply from the Water Resources Department advising the City that no applications for permits were pending in this area and to advise the Department when the City would no longer be interested in receiving this information.

On October 28, 1977 the City, by letter to William Porfily, the local Watermaster for District No. 5, requested that any permits for new well construction in the vicinity of Well No. 7 (Mission) be denied. On November 7, 1977 a letter drafted by William B. McCall of the Department's Salem office stated that there were no recent filings for ground water rights in the area of the Mission Well. Mr. McCall further advised the City that the Department under Oregon law cannot deny applications for the construction of wells and that there were certain exempt permitted uses that do not require the issuance of a permit. Mr. McCall did advise the City that they may file a protest from any future wells requiring a permit in the defined area of concern around the Mission Well and enclosed a copy of the applicable Oregon Revised Statutes. A letter dated November 8, 1977 from William Porfily reiterated the statements in Mr. McCall's letter and further stated that a well driller must declare the intent to and the intended use of any proposed well prior to drilling but such notices are confidential and the information cannot be released to anyone. Mr. Porfily said that the Salem office would advise the City when an application is received for a permit for the use of ground water in the vicinity of the Mission Well. Mr. Porfily also advised the City that an Indian wishing to drill a well on the Reservation may be exempt from State Statutes pertaining to water rights.

Uses of deepwater that are exempted by State Statute and do not require the issuance of a permit are:

a. Single or group domestic purpose in an amount not to exceed 15,000 gallons per day.

b. For watering any lawn or non-commercial garden not exceeding one-half acre in area.

c. For stockwatering purposes.

d. For single or commercial purpose in an amount not exceeding 5,000 gallons per day.

On March 10, 1978 an application for a permit (No. G8683) was filed by Jerry L. Haguewood in the Salem office of the Oregon Water Resources Department. The permit request was for 438 gpm (0.63 MGD) from a 230 foot deep 10 inch diameter well which was completed on March 3, 1978. The location of this well is approximately 2,000 feet southeasterly of the Mission Well and approximately 500 feet easterly of the Old Oregon Trail Highway (see map on page 57).

On May 1, 1978 a protest was filed by the City of Pendleton to the issuance of a permit for the Haguewood Well and a hearing on the protest was held on July 6, 1978 at the Umatilla County Courthouse in Pendleton. At the hearing oral testimony was given and exhibits submitted to support the City of Pendleton's protest. Additional supporting documentation was submitted after the hearing to document that a hydraulic connection existed between the Mission Well and the Haguewood Well.

On February 19, 1979 the Oregon Water Resources Department issued a Statement, Findings, Conclusions and Order in the favor of Jerry L. Haguewood. The findings acknowledged that a hydraulic connection existed between the Mission Well and the Haguewood Well but the finding based on the discretionary judgment of the Director of the Oregon Water Resources Department were:

1. That the granting of the Haguewood permit would not result in undue interference with the City's Mission Well; and

2. The rate of decline in the water table has not been determined to be excessive.

The above discretionary position taken by the Director of the Oregon Water Resources Department is inconsistent with a similar Findings, Conclusion, and Order issued April 2, 1976 for the Ordinance area. Deep well water tables in the Columbia River basalts in the Ordinance area were dropping an average of 5.0 to 7.0 feet per year in wells over 400 feet deep and dropping 1.6 to 2.0 feet per year for deep wells less than 400 feet deep when an attempt was made to curtail production for this area. The City of Pendleton Mission Well has experienced an annual lowering of the water table of 4.9 feet over the last 11 years. The City pumped 34,400,000 gallons in the spring of 1978 and with one month's pumping in June of the Haguewood Well, the Mission Well dropped an additional 15.83 feet (based on measurement 7/3/78) from the previous recorded depth of 54.90 feet on January 10, 1978. With no use of either the City or the Haguewood Well from July 3, 1978 to July 21, 1978 the water table had raised 11.72 feet.

As a part of this Study we also had the water from the Mission Well analyzed by the Carbon ^{14}C method for age. The result of this test indicated that the water is approximately 5,840 years old plus or minus 120 years. The results of this test are subject to the same factors that may affect the age dating analysis as discussed before (contact with limestone or mixing of modern and older waters). As stated for Well No. 4, the water in any case certainly is not totally modern because of the lack of presence of NBS Oxalic acid, a reference plane keyed to the detonation of the Hydrogen bomb.

Reviewing the historic data plotted on the chart on page 68 there is considerable consistency in the slope of the lines of the declining water table, the annual aggregate rainfall and the aggregate amount of water withdrawn from the underground reservoir from 1967 to 1973. The annual aggregate rainfall shown on this chart includes precipitation from November 1 to May 31 of each year. The remainder of the precipitation is assumed to be lost to evaporation or transpiration by plant growth. Nothing throughout this time period indicates, however, that there is a significant annual replenishment of the deep ground water.

From this chart for the time period of December 1974 to December 1977, we find contradictory data in that in years of decreasing rainfall there is a decreasing rate of lowering of the water table. On closer analysis, and discounting the concept of annual recharge, the chart shows for the prior year of 1974 that there was a very high withdrawal rate from the ground water reservoir followed by three years of greatly reduced withdrawal rates from the underground reservoir which would understandably result in a decreased rate of lowering of the water table elevation. Further, with last year being one of the highest rainfall years of record, the underlying water table continued to drop. The historic data on this chart supports the age dating analysis that this source is not subject to any significant annual recharge and is in reality a limited water resource.

It is the position of this author that there is sufficient basis for the City to file a petition for judicial review of the issuance of the Haguewood permit prior to April 19, 1978. To not defend the City's position on this issue at this time may well weaken the City's position at a later time and only after several billions of gallons have been withdrawn from this limited resource.

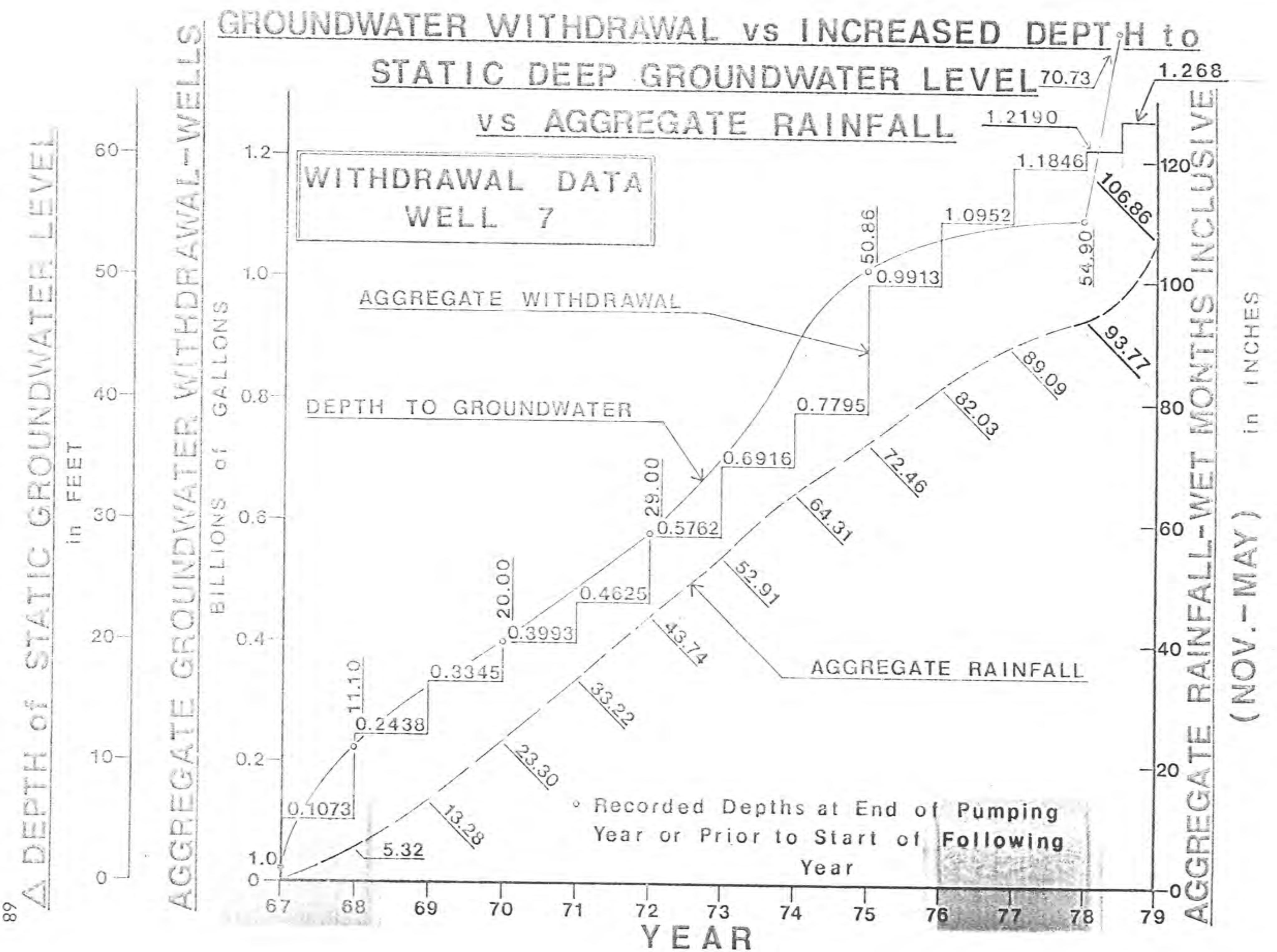
B. OTHER AREA WELLS.

An earlier study in 1962 investigated several other area wells which included the Biggs Well north of the City, the Crispin Well in Riverside, the Moore Well north of the Old Oregon Trail Highway approximately four miles east of the junction with U. S. Highway 11 to Walla Walla, the Poor Farm Well (Shadeview Mobile Home Park), the Lyons Well (east of U. S. Highway 395 on the approximate alignment of S. W. Quinney Avenue in Montee Addition), and the Union Pacific Railroad Wells at Rieth. All these were not deemed suitable for municipal wells because of either location, construction, or yield.

In another study in 1963 a well with a depth of 263 feet servicing the Glendale and Sunset Valley Tracts subdivisions was investigated and found to have the same approximate water table as the City wells.

1. BROGOITTI WELL.

In 1963 this well was determined to have approximately the same static elevation as the City Wells. This 12" well was drilled in 1957 to a depth of 399 feet and was cased to the 81.5 foot level. The well has a 10" liner from the 132 foot level to the 210 foot level. The static level was recorded at 200 feet. This well has the second highest yield in terms of specific capacity (gpm yield/ft of drawdown) of any of the wells which appear to be drawing from the common underground reservoir. From an interview with Jay and John Brogoitti on March 23, 1979 the following information was obtained:


GROUNDWATER WITHDRAWAL vs INCREASED DEPTH to
STATIC DEEP GROUNDWATER LEVEL
vs AGGREGATE RAINFALL
WITHDRAWAL DATA
WELL 7
AGGREGATE WITHDRAWAL
DEPTH TO GROUNDWATER
AGGREGATE RAINFALL
° Recorded Depths at End of Pumping Year or Prior to Start of Following Year
Δ DEPTH of STATIC GROUNDWATER LEVEL in FEET
AGGREGATE GROUNDWATER WITHDRAWAL-WELLS BILLIONS of GALLONS
AGGREGATE RAINFALL-WET MONTHS INCLUSIVE (NOV.-MAY) in INCHES
YEAR
67 68 69 70 71 72 73 74 75 76 77 78 79
0 10 20 30 40 50 60
0 0.2 0.4 0.6 0.8 1.0 1.2
0 20 40 60 80 100 120
0.1073
0.2438
0.3345
0.3993
0.4625
0.5762
0.6916
0.7795
0.9913
1.0952
1.1846
1.2190
1.268
1.0
11.10
20.00
29.00
50.86
54.90
70.73
5.32
13.28
23.30
33.22
43.74
52.91
64.31
72.46
82.03
89.09
93.77
106.86

1. That when the pump was first installed in 1957 their recollection was that the static water level was 190 feet below the surface.

2. That last year they had to add 60 feet of column to the existing 220 feet because of the lowering of the water table. On the bottom of the column there is approximately 10 feet of pump bowl assembly (10 stages-10" bowls).

3. That last year they installed 280 feet of air line to determine the water table level.

During the interview we did measure the elevation of the water table and found it to be at elevation 882. The surface elevation of 1109 feet for the Brogoitti Well is based on an altimeter reading. As they had not yet started any irrigation, this compares favorably with the City recorded static level elevation of 874 feet for Well No. 5 (Stillman) and an elevation of 877 feet for Well No. 6 (Hiersches).

Table 18 shows the specific capacity of all of the wells in the Pendleton area which appear to be withdrawing from the same underground reservoir.

TABLE 18

WELL	Well Depth in Feet	Test Pumping Rate GPM	Specific Capacity GPM/Ft Drawdown
New Hospital Well	500	945	189
Brogoitti Well	399	720	144
City Well No. 1 (Byers Avenue)	945	1155	96.25
City Well No. 2 (Round-Up)	761	1670	92.78
City Well No. 5 (Stillman)	700	2390	28.11
City Well No. 3 (S.W.21st St.)	1008	420	6.08
City Well No. 4 (Hospital)	1050	810	4.55
Glendale - Sunset	600	170	3.40
Sewage Plant Well	334	68.75	2.86
Well No. 6 (Hiersches)	1501	525	1.45
Smith Canning Well	665	?	?

As can be seen from the above table, the productivity of each well is entirely dependent on the porosity of the acquifers encountered at each well location.

2. SEWAGE PLANT WELL.

After an exhaustive review of the files, we were only able to come up with the following information. A review of the daily logs indicate that the static level of the well remained constant at the 98.5 foot level. The well was bail tested at the 294 foot level by withdrawing 55 bails at 50 gallons each in 40 minutes time with a drawdown of 24 feet. Based on this data the specific capacity of this well would be 2.86 gpm/ft of drawdown. If this information is correct this well would yield less than any of the producing wells listed in Table 18. The driller did not note at what level the water bearing aquifer was intercepted and in fact had erroneously noted the static water level depth before this depth was encountered. A memorandum placed in the file dated January 19, 1966 states that an induced flow by use of compressed air indicated an available yield of 600 to 700 gpm and that a 350 gpm test pump would not faze the static water level. This contradicts the information obtained from the well logs.

The well logs further indicate a well depth of 334 feet with 200 feet of 8" diameter casing placed in the hole. In the log the driller states he bailed the well dry. This would indicate that this length was sufficient to seal off the encountered aquifer. In reviewing the progress payments for this well, however, there is conflicting data which indicates that the well is 349 feet deep and that 334 feet of 8" diameter casing was installed.

Based on existing records, we can see no basis for justifying the projecting of this well as a major production well without a new test to determine the well's specific capacity. This well should be re-tested as soon as possible to determine its capabilities.

3. NEW STATE HOSPITAL WELL.

On January 29, 1976 the Eastern Oregon Hospital and Training Center completed the initial construction of a new well for irrigation purposes. Shortly after drilling the well was reworked with some additional reaming and the finished hole construction consisted of a 12" diameter cased hole to the 37 foot level, an open 12" diameter rock hole to the 375 foot level and an open 6" diameter rock hole to the 500 foot level. The first water bearing aquifer was encountered at the 185 foot level and additional water bearing aquifers were noted on the log at the 308-327 foot level and 348-365 foot level.

The well was subsequently tested for a 24 hour period at 945 gpm with drawdown of 5 feet. The specific capacity of this well is 189 gpm/ft of drawdown. Based on this test, the productivity of this well far exceeds any other in the Pendleton area. The present pump installation consists of 220 feet of 10" diameter column and 10 feet of bowl assembly. The rated output of the pump is 1100 gpm (1.584 MGD).

Discussion on the inclusion of this well into the City's well field system by the City staff and the Hospital staff has already been discussed on a preliminary basis. The reconstruction of this well to increase the capacity of this well to its full potential should be a high priority objective. The ability of a single well of this capability for meeting summer peaking demand conditions could be invaluable as a supplemental access to the underground reservoir.

4. MILTENBERGER WELL.

Based on an interview with Julian Miltenberger and a review of the log, the following information was obtained. That Mr. Rudd Davis in June of 1969 drilled for him a 6" diameter well to a depth of 548 feet and cased to the 92 foot level. At the 96 foot level the first water bearing aquifer was intercepted and today you can hear the water from this level cascading down to the lower aquifer. At a depth of 150 feet an opening was encountered in the strata of approximately 18" to 24" and released the flow of the upper aquifer into this aquifer. The static level in the well was recorded on the log at 196 feet below a ground level of approximately 1138 feet (based on an altimeter reading) for a water table elevation of 942 feet. The yield shown on the log was approximately 42 gpm.

Approximately six months ago the well was tested at 60 gpm with a pump setting at the 300 foot level with 300 feet of airline. An observed pressure was noted as being approximately 28 psi before and during the test period. Based on this information, the water table elevation would be 895 feet. On April 4, 1979 a measurement was made to the static level and found to be 254.5 feet below the surface. Based on this observation the elevation of the water table would be 883.5 feet. This compares reasonably well with an observed water table elevation of 882.0 at the Well No. 5 (Stillman) on April 3, 1979

Looking down the well bore, the alignment veers off to the southwest and the water surface could not be observed. The amount and extent of being off-plumb could not be determined.

Observations of the well should continue to determine if there is a hydraulic interconnection between this well and the City wells. This well in only 1400 feet from City Well No. 5 and its development may result in substantial interference, reducing the specific capacity of both wells. The well at this location would not be able to effectively supply water to the Southwest area where the majority of the future growth is anticipated to occur

5. CITY WELL PERMITS FOR UNDERDEVELOPED AND UNDEVELOPED WELLS.

On July 16 1962 the City of Pendleton filed for permits for the construction of two new wells and the reconstruction of an existing well plus supplemental water rights for Wells No's. 1, 2, and 3. The supplemental filings were 405 gpm (0.90 cfs) for Well No. 1 (Byers Avenue); 1,395 gpm (3.1 cfs) for Well No. 2 (Round-Up); and 90 gpm (0.20 cfs) for Well No. 3 (S. W. 21st Street).

On October 10, 1962 the City filed for a permit for water rights on the South Hill Well for 3,015 gpm (6.7 cfs); the McCormack Well for 3,015 gpm (6.7 cfs); and the Crispin Well for 3,015 gpm (6.7 cfs). The location of these wells are shown on the map on page 72. From the map on page 72 it is apparent that the South Hill Well permit area is occupied by a commercial development at the S. E. 3rd Street interchange. On March 22, 1966, the City filed for a permit for the Sewage Plant Well for 3.015 gpm (6.7 cfs). This well was discussed in the preceding pages of this report.

6. SUMMARY OF WELLS.

A Summary of the wells in the immediate Pendleton area and the Mission Well are shown on Table 19 on page72 in a descending order based on their productivity. In reviewing this table it is important to recognize that wells can be relatively close together, be poorly hyraulically interconnected and have a substantial difference in static water levels.

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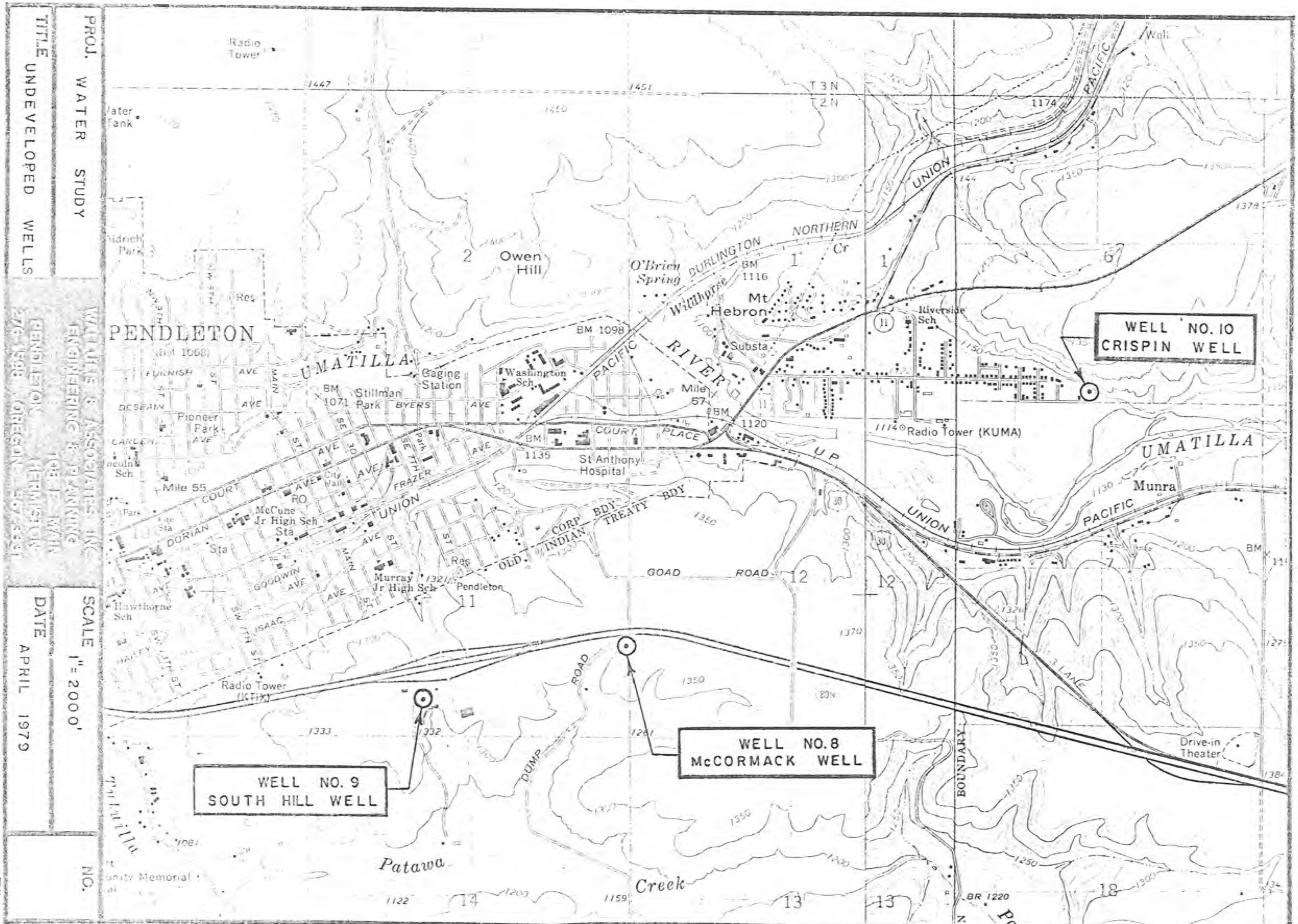

TABLE 19

GENERAL WELL INFORMATION

Well Name - Location	Present Pumping Capability GPM	Specific Capacity GPM per ft of Drawdown	Ground Elevation at Well	Elevations of water bearing acquifers Elevations in feet above sea level
New Hospital Well	1100	189	1021 *	713/694; & 673/656
Brogoitti Well	750	144	1109 *	849/843; 827/814; & 747/710
City No. 1 (Byers)	1800	96.25	1095	915/883; 635/632; 362/358; 301/284; & 189/173
City No. 2 (Round-Up)	2100	92.78	1054	unknown
City No. 5 (Stillman)	2300	28.07	1072	unknown
City No. 3(S. W. 21st St.)	750	6.08	1070	unknown
City No. 7 (Mission)	800	5.24	1463.1	1139; & 1116/1112
City No. 4 (Hospital)	800	4.55	1050	689/671
Glendale - Sunset		3.40		
Sewage Plant	-0-	2.86	1005.4	Yield tested when well was at 711.4 elevation
City No. 6	-0-	1.45	1068 *	826 and 762
Smith Canning Well	-0-	?	1045	867' (well yield was 900 gpm)

* Determined by Altimeter

The Salem, Oregon area has been identified as a part of the same Columbia River basaltic flow as our area as shown on the map on page 75. Salem, Oregon was one of the areas where artificial recharge was performed on an experimental basis. During several pumping and recharge cycles it was documented that two wells spaced 483 feet apart were hydraulically interconnected but one of the wells consistently maintained an elevation of 15 to 25 feet higher than the other one.

C. WELLS - GENERAL.

There is a general concern for the declining water table taking place in the Columbia River basalts in Oregon from Arlington to the foothills south and east of Pendleton. The Oregon Water Resources Department received special funding from the 1977 State Legislature to study this general area. Presently this study is being focused in Umatilla County Stage Gulch area with Hinkle being the westerly boundary, the Columbia River the northerly boundary, and the Rew Elevator the southeasterly boundary.

The study will also focus on the Butter Creek area. The earlier attempt to curtail withdrawals of deep ground waters from this area has been postponed by the courts on the basis that the hydrologic boundaries had not been adequately defined. Additional data is presently being collected to more accurately define the outer perimeter of the Butter Creek Basin. Unfortunately, our immediate area is not presently scheduled for an intensive ground water investigation.

1. PENDLETON UNDERGROUND RESERVOIR.

The ground water withdrawal information presented in this report includes only information from City records. Other area wells in the vicinity of Pendleton may also be withdrawing unknown quantities of water from the same underground reservoir for irrigation, domestic, and possibly industrial uses. It is at this writing impossible to determine the outer hydrologic boundaries of the underground reservoir below the City and how many others are affecting the depletion of this valuable resource.

With the historic decline of all of the City's wells, the City should seriously consider the consequences of initiating proceedings to have a state study made with the prospect of having this area declared a critical ground water area or the alternative of continuing to observe the depletion of the reservoir underlying the City. If the area would be declared a critical ground water area then an order would be made requiring:

(a) A moratorium on further well construction from within the established boundaries of the defined area, and/or curtailment of withdrawal amounts from wells with the possibility of shutting down some of the wells with the later priorities.

(b) Require accurate metering of all withdrawals from the deep ground waters.

(c) An accurate ongoing monitoring program of the underlying water table.

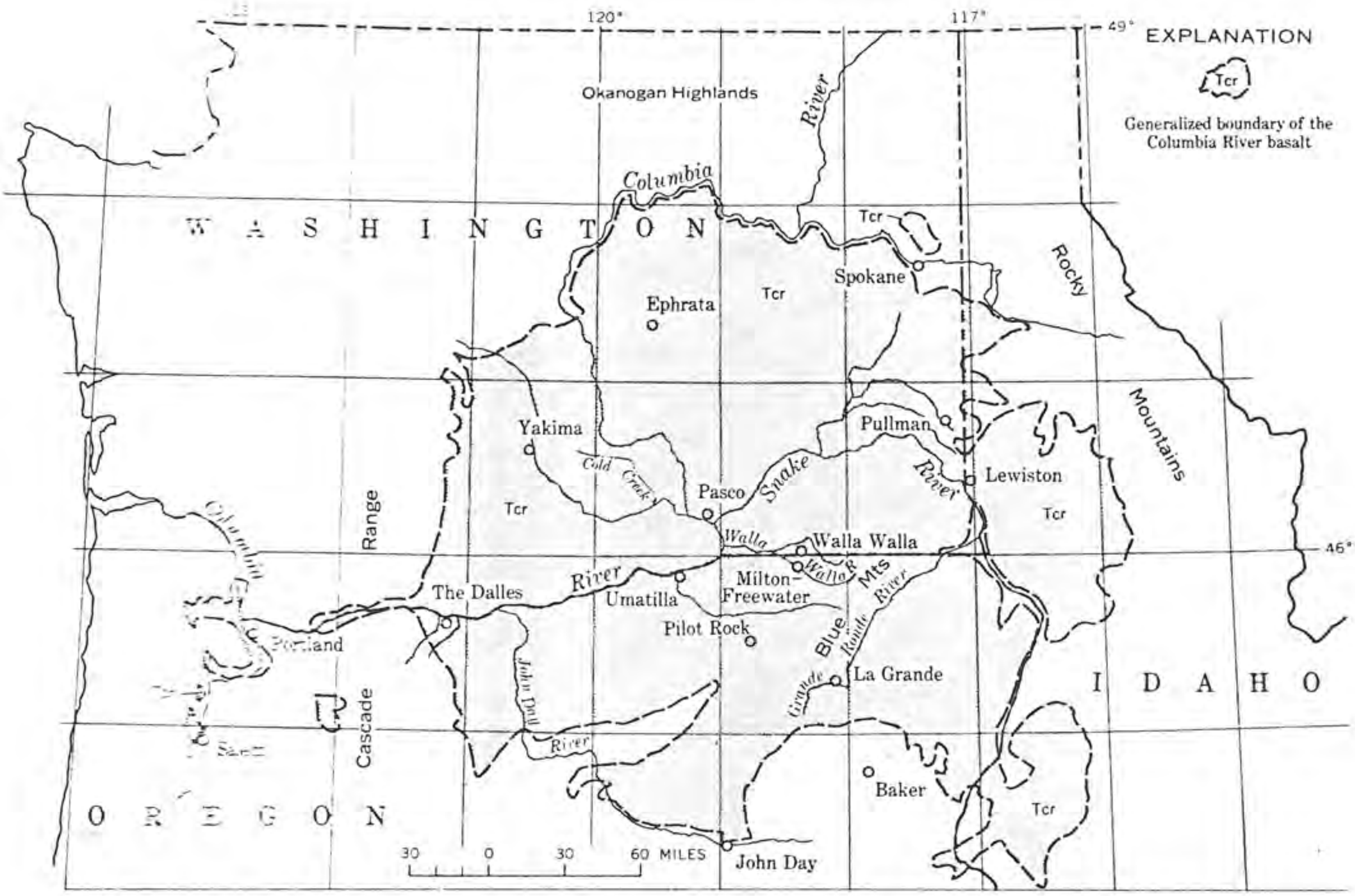

FIGURE 1.—Map showing the main area underlain by the Columbia River basalt, Washington, Oregon, and Idaho.

Reproduced from:

"Hydrology of Volcanic-Rock Terranes", a Geological Survey Professional Paper 383-A by R. C. Newcomb. U. S. Government Printing Office, Washington: 1961.

Both (b) and (c) are presently required but because of lack of manpower they are not normally enforced.

The present study for Stage Gulch is anticipated to take two years to collect data and present the findings at a public hearing and then if the findings so justify, issue the Order declaring the area a critical groundwater area. A similar period of time may be required for a study of the Pendleton area.

2. AREA GEOLOGY.

Several geological studies and papers have been prepared on the geology of the Columbia River basin. A separate detailed section of the area geology is presented in the Appendix of this report.

a. Columbia River Basaltic Lava Flows.

The Columbia River basaltic lava flows encompass a broad area over three states covering approximately 50,000 square miles as shown on the map on page 75. The depth of the lava flows varies considerably and is known to be in the central part of the flow as much as 5,000 feet thick. The total thickness of the basalt is the result of a series of individual sequential flows of lava ranging from 5 to 150 feet thick. Individual lava flows have been identified for distances up to ten miles and in our immediate area the total combined thickness ranges from zero to more than 2,500 feet. In practically all of the Umatilla River basin which covers some 2,700 square miles, the basalt either underlies the surface at a shallow depth or crops out above the surface.

The top of each lava flow cooled more rapidly than the lower portion of each individual flow. This resulted in the entrapping of gasses in the upper portion of each individual flow, leaving behind honey combed layers of rock which along with vertical cooling joints (contraction of rock) provide the means for ground water to travel over great distances. Unfortunately, successive lava flows often completely "melted" this "honey combed" layer, interrupting the ability of ground water to follow predictable routes.

Subsequent to the issuing of lava over this large land area, the earth's surface underwent areas where subsidence and uplifting occurred. Deep layers of lava rock have a considerable bending property and to a significant degree react to opposite forces similar to more pliable materials. When the bending stresses of lava rock were exceeded both the vertical and horizontal shearing of the rock occurred. Along these shearing lines the forces were of such magnitude that the rock interfaces were ground into a homogenious mass of fine lava and coarse rubble which resulted in the decomposition of lava along the shear lines into impermeable zones (faults) or subsurface dams.

The identification of a particular subsurface water bearing (hydrologic) boundary requires significantly more extensive geologic study than is currently available for the immediate study area. This report, however, adequately documents the fact that the amount of withdrawal from all deep wells in the immediate area has historically exceeded the annual amount of natural recharge.

b. Local Geological Sub-Areas.

The map shown on page 78 shows the principal features of the local geology. The basis for this map is based on the supplemental information given in the Appendix of this report and is based on the best current available geological information. In geological terms: anticlines are upward folds where subsurface "hills" have been formed; synclines are downward folds where "dips" have been formed; monoclines are areas where the lava has sloped from the level to a downward slope; slopes are areas between a monocline and a substantially reduced slope area; and plains are areas that are near level adjacent to synclines, anticlines, and monoclines. The letter "U" on the map refers to areas where uplift occurred and where the letter "D" occurs, refers to areas where subsidence occurred or fault zones. From a review of this map it appears that the City of Pendleton is situated at or near the low point of a "hydrologic bowl". That is to say that natural recharge in our immediate area should accumulate somewhere near the general vicinity of the Southgate interchange. These hydrologic boundaries also indicate that artificial recharge of the ground waters should be successful since the area is bounded by anticlines (subsurface hills) preventing the escapement of artificial recharged water.

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PROJ. PENDLETON WATER STUDY	WADLUIS & ASSOCIATES INC. ENGINEERING & PLANNING 109 E. MAIN PENDLETON HERMISTON 276-1588 OREGON 567-3331	SCALE 1 : 250,000	NO.
TITLE GEOLOGICAL SUB AREAS		DATE APRIL 1979	

3. ARTIFICIAL RECHARGE OF BASALTIC ACQUIFERS.

Recharge of ground waters is currently being investigated on a national scale as a means of collecting and storing of winter surplus runoff waters. In the West end of the County a recent program of recharging of the shallow ground waters was initiated and is proving to be a successful project. This section of the report, however, will be limited to the history of artificial ground water recharging of the Columbia River basalts.

a. Walla Walla, Washington, Experiment.

In 1956-57 the Ground Water Branch of the United States Geological Survey conducted an experimental recharge program at Walla Walla. Surplus water from Mill Creek was utilized to recharge the underlying basaltic aquifers. During this experimental period 23 million gallons of Mill Creek water was injected into the deep ground water of Well No. 3. The injection rate varied from 630 to 670 gpm. The Mill Creek water supply was analyzed prior to injection and found to be of excellent chemical and bacteriological quality with sediment ranging from 2 to 6 mg/L (ppm). (1 mg/L or 1 ppm=8.34 lbs of silt per million gallons of water) The general compatibility of the surface water and the deep ground waters was verified prior to injection by blending the waters and checking for the formation of precipitates.

As a result of this experiment the subsequent yield and specific capacity of Well No. 3 was reduced (35% reduction). The decrease in yield was attributed to:

i) The fine particulates in the Mill Creek surface water with sediment ranging from 2 to 6 mg/L (ppm).

ii) Dissolved air in the colder surface water being released when coming into contact with the warm lava and mixing with the warmer deep ground water causing air binding in the basalt aquifer.

iii) The releasing of entrained or entrapped air that originated from leaks, bends, fittings, and valves. A continuous blast of air was evident during the recharging through the measuring port to determine the level of the water table.

The conductance (ability to transmit electrical current) was considerably higher in deep ground waters which have been in intimate contact with mineralized rocks (basalts) than that of the surface water source. Based on the specific conductance differential in the two waters it was possible to document that recovery of the injected surface waters could be accomplished. After the injection test period withdrawals were made from Well No. 3 for one and one-half months.

At the start of the pumping cycle the water withdrawn had the specific conductance of the Mill Creek surface water and with a gradual blending of the deep ground waters taking place, the characteristic of the surface supply shifted to that of the deep ground water.

This report suggested that with refinements on the injection process that continuing the recharge program would be worthwhile. City staff with an impaired well chose not to pursue the experimental program.

b. Artificial Recharge of Basalts at The Dalles, Oregon.

In 1960-61 the United States Geological Survey with the permission of the City of The Dalles conducted a second experimental recharge of the deep ground waters. The Dalles surface water supply, which also happens to be named Mill Creek, from this source injected a total of 81.4 million gallons over a period of six months into the underlying basaltic aquifers. The Mill Creek recharge water was filtered, chlorinated, and flouridated prior to injection at an average rate of 1500 gpm. The surface water supply and the deep ground waters were checked for their compatability prior to injection and found to be compatible. The sediment concentrations of the injected water ranged from 0-3 mg/L (ppm) throughout the experimental period.

The injected surface water supply ranged from 13° to 25° Fahrenheit cooler than the deep ground water. As the injected water was cooler than the deep ground water, stratification occurred (cooler water being more dense settled to the lower parts of the underlying aquifer). The colder surface water also has a greater viscosity (less fluid-like) than the warmer deep ground waters and this resulted in the underground aquifers being less permeable in both accepting the cooler surface waters and during the pumping or withdrawal cycles. With this higher viscosity of the cooler surface water the specific capacity (yield) of the well during the withdrawal of injected waters was diminished until the native ground waters with a higher temperature moved in. In the range of temperatures (40° - 62° F) encountered in this recharge experiment, the effect of lowering the water temperature 1° F was computed to effectively reduce the specific capacity (yield) of the well by 1.8%.

During this experiment at The Dalles, the injection of surface waters was stopped whenever the water table raised rapidly indicating a plugging of the aquifers. The well would then be pumped and surged to restore the specific capacity of the well. The sediment concentration of the water withdrawn at the end of injection periods ranged from 0 to 18 mg/L (ppm) except for one occasion when coagulating chemicals (alum floc) at the treatment plant passed through the filters and was injected into the deep well. Withdrawal of the injected surface waters after this accident had a sediment content of 618 mg/L (ppm) or 5,154 lbs/million gallons of water. The well after 7½ hours of surge pumping still had a sediment content of 4 mg/L (ppm).

To minimize the problems of air entrainment evidenced at Walla Walla by permitting the water to freely fall into the pump column, the water at The Dalles was injected under 50 to 70 pounds per square inch of pressure. During recharge, however, crackling noises could be heard at several points along the pipeline from the well back to the system. As water is forced backwards through the pump bowl assembly (the impellers) there is great resistance to the backward flow of the water and this was presumed to be the major cause for the air to have come out of solution.

In this experiment the surface water and the deep well ground water was analyzed for dissolved oxygen content. The Mill Creek surface water ranged between 11-14 mg/L (ppm) and the deep well ground water was 1.2 mg/L (ppm). During pumping cycles following injection periods the withdrawn water was positively identified as predominantly surface water by its temperature and conductivity. Analysis of the withdrawn water based on several tests revealed, however, that the typical proportionate blend of surface water and well water which should have had a dissolved air content of 9.0 mg/L (ppm) had a dissolved oxygen content of 0.3 mg/L. Four possible explanations given for the removal of dissolved oxygen were:

i) That the dissolved oxygen in solution chemically reacted (oxidation) with minerals in the basaltic rock (iron), or

ii) That the dissolved oxygen could have come out of solution as air bubbles in the aquifer, or

iii) That the air could have come out of solution during the surface water injection process or during the pumping cycle, or

iv) That with the high water velocities during the injection cycle the air could have come out of solution at several locations such as meters, bends, valves, other fittings.

The clogging of the underground aquifer (with the exception of the alum incident) was attributed chiefly to the release of air in the form of air bubbles during the injection process. The higher levels of sediment content which were noted at the start of the pumping cycles were probably also a contributing factor.

The well selected for surface water injection had high specific capacity of 100 gpm/ft. of drawdown. This indicated that the underground aquifer was highly porous and would readily accept the injection of surface water. Tests later documented this fact as the cooler (denser) surface water occupied the lower portion of the acquifer and readily escaped from the immediate area. This was verified by the rapid decrease in the level of the mound of water created by the injection process and the inability to recapture only small portions of this water during pumping cycles. The geologists were not concerned at the inability to reclaim substantial amounts of the recharge water as the geologic and hydrologic conditions precluded the escape of the injected water from the immediate ground water basin. At the conclusion of the experiment the specific capacity of the well was 91 gpm/ft of drawdown versus a specific capacity of 100 gpm/ft prior to the experiment. The geologists were not concerned about this short term reduction in the specific capacity and recommended that the project be continued to evaluate the recharge experiment over a longer term.

Based on a discussion with C. Dean Smith, former City Manager of The Dalles, the City decided not to continue the experiment primarily because of complaints from private individuals, water districts and industries who were aware of the accidental injection of the alum floc incident. The complainants, concerned about contaminants being introduced into the deep ground waters, threatened to sue the City if they continued the artificial recharge experiment.

c. Artificial Recharge of Basalts at Salem, Oregon.

In 1962 the U. S. Geological Survey, with the consent of the Salem Heights Water District, conducted their third experimental artificial recharge of the underlying basalts. During this three month experiment in 1962 a total of 24.5 million gallons of winter surplus water (low cost - 50% summer rate) was purchased from the City of Salem and injected into one of the District's deep wells. The water was injected under pressure at an average rate of 830 gpm.

The City of Salem's water supply was obtained from infiltration galleries on Stayton Island in the North Santiam River. The only treatment this water received consisted of chlorination and flouridation. The purchased recharge water contained 11.7 mg/L (ppm) of dissolved oxygen and during each injection the sediment in the water greatly exceeded the anticipated 0.3 mg/L (ppm). During the first injection cycle, the injected surface water had a recorded high sediment content of 40 mg/L (ppm). On the second injection the sediment in the water had a high of 216 mg/L (ppm) and on the third injection cycle a high of 119 mg/L (ppm). The high sediment counts were attributed to sand in the water. The well's specific capacity at the completion of the experiment was considerably improved prior to the experiment. The specific capacity prior to the experiment was 14.3 gpm/ft of drawdown and at the conclusion of the experiment by surging it was improved to 17.1 gpm/ft of drawdown. This reflects a 20% increase in the productivity of this well.

Tests of the deep ground water also revealed a high dissolved oxygen content that ranged from 6.8 to 9.3 mg/L (ppm) which was attributed to water cascading down from upper water bearing aquifers above the static level of the well. The surface and well waters were analyzed prior to the experimental recharge and found to be compatible.

The temperature of the surface water supply and the temperature of the ground water were close to each other. Therefore, a reduction in the specific capacity of the well because of temperature which was encountered at The Dalles would not be a material factor in the Salem experiment.

To partially overcome the release of air from the water related to high water velocities through piping, valves and fittings, long radius bends were installed. Although a complete modification of the piping would have been desirable the cost on an experimental basis couldn't be justified. In contrast to the experiment at The Dalles, where practically all the air in solution in the water disappeared, the water withdrawn after each injection typically contained over 80% of the amount of the dissolved air during the injection cycle. With the incorporation of these piping changes it was believed that the release of air out of the water was greatly minimized and that clogging attributable to air was minor in this experiment. The high levels of sediment in the water was determined to be the chief cause of clogging of the aquifer.

Even with the excessive amounts of sediment in the recharge water, the pumping and surging cycles required only 3% of the total volume injected for this purpose. The net volume of water added to the underground reservoir was 97% of the total amount injected.

Similar to The Dalles, the local geologic and hydrologic conditions in the immediate area at the Salem Heights area precluded the escape of substantial volumes of recharge water from the general area. The selected well site was situated approximately 3,400 feet easterly of a local depressed basin in the underlying basalts. Near the center of this depressed basin the Water District owned another well which would be the primary beneficiary from the artificial recharge experiment.

In subsequent years the area served by the Salem Heights Water District was annexed to the City of Salem. Spurred by the low precipitation in the winter of 1976 the City decided to duplicate the 1962 experiment of artificially recharging the underlying basalts. The artificial recharge experiment conducted by the City of Salem started on March 1, 1977 and terminated on October 17, 1977. During this experiment the earlier test well and the well near the center of the depressed basin in the underlying basalts were both utilized as injection wells.

During this second artificial recharge of the underlying basalts, a total of 150,441,000 gallons of surface water were injected into the basalts. Of this amount 1,658,700 gallons of water was used for surging and cleaning of the underground aquifer. The net volume added to the underground aquifer was 148,782,300 gallons or 98.9% of the total.

Based on monitoring of subsequent withdrawals, 95,577,600 gallons (64.24%) of the surface water was recovered and the water table was 10 feet higher at the end of the withdrawal period, some of which may be attributed to natural recharge. The rest of the findings in the second experiment generally reaffirmed the findings of the first experiment.

With the impact of the drought diminished by a subsequent year of adequate rainfall, this artificial recharge of the deep basalts was suspended. This experiment will probably not be repeated until an urgent need again becomes evident. A continuation of the recharge experiment would also benefit other private wells within the boundaries of the confined aquifer without their bearing any of the financial burden.

D. SUMMARY AND FINDINGS.

i) Prior experiments have adequately demonstrated that artificial recharging of the deep basalts is technically feasible over the short periods of time. Long term compatability of mixing of surface and deep waters has not been documented as of this date.

ii) That a specially designed artificial recharge facility would overcome several of the problems encountered in previous recharge experiments.

iii) The underground basaltic aquifers have tremendous storage capabilities for water that can be injected during low demand periods.

iv) There is a need for state legislation to assure equitable financial participation from all individuals who would benefit from an artificial recharge project.

v) That the implementation of a recharge project would require an ongoing supervisory, monitoring and maintenance program to maintain the capacity of the well and the quality of the underground deep water.

vi) That in prior experiments large degrees of sediment were dislodged from existing pipelines because of reversal of historical flow patterns and the high water velocities in pipes during the injection process. On permanent installations the use of pressure filters at the surface would partially or completely eliminate this problem.

vii) That some means other than injection of the surface water through the pump impellers in a reverse direction be utilized for permanent installations. Hydraulic wear over an extended period of time could materially reduce the useful life of the impellers. Also, in the pump bowl assembly, there is a substantial restriction in the cross sectional area for the passing of the injected water which results in the formation of air bubbles.

viii) That the higher specific capacity wells would be the best wells for an artificial recharge program. The higher specific capacity wells penetrating the more porous aquifers are capable of higher rates of injection with less risk of clogging.

ix) That a small diameter monitoring well be constructed near the injection well. The purpose of the monitoring well would be to verify that changes occurring in the water table level were also occurring in the general area. A large rise in the water table in the injection well without a proportional rise in the monitoring well would indicate clogging taking place in the injection well and the need for surging and cleaning.

CHAPTER V

WATER DEMANDS VERSUS AVAILABLE SUPPLY

A. WATER DEMANDS

There are three types of demands that must be considered in the design of any water system to adequately meet present and future needs of the City. These three types of demands are annual, irrigation and fire and will be analyzed individually as to their impact on the Pendleton water system.

The water demands for future projected population shall be premised on historic consumption records from 1960 through 1978. The population throughout this time period varied from 13,197 to 15,000 and should adequately cover high and low consumption years.

A study of the charts developed later in this Chapter demonstrates that a large per-centage of the annual consumption takes place during the summer months. Temperature, construction activity, and local economics all affect the water consumption in the summer months. The water consumption during the summer months dramatically affects the average annual per capita consumption and water department revenues.

1. Average Annual Demand

The average annual demand incorporates all demands encountered during a typical year and provides a basis for determining the aggregate water needs of the City. To arrive at a per-capita average demand we analyzed the historic consumption from 1960 to the present. For each year we used the total consumption including system losses (but excluding cannery uses) and this information is shown in Table 20 on page 87. From this table we were able to establish an historic range of 272 to 329 gallons/person/day.

TABLE 20

PER CAPITA WATER CONSUMPTION

Total in System
(Less Canneries)

Year	Less Canneries	Population	Yearly Per Capita Use	Yearly Average Gal/Cap/Day
1960	1,708,202,752	14,434	118,346	324
1961	1,561,291,984	14,434	108,168	296
1962	1,465,458,476	14,557	100,670	276
1963	1,454,897,824	14,253	102,077	280
1964	1,402,710,120	14,146	99,159	272
1965	1,467,677,960	14,100	104,091	285
1966	1,435,353,084	14,145	101,474	278
1967	1,518,024,492	14,300	106,156	291
1968	1,570,957,824	14,600	107,600	295
1969	1,477,058,444	13,740	107,501	295
1970	1,585,884,636	13,197	120,170	329
1971	1,393,078,808	13,450	103,575	284
1972	1,485,471,516	13,600	109,226	299
1973	1,588,257,000	13,982	111,447	305
1974	1,540,550,500	14,010	109,961	301
1975	1,478,385,600	14,186	104,214	286
1976	1,459,900,000	14,302	102,077	280
1977	1,456,767,000	14,600	99,779	273
1978	1,558,351,000	15,000	103,890	284

PER CAPITA WATER CONSUMPTION

History	Year	Consumption Year	Consumption Day
High Year	1970	120,170 gallons	329 gallons
Low Year	1964	99,159 gallons	272 gallons
Average Year	1960-1978	106,294 gallons	291 gallons

If artificial recharge of the underground basalts with their tremendous storage capabilities proved to be successful only long term average annual per - capita consumption requirements would need to be considered in the determination of the City's future annual needs. The underlying aquifers could serve as a large equalzing reservoir as long as wells with sufficient capacity were able to withdraw adequate quantities to meet peak demands.

Over the last 19 years 16.7% of the total water used was unmetered. The least amount of unmetered water was 8.4% in 1961 and the highest was 27.8% in 1974. For the most recent year it was 22%. Each year during the winter months water turned in the springs is often in excess of actual needs and the surplus overflows unmetered out at the South Hill Reservoir. This probably accounts for the majority of the unaccounted for water. A good tight water system with accurate meters and records of unmetered uses should be able to account for at least 95% of the total consumption. It would seem likely that at least a 10% reduction in the average annual per - capita requirements could be projected or that the present meters are inaccurately recording the actual consumption. We are of the opinion that with the increased cost of treating the water that there will be an additional 10% decrease in the water consumption. A 20% reduction in the consumption requirements for populations of 15,000, 20,000, and 25,000 persons would reduce the total annual demands by 318.6, 424.8, and 531.0 million gallons respectively.

TABLE 21

PROJECTED ANNUAL SYSTEM DEMANDS

In Millions of Gallons

Design Population	High Year	Low Year	Average Year
15,000	1441.0	1191.3	1274.5*
20,000	1921.3	1588.4	1699.3
25,000	2401.7	1985.5	2124.2

*291 gallons per person per day x 15,000 x 365 x 80%.

We anticipate that during the first year when water rates will have to be significantly increased there will be a material drop in usage. The consumption in following years should pick up after the public becomes accustomed to the higher rates.

The Confederated Tribes of the Umatilla Indian Reservation is presently conducting an independent water study for their needs. Not knowing what this study may recommend we shall assume for the balance of this study that the Confederated Tribes will continue to obtain their water from the City of Pendleton Supply System. We are assigning the production from Well No. 7 (Mission) to meet this particular need.

2. Irrigation Demands.

Depending on the weather, the consumption of water increases dramatically when yard irrigation starts. The length of the yard irrigation period varies in both length and intensity considerably from year to year. Average daily consumption for each month of the year for the years of 1967 through 1978 are shown graphically on pages 92 through 103 inclusive. For the years that the canneries were using City water we have shown the previous impacts the canneries placed on the supply system. The gravity and the well sources of water to meet these various monthly demands are shown separately on these graphs. Unfortunately, during the irrigation season the supply from the springs are normally at their lowest yields. In recent years a substantial amount of water was supplied by Well No. 7 (Mission) because the spring supply was too turbid (dirty) for domestic consumption. With recent EPA mandated turbidity requirements the usage of the spring sources can be expected to supply even less of the total water demand until a treatment plant can be constructed.

A review of the consumption graph for 1976 demonstrated that a variation of 413% can be anticipated for the average day during the maximum month versus the average day during the minimum month in any one year. To provide conventional storage facilities to meet high demands of a month duration or longer as experienced in July of 1976 would be financially prohibitive. To maintain a reasonably low cost and adequate supply capability it is essential that the reliability of the underground basaltic reservoir be preserved. With the preservation of the integrity of the underground reservoir and adequate production wells providing access to it, the irrigation demands can be easily satisfied from the underlying basaltic acquifers.

3. Irrigation Demand Versus Temperature.

We compared the climatological records and the summer per capita consumption to study the effect of temperature on irrigation usage. The consumption data shown on the graph on page 90 for the years 1960-1978 excludes cannery use. The consumption data shown does, however, include all unaccounted for water in addition to the metered consumption. Population data used to calculate the average per capita use during the maximum demand months for each year were obtained from the City records.

Temperature records from June 1 through September 1 were selected as

BRUNING 40-22 33381

PROJ. PENDLETON WATER STUDY

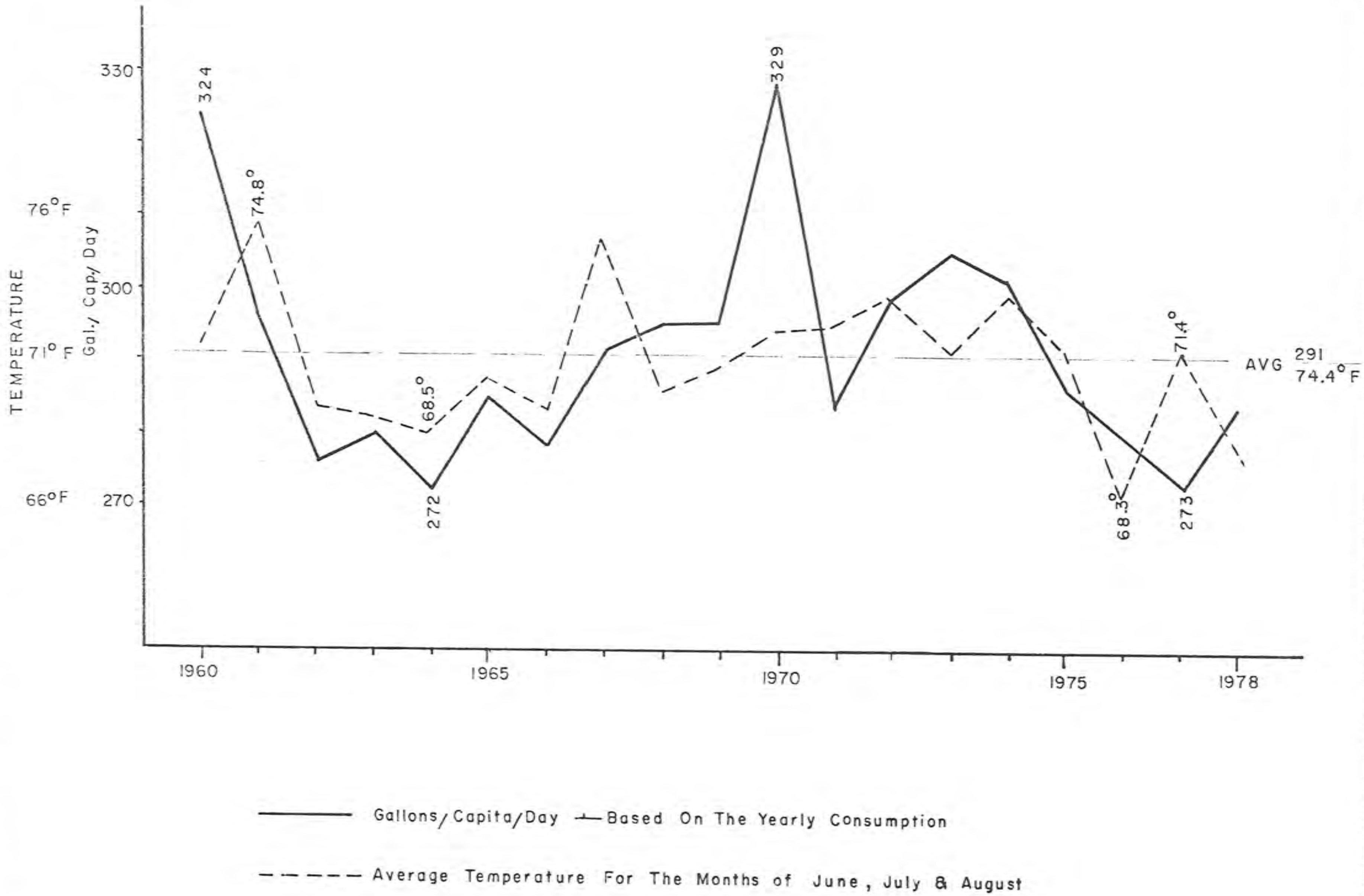
TITLE CONSUMPTION vs. TEMP.

ENGINEERING & PLANNING
108 E. MAIN
PENDLETON HERMISTON
OREGON

SCALE AS SHOWN

DATE APRIL 1979

NO.



typical summer months. This time period was felt to be most representative and for all years includes the month of the highest yard and garden irrigation requirements. A close correlation would show a direct relationship between temperature and the summer irrigation demands.

As anticipated, a correlation did exist in almost every year, but there were some exceptions. In the year 1960 and again in the year 1970, unusually high consumption was experienced for average summer temperatures. During each of these years the reason for the disparity could not be found. In the year 1977 an unusually low consumption was experienced for an average temperature year. This can probably be explained by the drought during the winter of 1976 and the public awareness of the water shortage during the following summer in 1977.

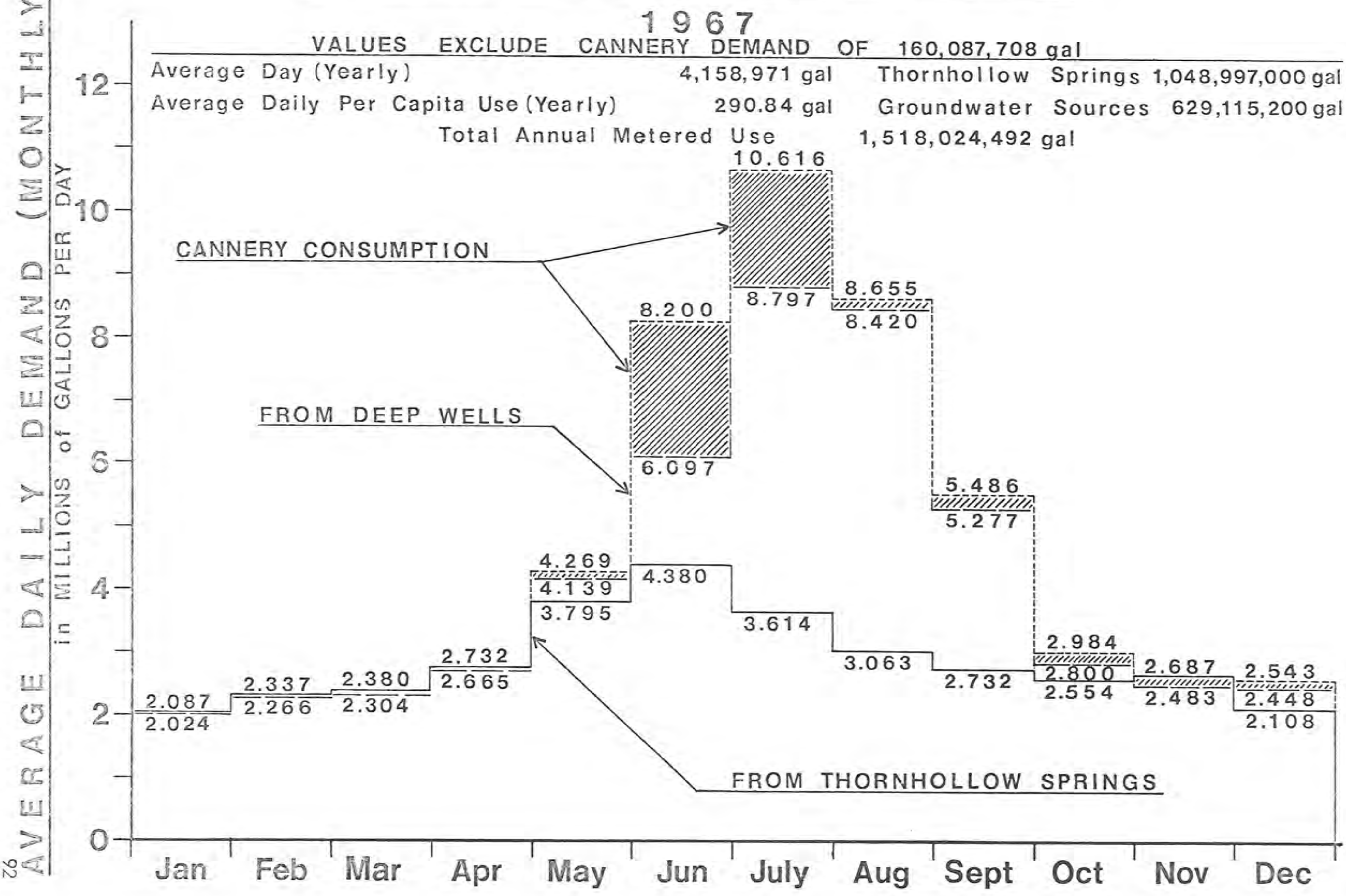
Some of the factors that are responsible for variations in annual usage are as follows:

1. Summer yard irrigation requirements are to a large measure temperature dependent and vary considerably from year to year.

2. During periods of population decline the requirements for irrigation of established yards remains virtually unchanged.

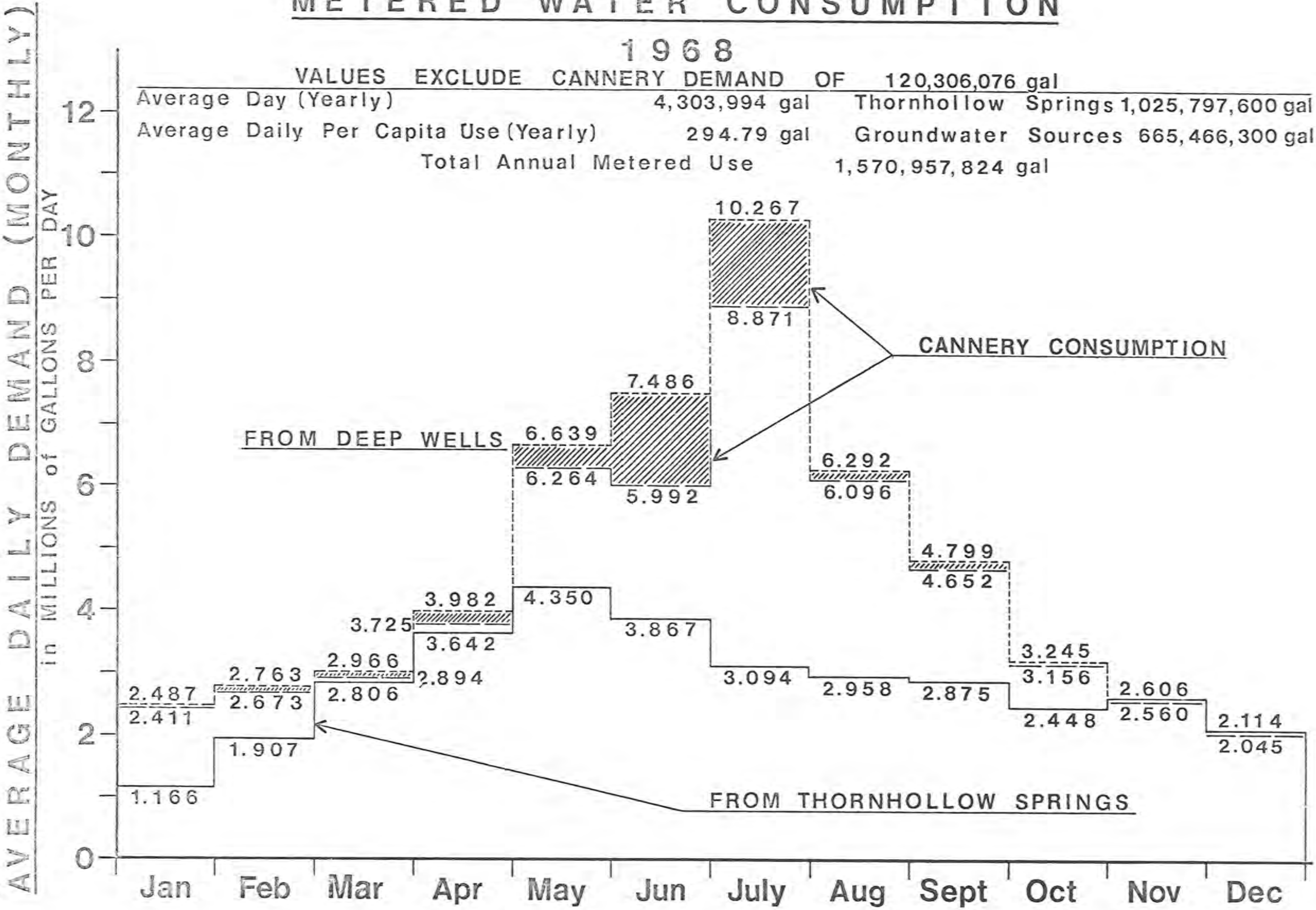
3. That higher than normal use of water occurs during growth periods when new yards are being installed and for control of dust during the period of construction activities.

METERED WATER CONSUMPTION

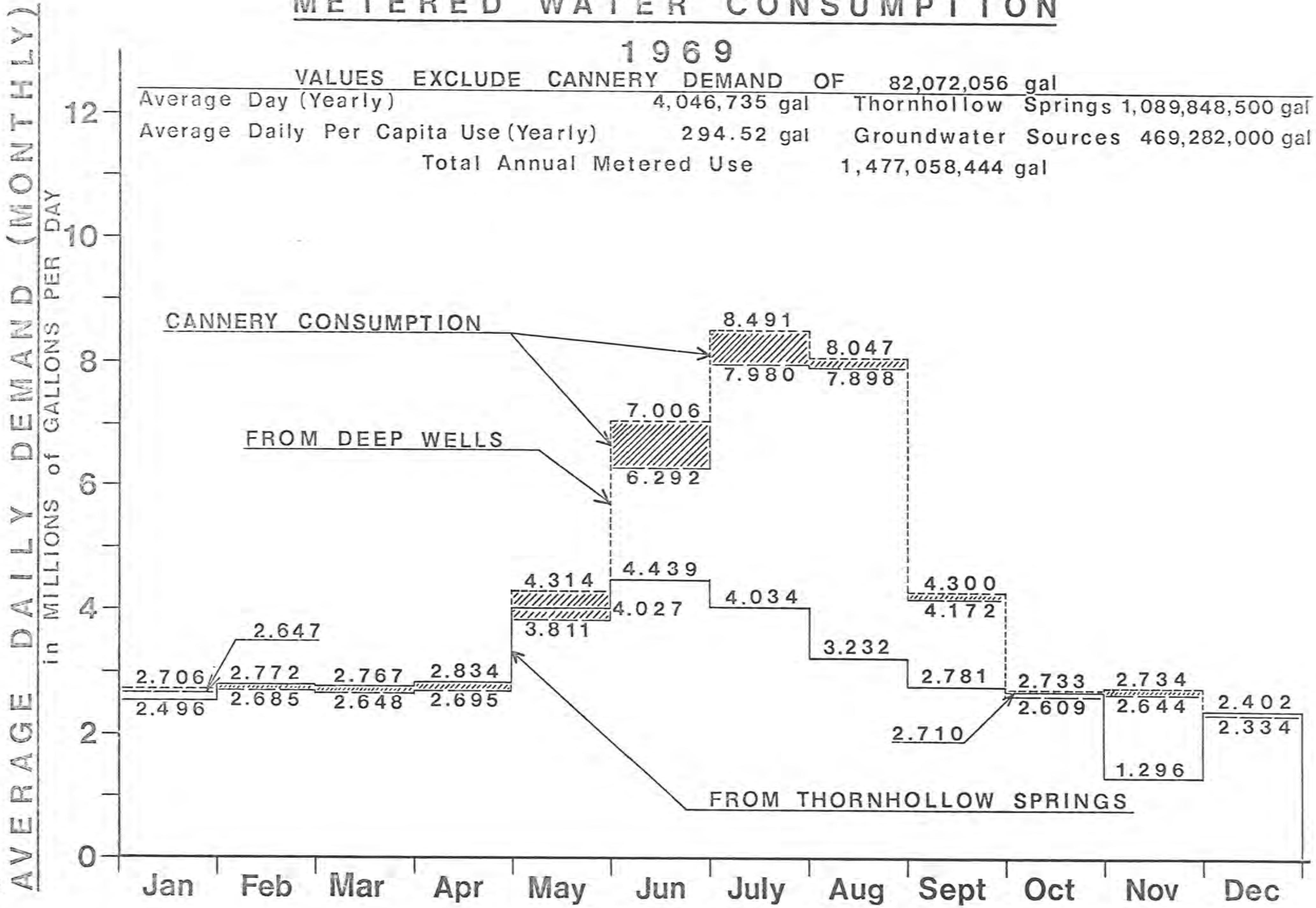


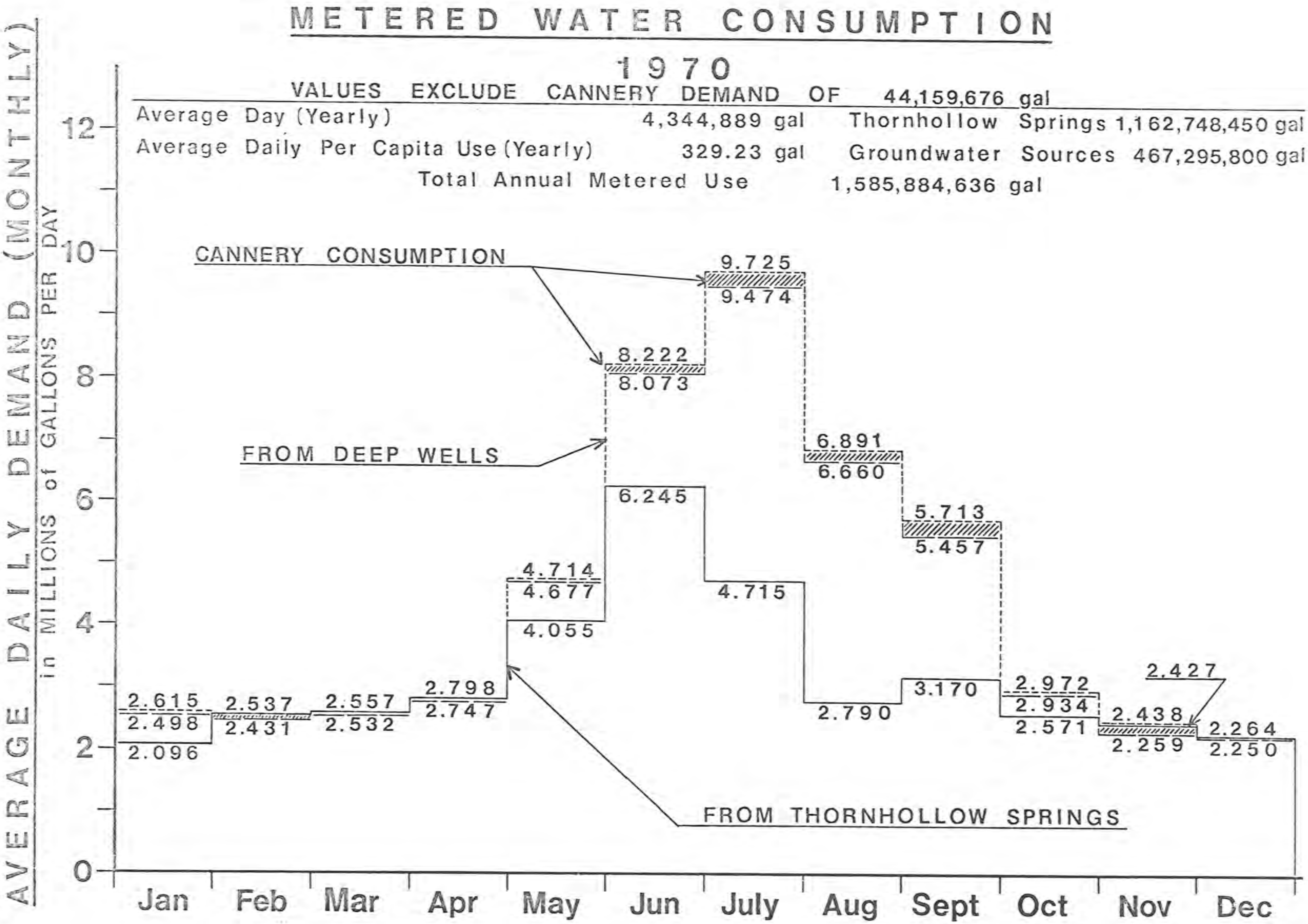
METERED WATER CONSUMPTION

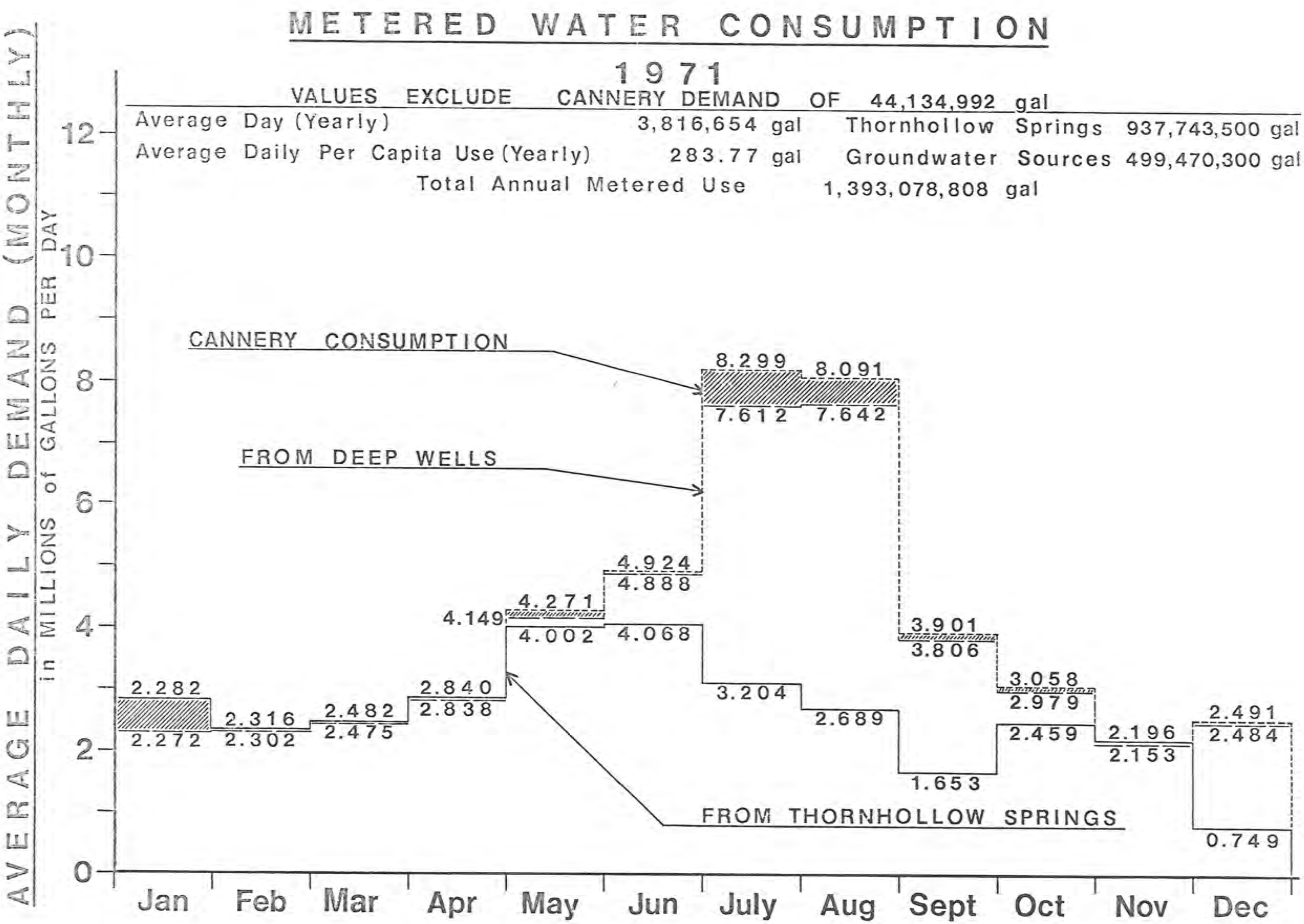
1968



METERED WATER CONSUMPTION

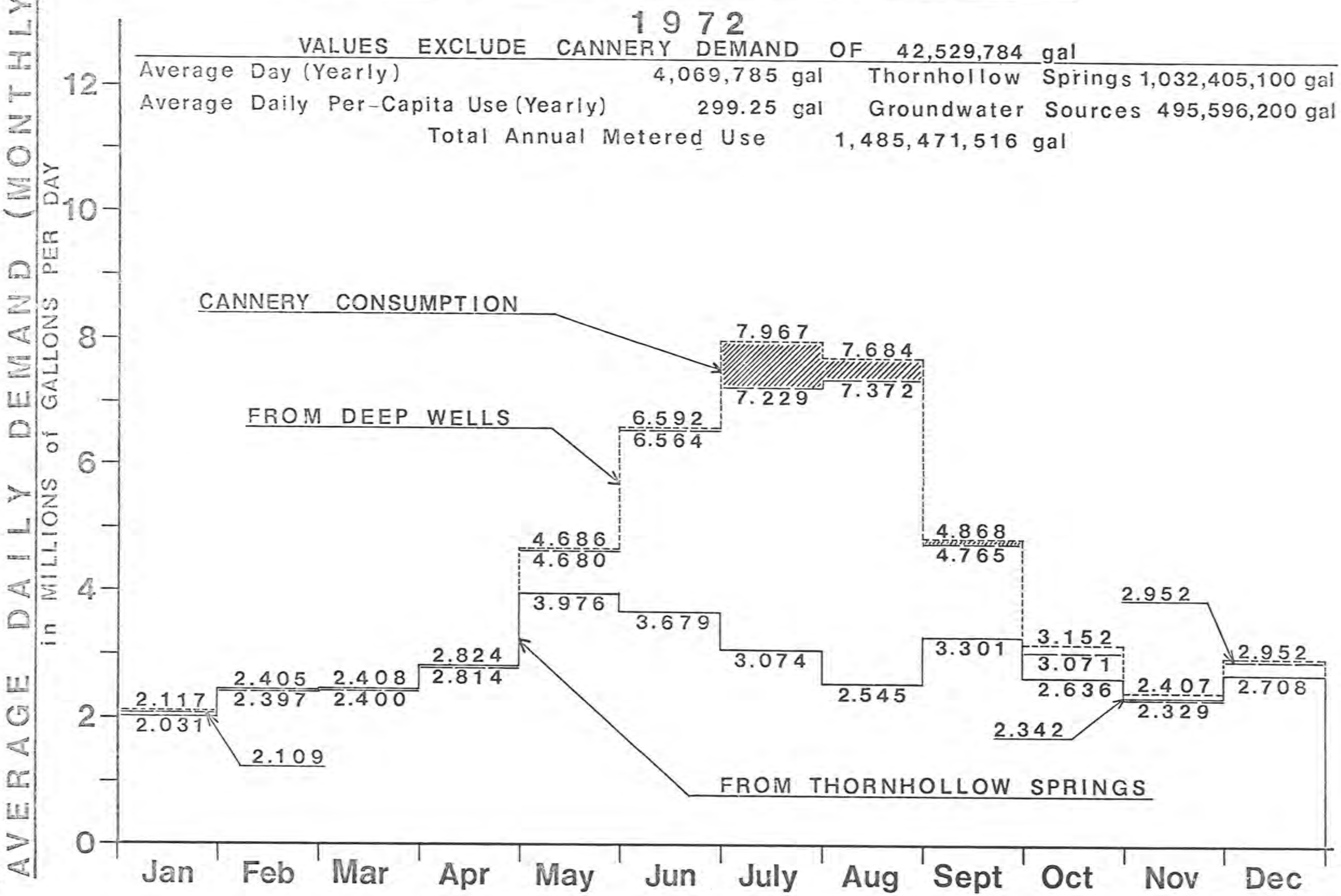



METERED WATER CONSUMPTION
1970
VALUES EXCLUDE CANNERY DEMAND OF 44,159,676 gal
Average Day (Yearly) 4,344,889 gal
Average Daily Per Capita Use (Yearly) 329.23 gal
Total Annual Metered Use 1,585,884,636 gal
Thornhollow Springs 1,162,748,450 gal
Groundwater Sources 467,295,800 gal
AVERAGE DAILY DEMAND (MONTHLY)
in MILLIONS of GALLONS PER DAY
0
2
4
6
8
10
12
CANNERY CONSUMPTION
FROM DEEP WELLS
FROM THORNHOLLOW SPRINGS
2.615
2.498
2.096
2.537
2.431
2.557
2.532
2.798
2.747
4.714
4.677
4.055
8.222
8.073
6.245
9.725
9.474
4.715
6.891
6.660
2.790
5.713
5.457
3.170
2.972
2.934
2.571
2.427
2.438
2.259
2.264
2.250
Jan
Feb
Mar
Apr
May
Jun
July
Aug
Sept
Oct
Nov
Dec

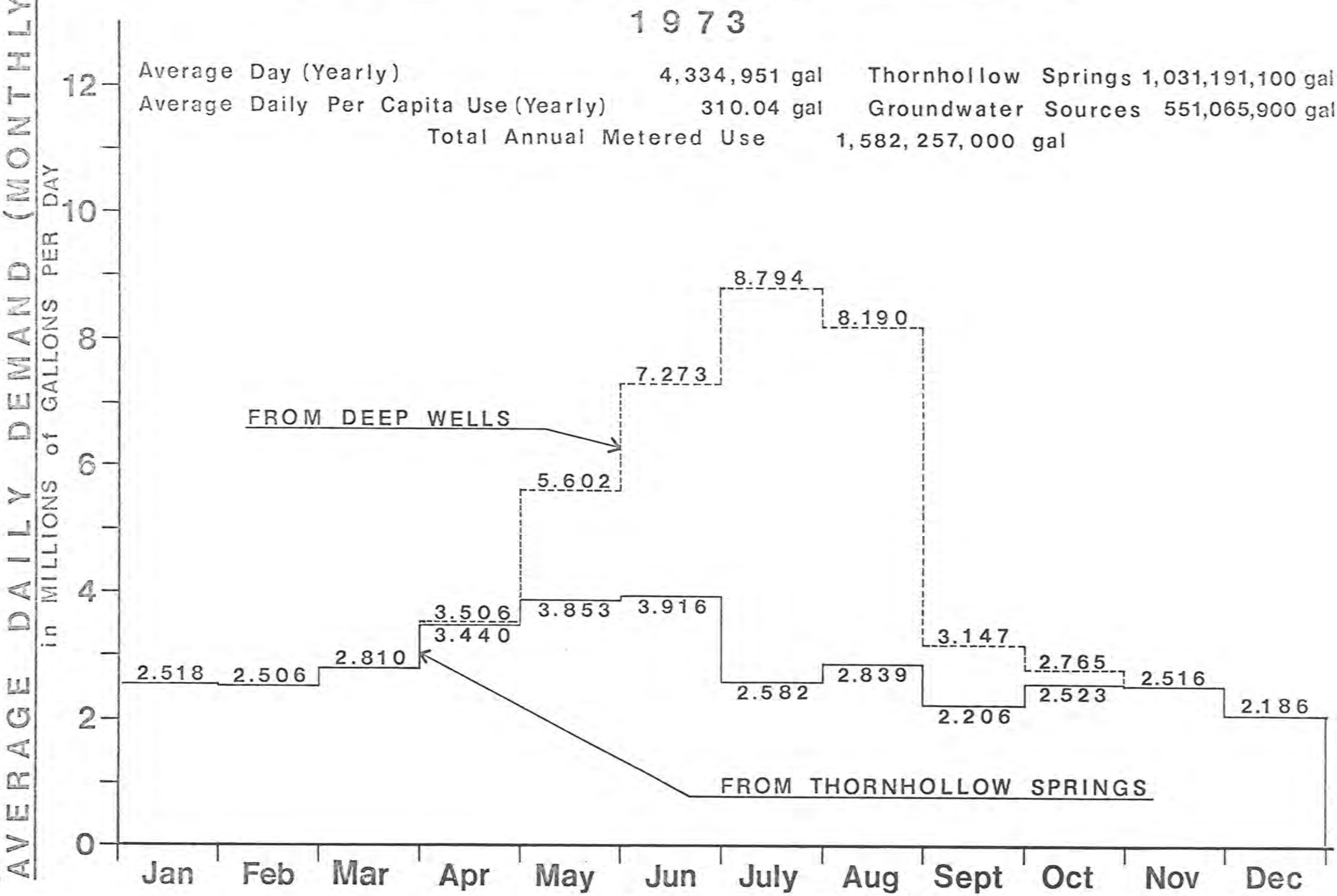

METERED WATER CONSUMPTION
1971
VALUES EXCLUDE CANNERY DEMAND OF 44,134,992 gal
Average Day (Yearly) 3,816,654 gal
Average Daily Per Capita Use (Yearly) 283.77 gal
Thornhollow Springs 937,743,500 gal
Groundwater Sources 499,470,300 gal
Total Annual Metered Use 1,393,078,808 gal
AVERAGE DAILY DEMAND (MONTHLY)
in MILLIONS of GALLONS PER DAY
0
2
4
6
8
10
12
CANNERY CONSUMPTION
FROM DEEP WELLS
FROM THORNHOLLOW SPRINGS
2.282
2.272
2.316
2.302
2.482
2.475
2.840
2.838
4.149
4.271
4.002
4.924
4.888
4.068
8.299
7.612
3.204
8.091
7.642
2.689
3.901
3.806
1.653
3.058
2.979
2.459
2.196
2.153
2.491
2.484
0.749
Jan
Feb
Mar
Apr
May
Jun
July
Aug
Sept
Oct
Nov
Dec

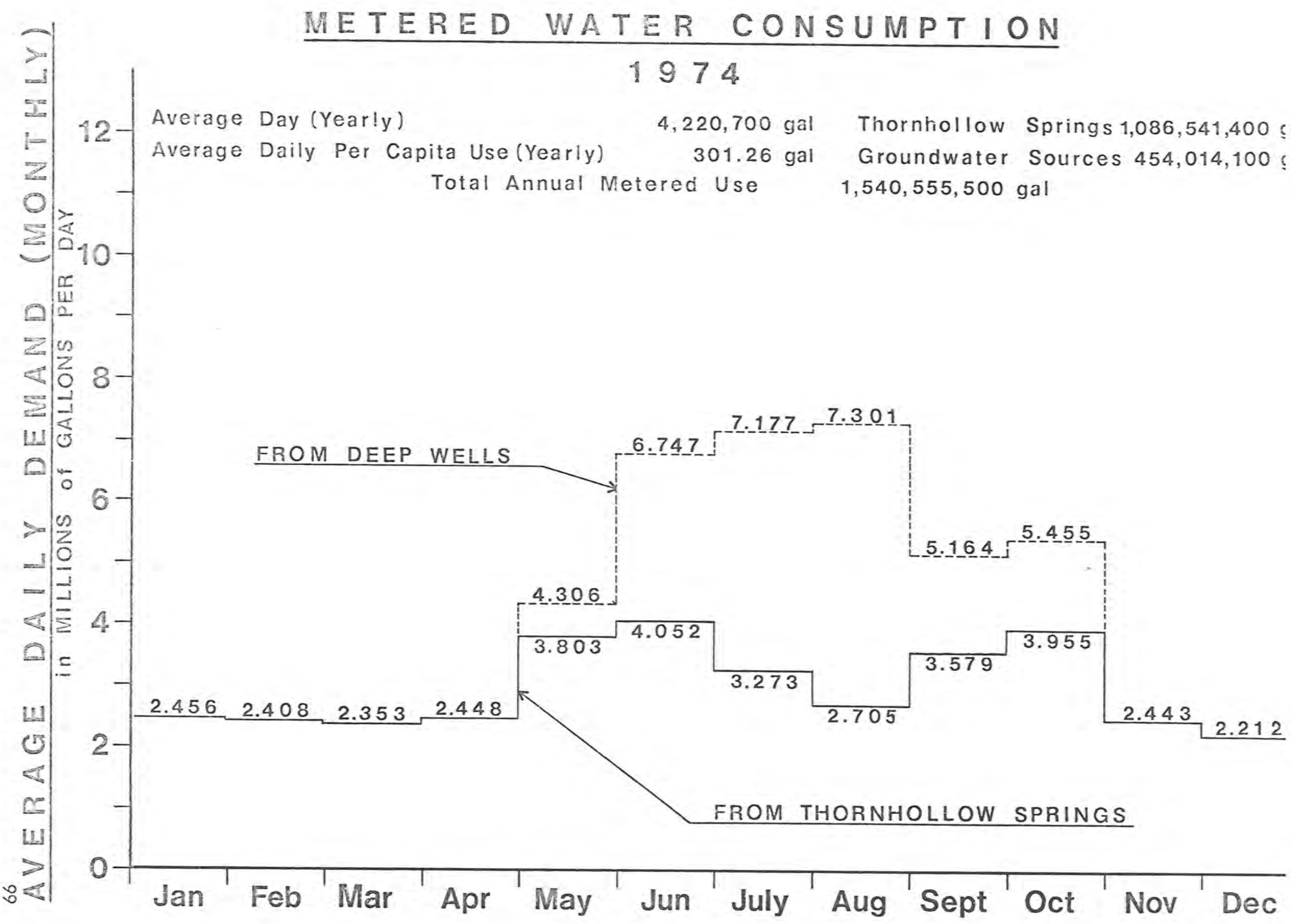
METERED WATER CONSUMPTION

1972



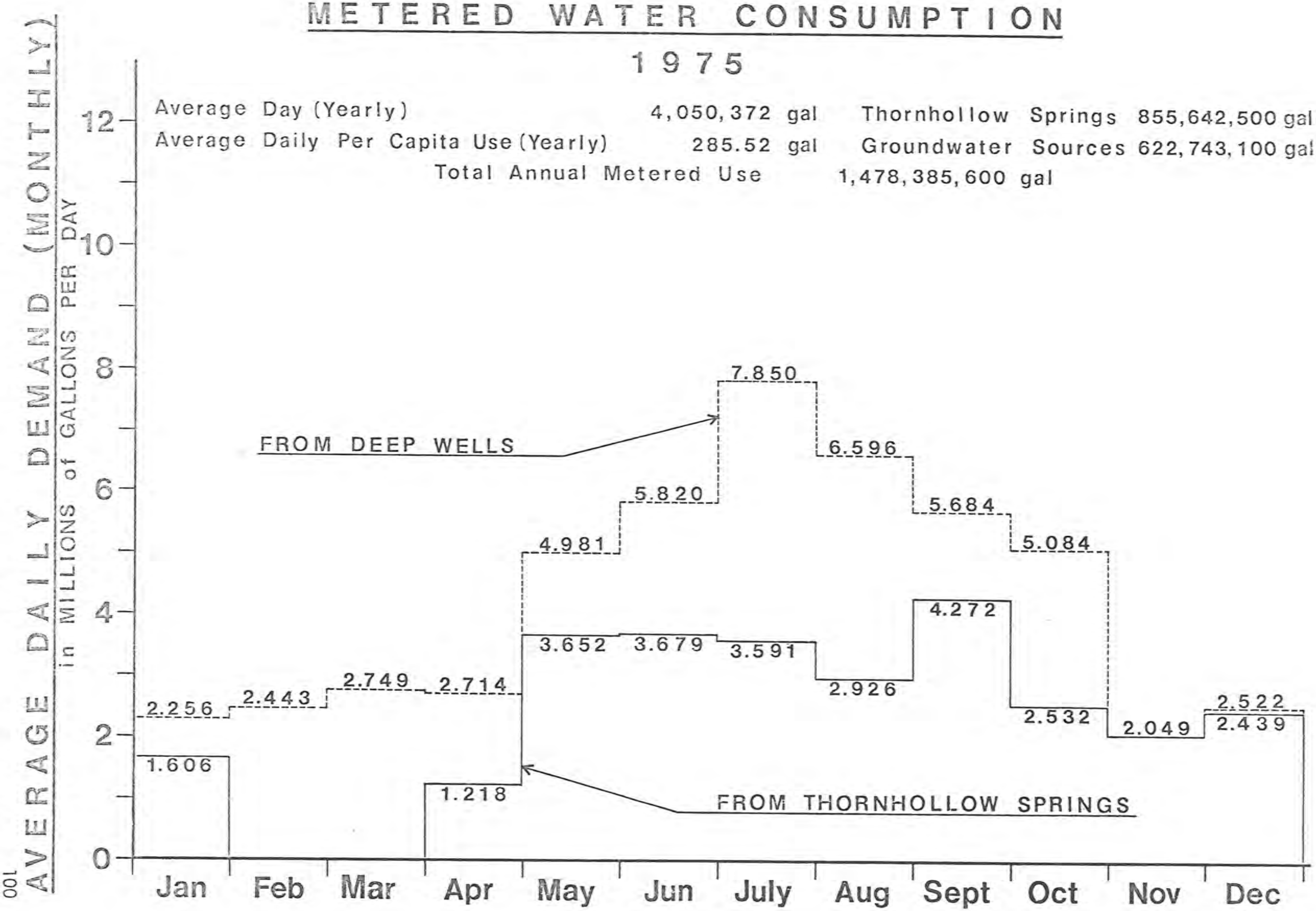
METERED WATER CONSUMPTION

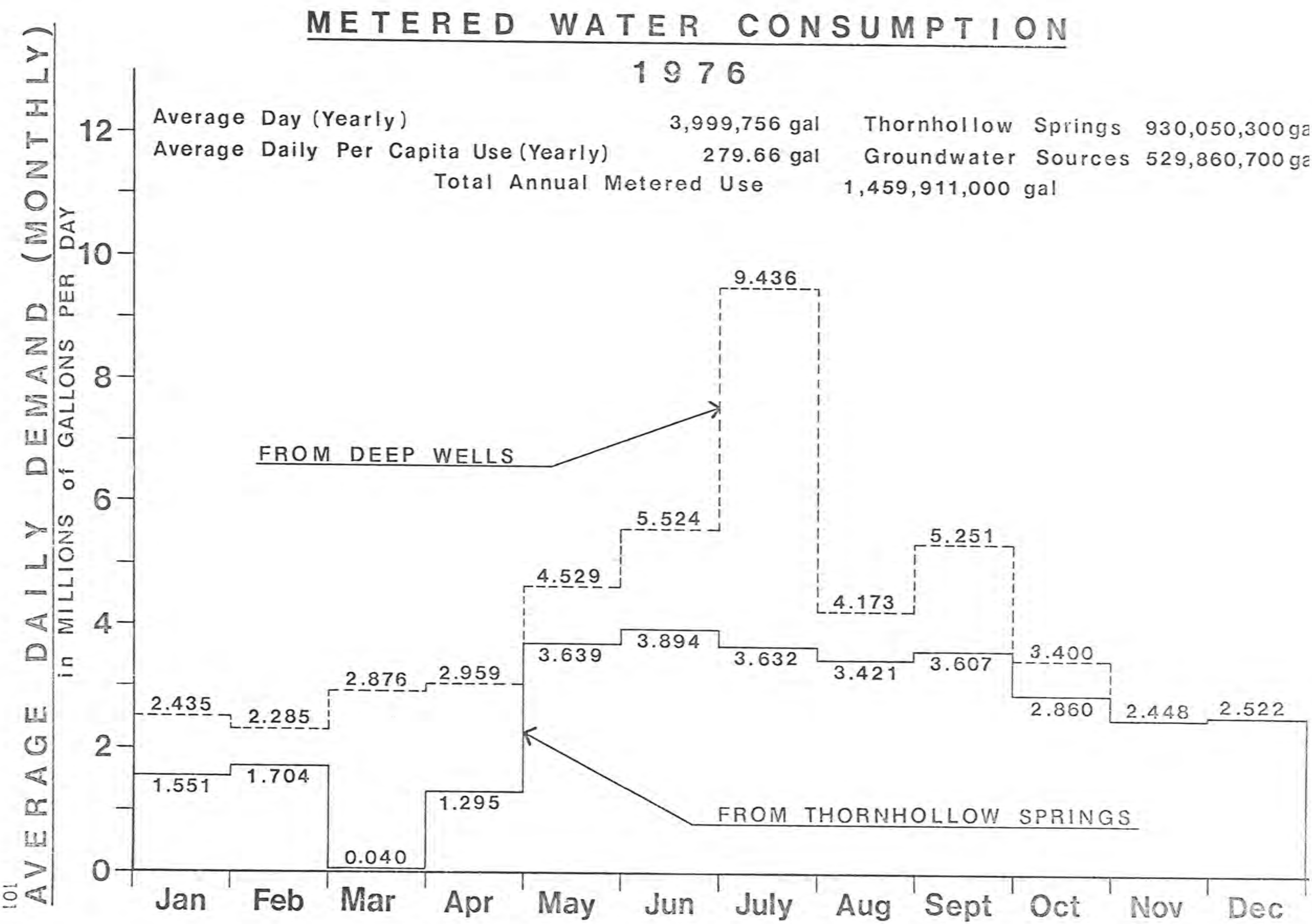



METERED WATER CONSUMPTION
1974
Average Day (Yearly) 4,220,700 gal
Average Daily Per Capita Use (Yearly) 301.26 gal
Thornhollow Springs 1,086,541,400
Groundwater Sources 454,014,100
Total Annual Metered Use 1,540,555,500 gal
AVERAGE DAILY DEMAND (MONTHLY)
in MILLIONS of GALLONS PER DAY
FROM DEEP WELLS
FROM THORNHOLLOW SPRINGS
0
2
4
6
8
10
12
2.456
2.408
2.353
2.448
3.803
4.306
4.052
6.747
3.273
7.177
2.705
7.301
3.579
5.164
3.955
5.455
2.443
2.212
Jan
Feb
Mar
Apr
May
Jun
July
Aug
Sept
Oct
Nov
Dec

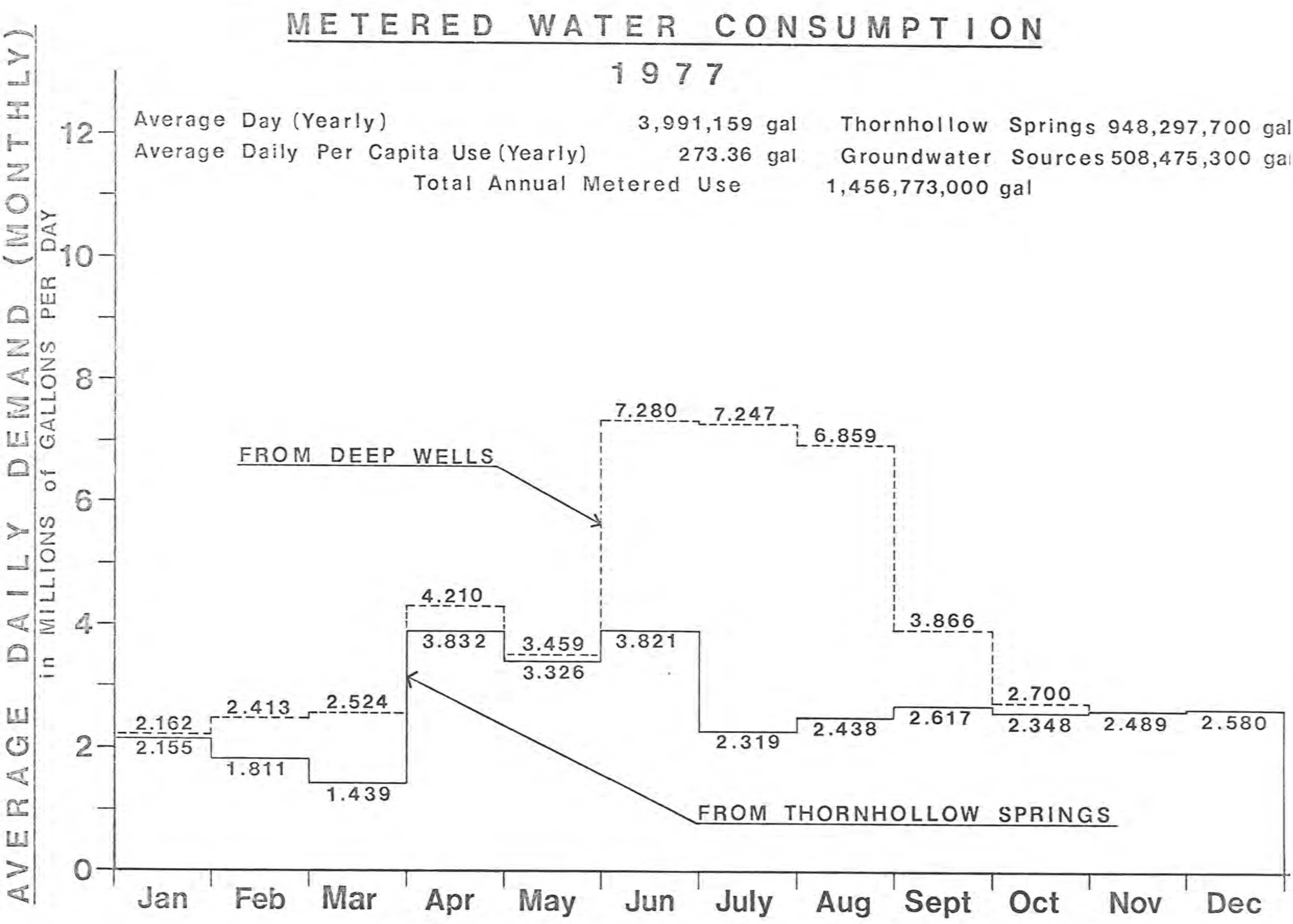
METERED WATER CONSUMPTION

1975



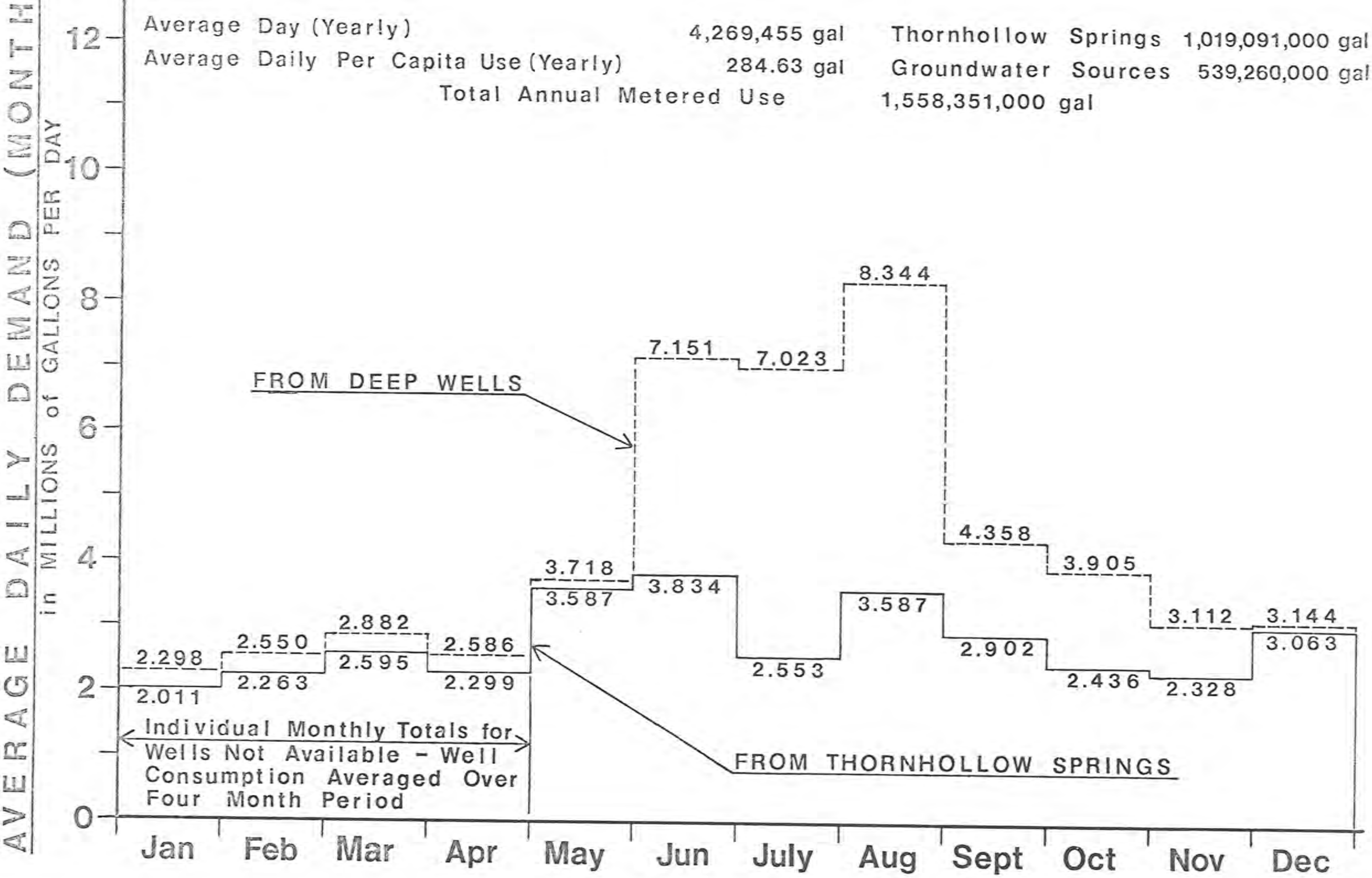


METERED WATER CONSUMPTION
1976
Average Day (Yearly) 3,999,756 gal
Average Daily Per Capita Use (Yearly) 279.66 gal
Thornhollow Springs 930,050,300 ga
Groundwater Sources 529,860,700 ga
Total Annual Metered Use 1,459,911,000 gal
AVERAGE DAILY DEMAND (MONTHLY)
in MILLIONS of GALLONS PER DAY
FROM DEEP WELLS
FROM THORNHOLLOW SPRINGS
2.435
2.285
2.876
2.959
4.529
5.524
9.436
4.173
5.251
3.400
1.551
1.704
0.040
1.295
3.639
3.894
3.632
3.421
3.607
2.860
2.448
2.522
0
2
4
6
8
10
12
Jan
Feb
Mar
Apr
May
Jun
July
Aug
Sept
Oct
Nov
Dec


METERED WATER CONSUMPTION
1977
Average Day (Yearly) 3,991,159 gal
Average Daily Per Capita Use (Yearly) 273.36 gal
Thornhollow Springs 948,297,700 gal
Groundwater Sources 508,475,300 gal
Total Annual Metered Use 1,456,773,000 gal
AVERAGE DAILY DEMAND (MONTHLY)
in MILLIONS of GALLONS PER DAY
FROM DEEP WELLS
FROM THORNHOLLOW SPRINGS
2.162
2.155
2.413
1.811
2.524
1.439
4.210
3.832
3.459
3.326
7.280
3.821
7.247
2.319
6.859
2.438
3.866
2.617
2.700
2.348
2.489
2.580
0
2
4
6
8
10
12
Jan
Feb
Mar
Apr
May
Jun
July
Aug
Sept
Oct
Nov
Dec

METERED WATER CONSUMPTION

1978



B. WATER SUPPLY.

Existing water sources are segregated into the gravity supply and wells. The concept of wells as a supply source may be a misnomer because on the basis of the available evidence, the amount of natural recharge, if any, occurring in the basalts underlying the City cannot be documented.

1. Surface Sources.

The development of a gravity line was initiated in 1913 and was operational a few years later. Over the last 26 years the surface supply has met 69.8% of the community's total water needs. This surface supply system has provided the community with low cost water for decades and appears to be destined to play an even more important role in the future of the community. The present water rights on the surface sources held by the City as presented in Table 22 which follows:

TABLE 22

SURFACE WATER RIGHTS - SPRINGS

Source or Location	Quantity CFS-MGD	Priority Date	Certificate No.
Thornhollow	4.0 -- 2.585	Nov. 28, 1910	3927
Chaplish	3.0 -- 1.939	May 20, 1912	7993
Longhair and Squaw Creek	2.0--1.292	April 22 1929	8051
Three Simon	2.7--1.745	April 22, 1929	8052
	11.7--7.561		

SURFACE WATER RIGHTS - UMATILLA RIVER

At Pendleton	2.0--1.292	Nov 11, 1885	2604
Johnson Well	1.54--1.0	?	?
	3.54--2.292		

SURFACE WATER STORAGE RIGHTS

North Fork Dam	8.0--5.170	?	?

From the information developed in Table 21, on page 88, we have converted projected annual total water demands to average day demands.

TABLE 23

PROJECTED AVERAGE DAY (ANNUAL BASIS) DEMANDS

In Millions of Gallons Per Day (MGD)

Design Population	High Year	Low Year	Average Year
15,000	3.948	3.264	3.492
20,000	5.268	4.352	4.656
25,000	6.580	5.440	5.820

The amount of water provided by the City's deep wells to meet domestic needs (excluding canneries) over the last ten years to augment the springs averaged 35,669 gal/person/year or 1.466 MGD for a population of 15,000 persons. This 1.466 MGD represents the average amount of short fall in the delivery capabilities of the springs to meet our annual need. If all of the present spring water sources were utilized, treated and placed in the underground basaltic reservoir, and recovered for peak demands, the present springs could meet 108.4% of the present City's total demand (based on population of 15,000 persons). See computation shown below:

Pipeline Full Capacity	5.250 MGD
Less Average Spring Short Fall	1.466 MGD
Net Available for City Use	3.784 MGD
Amount Required for 15,000 Population	3.492 MGD
% = Amount Available/Amount Required	108.4%

With treatment and upstream storage on either the North Fork or the South Fork of the Umatilla River, Ryan Creek, Lick Creek, Squaw Creek, or the North Fork of Meacham Creek and the full use of the existing capacity of the gravity line, the water supply system could meet the needs of 22,550 persons without modification of the present gravity line. Both Tables 15 and 16 on pages 53 and 54, respectively, indicate that there is surplus water in the winter months that could be stored for release in summer months.

2. Surface Storage Required for Future Demands.

Only a modest amount of upstream storage would be required to meet the future needs of the City if the full capacity of the gravity line was utilized and the water was treated and stored in the underground basalts for peaking periods. An analysis of the usable upstream storage required to maintain year around average flows during the summer months for population designs of 15,000, 20,000, and 25,000 persons are shown in Table 24 below.

TABLE 24
UPSTREAM SURFACE STORAGE REQUIREMENTS
BASED ON AVERAGE ANNUAL REQUIREMENTS

Design Population	Total Annual Average Daily Demand	Supplemental Flow Required from Storage in MGD Annual Basis	Usable Annual Storage Required in Millions of Gallons	Usable Annual Storage Required Required Acre Feet
15,000	3.492	-0-	-0-	-0-
20,000	4.656	0.872	318.28	976.8
25,000	5.820	2.036	743.14	2,280.8

The construction costs of storage dams on any of the possible potential upstream sites would vary considerably with the site topography, the underlying soil profiles, spillway requirements, fish ladders, control works, and availability of materials to construct an impermeable core. The impoundments would be a combination earth-rock fill structure with the optimum use of an on-site material. Without the benefit of an on-site detailed investigation of any of the impoundment sites, the following costs have been adjusted from information in past studies and are offered as "shotgun" estimates only.

TABLE 25
ESTIMATED COSTS FOR UPSTREAM IMPOUNDMENTS

Design Population	Usable Storage Required Acre Feet	Total Storage Required Acre Feet	Construction Cost Per Acre Foot in $1,000	Total Estimated Cost In Millions
15,000	-0-	-0-	-0-	-0-
20,000	976.8	1,200	2.5	3.0
25,000	2,280.8	2,600	1.8	4.68

In the above tables the estimated total storage and usable storage are premised at this writing on several unknowns such as stream transport losses, evaporative losses, direct subsurface leakage, leakage around the structure and the ratio of depth to storage volume. Several basic costs affect the cost of impoundments regardless of size. Some of the costs that are not proportional to size are spillways to bypass floods, seepage cut-off trenches, concrete foundation and grouting, outlet structures, contractor mobilization costs, relocation of existing improvements such as roads, diversion of stream flow during construction, and the obtaining of approvals for the construction of a dam. From a quick review of the above fixed costs it is obviously less economical on a unit acre of storage to build the smaller impoundment in any of the considered principal streams.

In recent years several studies have pointed out the value of upstream storage to lower stream turbidities, reduce erosion, and with summer releases improve minimum stream flows for the migration of fish. To satisfy future annual water requirements for populations in excess of 16,250 persons and provide adequate supply to maintain an artificial recharge program would require the construction of a dam on the upper reaches of the Umatilla River. Release of needed water in the summer months would improve the summer stream flows for an approximate distance of 16 miles to a pick-up point near the present Weir house. For population designs above 22,550 persons, the capacity of the existing gravity line would have to be increased by laying an additional pipeline, pressurizing the existing line by inserting a liner pipe or the additional flow would have to be "picked up" approximately 14 miles downstream from the Weir house at Site A, one of three sites considered for the construction of a water treatment plant. The latter case of picking up the water further downstream would probably receive more popular support and depending on stream transmission losses, it may be the most cost effective solution for the City. Below any site considered for a water treatment plant, the existing gravity line capacity would have to be increased for population designs above 22,550 persons. Water storage permits should be sought from the State Water Resources Department on all potential sites where upstream impoundments might prove feasible. The amount of the requested storage should be in excess of the amounts suggested by this study. It would seem only prudent at this time to secure sufficient future rights to permit growth beyond the contemplated population design of 25,000 persons. The costs of maintaining these permits are extremely low when one contemplates the consequences of failing to act at the earliest possible opportunity. A legal opinion should be sought on the status of the existing Umatilla River surface rights, the North Fork storage rights and the Johnson Well for which the rights may have been non-existent.

The location of the springs, the Johnson Well, and the gravity supply line is shown on a vicinity map on page 108. Information in prior studies indicated that during very low river flows that priorities from 1905 back would have prior rights to the total Umatilla River flow and may result in rendering the spring

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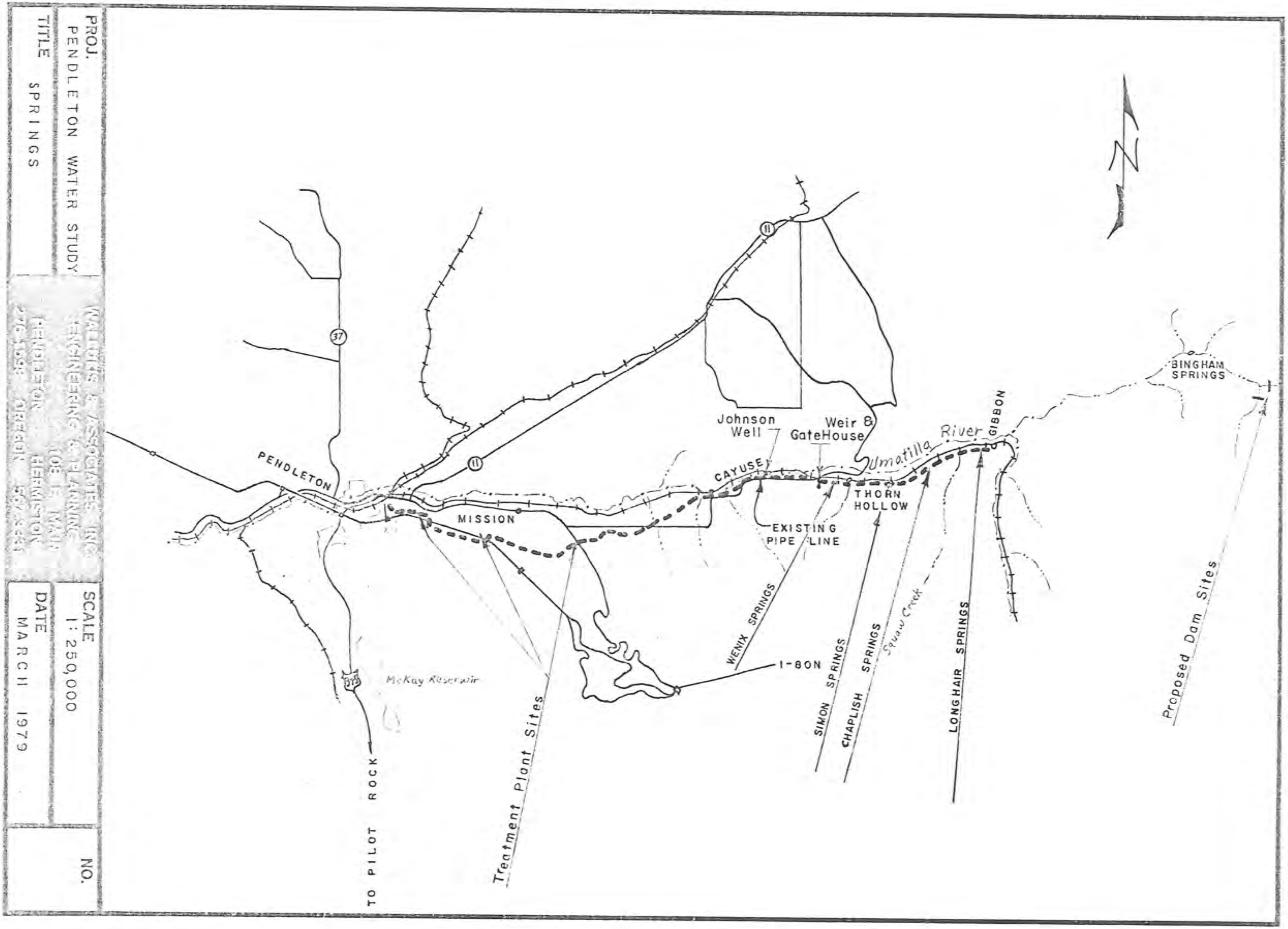


sources useless. In this same report it was also stated that since the City acquired the land directly from the Umatilla Indians that they may be able to claim their prior aboriginal rights. Whatever the legal position might be, the City's use of the springs was uncontested in the drought year of 1977 which may or may not provide a false sense of security.

During the winter months of each year when there is a surplus of flow in the Umatilla River, the City could exercise the full amount of their water rights of 7.561 MGD without complaint. The capacity of the existing gravity line has been determined to be 5.25 MGD and therefore limits the amount that can be transmitted to the City.

3. Deep Well Sources.

Since 1948 the City has relied on the underground aquifers in the basalts to augment normal demands and other peak water demands when the flow from the springs dropped off. The City presently uses six wells for meeting these demands. Their present yields, present water rights, and dates of water rights are shown in Table 26 below:

TABLE 26

DEEP WELLS - WATER RIGHTS

Well No.	Well Name	Present Yield MGD	Original Water Right MGD - Date	Supplemental Water Right MGD - Date
1	Byers	2.592	2.003 - 2/23/44	0.582 - 7/16/62
2	Round-Up	3.024	1.622 - 9/16/53	2.003 - 7/16/62
3	S. W. 21st Street	1.080	0.717 - 12/31/51	0.129 - 7/16/62
4	Hospital	1.152	1.293 - 10/18/54	-0-
5	Stillman	3.312	3.425 - 10/3/58	-0-
6	Hiersche's	-0-	-0-	-0-
7	Mission	1.152	4.330 - 3/22/66	-0-
	Totals:	12.312	13.39	2.714

All of the City wells currently in use are withdrawing ground water from the underlying basalts at a rate less than their present water rights with the exception of the S. W. 21st Street well. If the production of the S. W. 21st Street Well was limited to the amount of its water right the total production from City wells would be reduced from 12.312 MGD to 12.078 MGD.

By transferring water rights from Well No. 2 that is not pumping up to the full amount of its water right, the total amount of 12.312 MGD could be legally re-established.

The waterworks field generally works on the criteria that 75% of the yield from multiple well sources should be used as a basis for a firm supply to meet peak demand conditions. This 25% allows for avoiding short cycling periods for electrical motors, mechanical failure, electrical outages, unscheduled maintenance, malfunction of controls, accidental rupture of supply line from the source and a host of other possibilities. Based on 75% of the total yield from the existing City wells, excluding the Mission Well, the firm supply from the wells without modification of existing water rights is:

$$10.926 \text{ MGD} \times 75\% = 8.195 \text{ MGD}$$

For the balance of this report we have assigned the production of Well No. 7, Mission, to be the source to meet the demands of the Umatilla Indian Reservation and other users from this system. As the yield from Well No. 7 far exceeds the present demands of these users this is a conservative position in determining the future total water demands of the Pendleton water system.

In reviewing the historical data from the water consumption graphs on pages 92 through 103 for the last 12 years, the amount of water supplied from wells to meet the maximum monthly demands for each year are shown in Table 27 which follows.

TABLE 27

MONTHS OF MAXIMUM WELL USAGE

Average Day (Monthly Basis) in MGD

Excludes Canneries

Year	Max. Month	Gravity Supply	Well Supply	Water Demand on Well Supply Adjusted to 15,000 Population	Demand on Wells With Projected 20% Reduction In System Demand
1967	8.420	3.063	5.357	5.769	4.003
1968	8.871	3.094	5.777	6.020	4.197
1969	7.898	3.232	4.666	5.390	3.666
1970	9.474	4.715	4.759	6.053	3.899
1971	7.642	2.689	4.953	5.834	4.129
1972	7.372	2.545	4.827	5.586	3.959
1973	8.794	2.582	6.212	6.852	4.965
1974	7.301	2.705	4.596	5.112	3.549
1975	7.850	3.591	4.259	4.709	3.049
1976	9.436	3.632	5.804	6.265	4.286
1977	7.247	2.319	4.928	5.127	3.638
1978	8.345	4.757	3.588	3.588	

Based on information developed in Table 27, the highest demand (adjusted to 15,000 population) placed on the deep well sources occurred in August of 1973. During this maximum month the wells provided an average daily supply of 6.852 MGD or only 83.6% of the combined firm well supply of 8.195 MGD (75% of total well yields excluding the Mission Well). The amount or reserve supply capacity available to meet daily peak demands occurring during this maximum month is shown below:

Firm Well Supply (excludes Mission Well)	8.195 MGD
Gravity Supply	2.582 MGD
	10.777 MGD
Less:	
Average day @ max. mo. demand: 8.794 MGD adjusted to 15,000 population: 9.434 MGD less 20% decreased future use	7.547 MGD
Firm reserve available for daily demand	3.230 MGD
Amount of firm supply in reserve to meet maximum day demands during maximum month	
3.23 MGD ÷ 8.195 MGD =	39.4%

In communities in semi-arid areas without the presence of large industrial demands the variation between the average day during the month of maximum demand versus the maximum day demand during abnormally hot summers is generally only 10 - 15% higher than the average day during the maximum month or considerably less than the available 39.4%. This is because irrigation habits in unusually hot summers are well established prior to the occurrence of the maximum demand month.

With upstream storage, summer releases, and the utilization of the full capacity of the existing gravity line and treatment, a base flow of 5.25 MGD could be assured. In reviewing the water consumptions graphs, the highest aggregate total monthly demand of 10.768 MGD (adjusted to 15,000 population) excluding industry, occurred in July of 1970. With a larger summer base flow of 5.25 MGD instead of the recorded low spring flow of 2.582 MGD; a 20% reduction in future water use; Wells No's. 1, 2, 3, 4, and 5 could easily satisfy future projected summer demands as shown on Table 27 on page 112 for a population of 15,000 persons. Presented below are calculations which demonstrate the value of having the larger base flow during extremely adverse summer months which as of yet are not of record.

Firm Well Supply @ 75% (excludes Mission Well)	8.195 MGD
Improved Gravity Supply	5.25 MGD
	13.445 MGD
Less:	
Average day @ max. mo. demand: 9.474 MGD (1970) Adjusted to 15,000 population: 10.77 MGD Less 20% decreased future use	8.614 MGD
Firm reserve available for daily peak demand	4.831 MGD

Amount of firm supply in reserve to meet maximum day demand during maximum month:

4.831 MGD ÷ 8.614 MGD = 56.1%

The construction of an upstream impoundment, full utilization of the gravity line, and artificial recharge of the underground basalts would meet the maximum month requirements of a population of approximately 20,360 persons with a 15% reserve for the maximum day. When the population exceeds 20,360 persons additional wells to withdraw water from the underground reservoir would be required to meet summer demands. For populations above 20,360 persons, the additional supply from wells to meet summer demands are shown in Table 28. Table 28 is premised on upstream storage and a treatment plant being constructed prior to the City reaching a population of 20,000 persons.

TABLE 28

FUTURE WELL REQUIREMENTS
(Premised on adequate upstream storage, treatment, and increased gravity line capacity from the treatment plant to the City)
in MGD

Population Design	Average Day Maximum Month	Maximum Day 115% Average Day	Total Combined Firm Supply	Additional Firm Capacity Required	Total Additional Capacity Required
20,000	11.485	13.208	13.445	-0-	-0-
25,000	14.356	16.509	13.445	3.064	4.085

Based on information developed in Table 28, additional well production of 2,840 GPM (4.085 MGD) would be required to meet summer demands for a population of 25,000 persons.

The new Hospital Well with a present yield of 1,100 GPM (1.584 MGD) would easily satisfy water demands for a population design of 22,157 persons and has the further potential of increasing the capacity of the well to 3,000 GPM to also meet the needs for a future population design of 25,000 persons. It may be more prudent, however, to explore the possibility of acquiring the use of the Brogoitti Well for the long term. This would increase the overall reliability of the well field by not concentrating so much production in one well. The Brogoitti Well and the new Hospital Well are strategically located where they could service the growth in the western and southwesterly areas of the City. These wells also possess the highest specific capacities which make them excellent wells for artificial recharge.

CHAPTER VI

TREATMENT OF WATER SOURCES

A. PURPOSE

The reclassification of the water supply from the Thornhollow Springs from a groundwater source classification to a surface source classification by EPA placed the continued use of the City's major source of water in jeopardy. EPA's justification for their position on the Thornhollow Springs was stated in Chapter III. EPA as a federal agency has had more experience with the administration of wastewater facilities (Public Law 92.500, enacted in October of 1972) than the administration of the Safe Drinking Water Act (Public Law 93.523, enacted in December of 1974).

From attendance at State, Regional, and National meetings at which EPA representatives were present as guest speakers, this author had the opportunity along with others concerned in the water and wastewater field to enter into dialogue with personnel from the EPA staff. The staff of EPA state their case as trying to follow a middle road between environmentalist groups who "haul" them into Federal courts for inaction and the entities charged with supplying water and the treatment of wastewater. Based on personal experience of these meetings, an opinion which is shared by several others in the supplying of water and treatment of wastewater is that EPA generally over-reacts to pressure from environmentalist groups.

The position of no treatment being required for the Thornhollow Springs sources would be particularly difficult to defend because of the seasonable high turbidities and lack of complete control over the contributing watershed. Prior studies performed for the City by others and Federal geologists have indicated in their reports that the Umatilla River is a contributor to the flow from Thornhollow Springs. At a seminar held in Boise, Idaho, the only two variances granted for turbidity for surface water supplies by EPA were:

1. Cities in Alaska where ice crystals form when the water is withdrawn from pressurized systems; and

2. Cities whose water sources were from protected watersheds high in the Rocky Mountains and only then after the turbidity was documented to be fine granite particles suspended in the water.

The EPA has an authorized staff level in their Washington, D.C. office of 448 persons to implement enforcement proceedings plus support staff from their regional offices. Mandates from EPA should receive serious consideration or local resources that otherwise could be placed to beneficial use might be expended in the courts.

Upon review of the declining water table in the underlying basalts, the reclassification of the Thornhollow Springs as a surface supply may actually prove to be a "blessing" for the City. Without this reclassification the City may have continued to overlook the problem of the declining level of the underground reservoir of water and its potential for being utilized as a large equalizing reservoir.

B. FACTORS AFFECTING HYDRAULIC DESIGNS.

The quantative flow requirements for the proposed water treatment plant will be premised on artificial recharge into the underlying basaltic aquifers. If this concept cannot be effectuated, the size of the water treatment plant would have to be increased 284% in size to meet summer peaking demands and the size of upstream storage impoundments and the existing gravity line would have to be similarly enlarged. Based on data in Table 29, a comparison is made on the hydraulic requirements of a water treatment plant based on the concept of artificially recharging the underlying basalts versus the eventual loss of the deep wells as a source; and summer demands met solely from the springs and upstream storage.

TABLE 29

REQUIRED TREATMENT PLANT HYDRAULIC CAPACITIES GROUNDWATER RECHARGE vs LOSS OF GROUNDWATER SUPPLY in MGD

Design Population	Without Recharge High Demand Day *	With Recharge Average Demand Year
15,000	9.906	3.492
20,000	13.208	4.656
25,000	16.509	5.820

* Table 27, for the year 1970 max. mo. adjusted to a population of 15,000 persons, the average day during the maximum month was 10.768 MGD less projected 20% consumption reduction; multiplied by 115% for the peak day results in a maximum day demand of 9.906 MGD.

Assuming artificial recharge can be implemented and upstream storage provided, the water treatment plant would be capable of producing the same amount of water around the clock every day of the year. This would provide a maximum return on the capital invested, stabilize manpower requirements and utilize the existing gravity lines and springs to their full potential.

Assuming the worst possible circumstances of the underlying reservoir being pumped dry and that artificial recharge could not be implemented, the City would be required to:

a. Construct a water treatment plant 284% of the size considered in this report.

b. Significantly increase the carrying capacity of the gravity line from the treatment plant site at the Mission Well or move the treatment plant to a location along the river in the City. An in-city location would require high energy costs because of the pumping head that would be required to introduce the water into the distribution system.

c. Additional transmission capabilities would have to be constructed in the existing distribution system to permit mass transfers of water to conventional reservoirs. Presently wells located in different areas of the City enable localized demands to be satisfied from nearby wells.

d. Substantial increase in the capacity of conventional reservoirs would be required if the underground reservoir became completely depleted.

e. Upstream river impoundments would have to be substantially increased to meet the total annual water demands. A comparison of storage requirements in upstream dams with and without artificial recharge in shown in Table 30.

TABLE 30

UPSTREAM RIVER IMPOUNDMENT REQUIREMENTS

RECHARGE vs LOSS OF WELLS

in acre feet

Design Population	With Recharge Average Demand Year *	Without Wells Maximum Demand Year **
15,000	-0-	2,050
20,000	1,200	3,730
25,000	2,600	5,412

* From Table 25, page 106

** From Table 21, page 88, high consumption years less low year gravity supply of 854.4 mil. gal. in Table 6, page 32, plus loss allowance.

The balance of this report will be predicated on the premise that an artificial recharge program can be implemented.

C. SURFACE WATER QUALITY.

The turbidity of the gravity water varies considerably each year with the higher turbidities occurring during or following periods of precipitation or snow melt. When the turbidity of the gravity supply reaches a predetermined level this source is automatically turned out to the Umatilla River.

A future water treatment plant would treat all of the water from the springs. Without historic information on actual water quality during periods of high turbidity, it is difficult to establish the amount of chemicals that will be required to provide a satisfactory quality water.

At the insistence of EPA the City purchased an approved type turbidimeter (a nephelometer) to monitor the turbidities of the gravity supply at the South Hill Reservoir. A comparison between the turbidimeter at the Weir house and the nephelometer at the South Hill Reservoir shows that above 4 mg/L (NTU - Nephelometer Turbidity Unit) the turbidimeter registered 80 to 100% of the nephelometer readings; in the range of 1.5 to 2.5 mg/L (NTU) the turbidimeter registered 65% of the nephelometer readings; in the range of 1.0 to 1.5 mg/L (NTU) the turbidimeter registered 60% of the nephelometer readings; and in ranges below 1.0 mg/L (NTU) the turbidimeter registered 50% of the nephelometer readings.

Based on the above relationships and a review of the City's records the spring turbidities are normally low enough by May of each year to be under the maximum allowable turbidity established by the Safe Drinking Water Act. In 1978 the turbidity of the springs exceeded the allowable maximum turbidity of 1 mg/L (NTU) until the middle of July.

The estimated cost of treating the spring water source is $0.10/1,000 gallons (1979 cost basis). This estimate of operational cost is based on the author's personal experience gained from being Utility Engineer for Corvallis, Oregon for three years. The Rock Creek Water Treatment Plant (one of two) treated raw water from the protected Mary's Peak watershed which was stored in a 100,000,000 gallon open reservoir prior to treatment. The raw water received at the Rock Creek Water Plant was also low in turbidity and required a minimum of treatment to produce water of excellent quality.

Based on limited data available it is probable that for several months of each year that only filtration would be required. During this time period when only filtration would be required, the only chemicals added to the water would be chlorine and flouride.

D. TREATMENT PLANT: FUNCTIONS AND COMPONENT.

For most months of the year the water treatment plant would act as a "polishing plant" to remove minute colloidal particles (in suspension) that were not naturally filtered out in the upstream infiltration galleries. During other periods of the year when turbidities where high, chemicals would be added to facilitate consolidation and settling of the suspended particles. Alum (aluminum Sulphate) is the chemical normally used to enhance coagulation of the suspended particles into a sticky, milky-colored precipitate called "floc" which settles out into a settling basin. The purpose of the use of coagulants and sedimentation is to reduce the amount of the suspended matter deposited on the following filter media.

After the introduction of coagulants and prior to sedimentation, the water is gently mixed in a coagulation chamber to permit the injected chemicals to react with the suspended materials in the raw water. This function has been historically accomplished by paddle wheels moving slowly through the coagulation basin.

After settling in a sedimentation basin (settling tanks) the water is passed through filtering beds containing filtering media. Multiple filter beds are provided so that some can be cleaned by backwashing without reducing the production capacity of the plant. The filters intercept any floc that has failed to settle out and also remove bacteria and colloidal material that failed to coagulate and settle out. Filters also improve the color and taste of water and reduce the amount of iron and manganese in the raw water.

Water from stream impoundments often create taste and odor problems if algae growths are permitted to flourish. The periodic introduction of small amounts of copper sulphate in surface impoundments inhibits the growth of troublesome algae. Copper sulphate in the quantities required to control algae is not hazardous to humans or fishlife. The removal of taste and odors in raw water received at a treatment plant is normally accomplished by the injection of a slurry of activated carbon over the top of the filter beds.

After the water leaves the filters it is chlorinated and flouridated prior to being placed in a clear well (storage tank). The functions of a clear well is to act as a contact chamber (30 minutes required for sterilization by chlorination); provide a supply of water for the backwashing to filter media; for visual observations, and provide automatic monitoring equipment with adequate response time to effectuate a by-pass if required, prior to the release of the treated water into the system.

The Umatilla River water and the water from the springs tend to be corrosive. The addition of alum to facilitate coagulation and sedimentation will aggravate this characteristic. This will require the addition of hydrated lime to lower the corrosiveness of the treated water.

In recent years the use of Polymers (organic chemicals) have become popular both as a coagulant aid and for the reduction in the amount of chemical sludges which precipitate in the sedimentation chamber. The Federal Water Pollution Control Act enacted in 1972 (Public Law 92.500) prohibits the discharge of pollutants into streams. Prior to the enactment of this act the standard practice was to discharge filter backwash water from the filters and the chemical sludges from sedimentation basins to the nearest receiving stream. Now this backwash water with its chemical sludges must be deposited into holding ponds (lagoons); and the settled water recycled through the water treatment plant or used to irrigate land. Alum sludges are gelatinous and are best dewatered where possible by natural freezing and thawing. The freezing and thawing concentrates the particles by an irreversible process into a brown-like powder or sand that can be used as a soil conditioner. Unlike Alum s ludges, Polymer sludges are easily dewatered and are suitable for use as soil conditioners. In some instances a combination of Polymers and Alum can be practiced, substantially reducing the amount of chemical sludge that would be generated by Alum alone. The responsiveness of the different Polymers available to a particular raw water source can only be reasonably ascertained by an extended period of trial and error. In some instances the uses of Polymers has been completely ineffective. Multiple ponds for sludge settling are proposed for the City of Pendleton to take advantage of the freezing winters which will concentrate and convert the waste sludges into a soil conditioner.

Some communities have taken the position that the decanted liquid from the settling ponds contain carcogenic (cancer causing agents) complex chemicals that may not be removed in recycling back through the treatment processes. Their concern is the continued recycling of the decanted liquid will eventually raise the carcogenic chemical levels to a point where it would endanger public health. EPA and the scientific community have not reached a definitive position on the risks of recycling the settled wastewater. In this study we have provided estimated costs for both recycling of the settled wastewater and for holding ponds with disposal by irrigation.

For the concept of using the settled wastewater we are assuming that Pendleton's water source would be similar to that of the Rock Creek Plant at Corvallis. Three percent of the total water produced at this plant is required to aid coagulation and for the backwash the filters. Designing the Pendleton Plant to the size of the capacity of the existing gravity line of 5.25 MGD (Design population = 22,500 persons) the wastewater volume of design capacity would be 176 acre feet per year. During the spring, summer and fall seasons the decanted liquid could be used for on-site irrigation of a crop. This would require approximately 80 acres to dispose of the wastewater by irrigation. Complete holding would be planned for three months during the winter. This would require approximately 44 acre feet of storage. For the freezing to be effective the holding cells would be designed for a depth of two feet. A minimum 40 acres (includes dikes) would be needed for this means of disposal.

Chemical sludge settling ponds (lagoons) could also be beneficially utilized to level out the impact of by-passed flows from the treatment plant if required by a malfunction. This would also eliminate the requirement of constructing a separate waste line to the Umatilla River and the threat of fines by EPA for the polluting of the Umatilla River.

E. TREATMENT PLANT COMPONENTS AND COSTS.

1. Treatment Plant Component and Structure.

The principal components of a water treatment plant have already been discussed generally in the preceding sections of this chapter. A schematic flow diagram of the type of water treatment plant required for the City of Pendleton is shown on page 124.

Incorporated into the cost of the water treatment plant will be the complete enclosure of the area over the flocculation chamber, the sedimentation basin, and the filter beds. The basis for the complete enclosure of this surface area is as follows:

a. Protection from aerial applied insecticides, herbicides, and fertilizer coming in contact with the surface water.

b. Shelter from dust during windstorms with deposition of soil particles over the total surface area overloading the filter media.

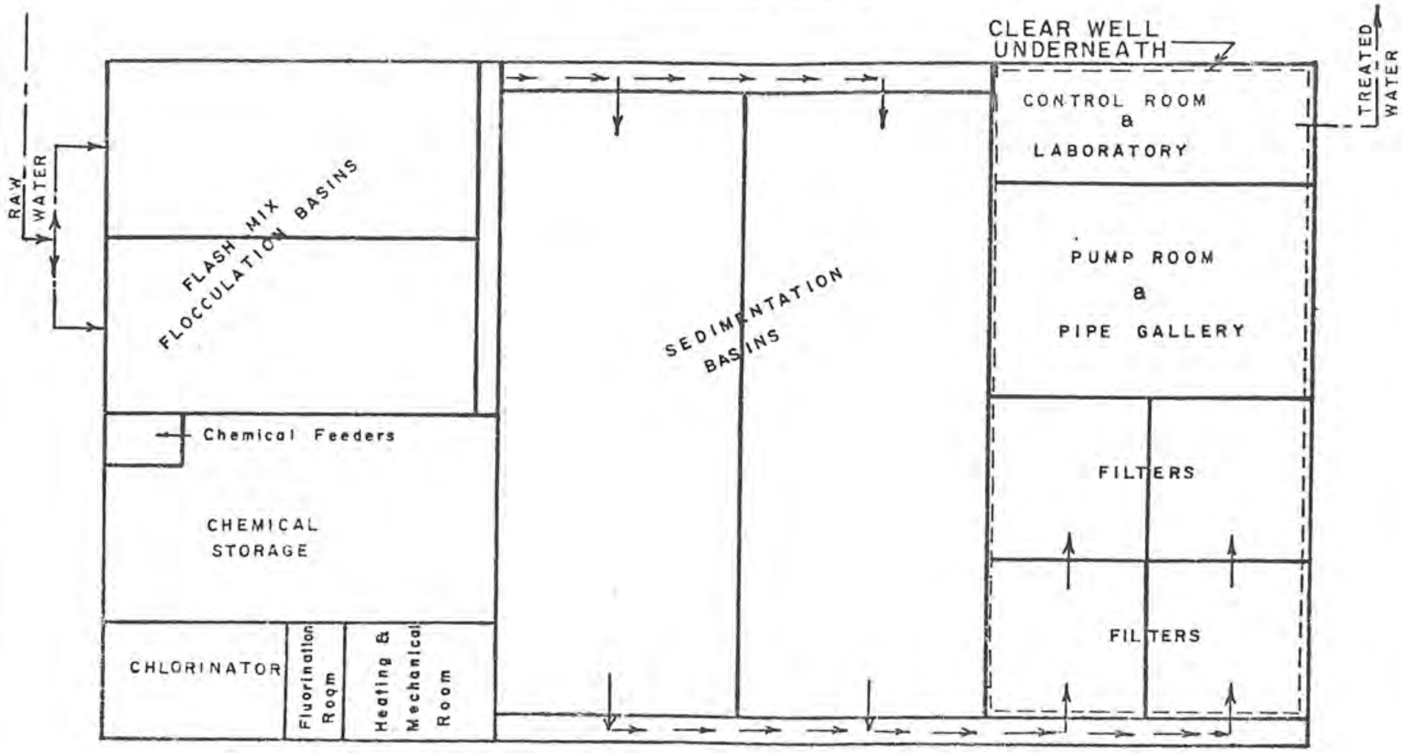
c. Retard the growth of algae promoted by sunlight in what may be nutrient rich waters from upstream surface impoundments.

d. Prevent the formation of surface ice during cold windy winters.

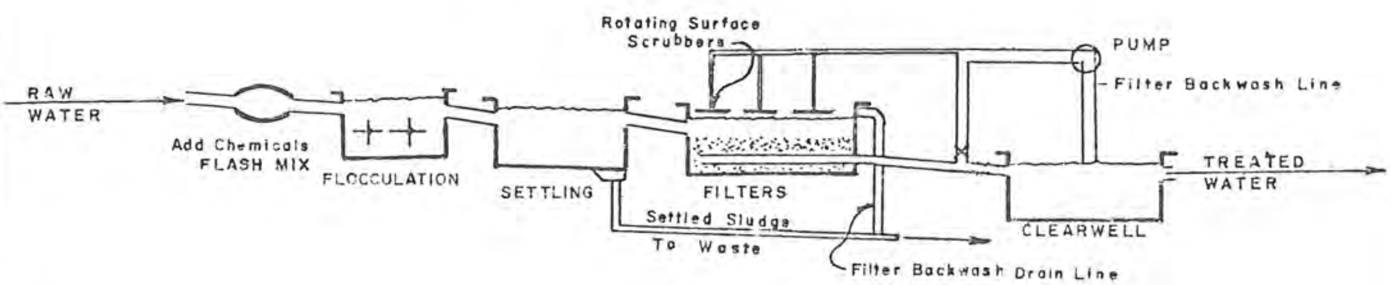
e. Guarantee the integrity of the water supply by preventing vandals from introducing chemicals, drugs or other foreign substances into the public water supply.

To minimize vandalism, it is our recommendation that consideration be given to the construction of two modest residences for treatment plant operators or that sewer and water facilities be provided to accomodate two mobile homes. On-site residency would minimize damage from vandals. Facilities of this type left unattended often become prime candidates for vandalization and general mischief.

BUILDING PLAN



SCHEMATIC FLOW DIAGRAM



BRUNING 40-22 33381

PROJ. PENDLETON WATER STUDY	WALLULIS & ASSOCIATES INC ENGINEERING & PLANNING 108 E. MAIN PENDLETON HERMISTON 276-1598 OREGON 567-333[illegible]	SCALE N.T.S.	NO.
TITLE WATER TREATMENT		DATE APRIL 1979	124

2. Treatment Plant Costs.

The estimated cost for the water treatment plant is based on a production capacity of 5.25 MGD. The treatment plant as preliminary designed should be capable of sustaining a continuous output of 5.25 MGD based on the plant being operational 24 hours per day. Since only low head pumping is required, standby power generation facilities are proposed to automatically take over in the event of power failure. For a basic plant with recycling of the wastewater back through the water treatment plant the estimated costs for construction in 1981 are as follows:

TABLE 31

BASIC WATER TREATMENT PLANT COSTS

a) Site acquisition: five acres @ 2,000	$ 10,000
b) Mobilization	25,000
c) Bond and insurance	30,000
d) Piping to and from plant	70,000
e) Excavation and backfill	30,000
f) Reinforced concrete	360,000
g) Concrete block walls	45,000
h) Roof structure	50,000
i) Miscellaneous concrete work	5,000
j) Painting - walls, piping, etc.	15,000
k) Plant piping and fittings	110,000
l) Manual and automatic valves	42,000
m) Drainage systems, overflows, etc.	23,000
n) Heating, ventilation	30,000
o) Flocculation and sedimentation equipment	100,000
p) Filter beds and equipment	130,000
q) Chemical feeder - all	60,000
r) Electrical, pumps, motors	105,000
s) Plant control and monitoring equipment	75,000
t) Chemical storage facilities	24,000
u) Miscellaneous metalwork	20,000
v) Laboratory equipment	25,000
w) On-site subsurface waste disposal	7,000
x) Landscaping, access road, parking, and fencing	12,000
y) Three one-acre wastewater ponds	40,000
z) Operational testing and final cleanup	10,000
	$ 1,453,000
Contingencies and Engineering - 25%	363,000
Total estimated cost	$ 1,816,000

For the non-recycling of wastewater, additional land would have to be acquired for increased holding cell capacity and land for irrigation on which to apply the decanted liquid. As this water is suitable for the production of animal crops, revenue would be derived from the custom cropping of the 70 acres that would be irrigated, offsetting all or most of the additional capital costs. The additional costs to implement this alternative are as follows:

TABLE 32

OPTIONAL WATER TREATMENT PLANT COSTS

a) Land requirement = 90.0 acres See alternative sites on map on page Estimated value 120 acres @ $500	$	66,000
b) Additional pumping equipment for irrigation		5,000
c) Piping to ponds, from ponds and irrigation (manual set)		33,000
d) Security and stock fencing		17,000
e) Construction of 22 acre (water surface) of holding ponds		120,000
Sub total	$	241,000
Add Basic Plant Cost	$	1,453,000
	$	1,694,000
Contingencies and Engineering - 25%		423,500
	$	2,117,500

Additional options to consider are on-site residential units that would probably cost $50,000 each or mobile home sites (sewer and water facilities only) at $3,000 each.

The various alternative treatment site locations considered are shown on the map on page 57. If land acquisition costs were the same for all sites, Site "A" would be preferred because it is closest to the river, has good access and would require the least amount of additional gravity pipeline construction for populations above 22,500 persons. Site "B" would be the second best site and Site "C" would require the largest capital expenditure for populations above 22,550 persons. Site "C", however, is the only treatment plant site that is capable of serving the Umatilla Indian Agency. For this reason the Mission Well was assigned to providing water to the Agency and other area users. With this reduced demand on Well No. 7 of approximately 57,320 gal/day (1973, 1974, and 1975 metered use), the life of the Mission Well should be greatly extended. Based on prior usage records the Mission Well pumping at the current 800 GPM could supply this area's total annual water demand with 18.2 days of pumpage. A water system feasibility study dated July 1979 by Anderson - Perry & Associates, Inc., for the Tribe has projected the following:

DOMESTIC & INDUSTRIAL DEMANDS IN GPM

Location	Year 1980 Ave. - Peak	Year 2000 Ave. - Peak	Year 2020 Ave. - Peak
Area 1	73 - 218	211 - 745	281 - 1025
Area 1 & 2	117 - 351	292 - 1068	389 - 1457
Area 1,2,3	129 - 388	318- 1173	426 - 1607

The average water demands shown above are based on annual average, daily per - capita demand and peak per - capita day demands. Area 1 includes the river valley floor south of the Umatilla River from 0.4 mile west of (North - South) State Hwy. 331 and 2.0 miles east of State Hwy. 331. Area No. 2 is the Westerly extension of Area #1 to the West border of the Reservation lying south of the Umatilla River. Area #3 consists of the valley floor North of Areas #1 & #2 (North of the Umatilla River).

This study envisions major improvements in distribution, storage and source development. The Indian Health Service has agreed to construct at least one well and provide flouridation and disinfection if required. With the relatively low demand the proposed improvements should make the new proposed system self sufficient to serve the needs of the Mission area. The Tribe does, however, expect the City to continue to provide some amount of "free water" in accordance with a prior agreement as a consideration in granting the easement for the gravity line. Until the Tribe obtains a grant; additional water supply; or a new agreement is negotiated, the yield from Well No. 7 shall be considered as the source to meet the water demands of all users in the Mission area.

CHAPTER VII

CONVENTIONAL STORAGE

A. EXISTING STORAGE.

The City presently has two reservoirs serving the basic distribution system (low level), and two more servicing high level service areas, one of which services the upper North Hill and the second one services the airport. The size and location of these reservoirs is given below:

Name/Location	Service Level	Capacity in Million Gallons
South Hill	Low	1.9
North Hill	Low	1.1
North Hill	High	0.3
Airport	High	0.5
	Total:	3.8

The Airport reservoir is a ground level reservoir with system pressures maintained by pumps pumping out of the ground level reservoir. There are high level areas that are dependent on pumping stations from the distribution system of the low level system for their supply of water. These areas are:

1. Mt. Hebron
2. South Hill, including Indian Hills area
3. Youngs Addition - Community Hospital
4. S. E. 20th Street and S. E. Court Avenue

B. FIRE DEMANDS.

Several years ago the National Board of Fire Underwriters (NFBU) adopted an emperical formula for recommending to cities the amount of Fire Demand to build into their systems. For cities less than 5,000 persons the demand flow determined by the formula was to be sustained for a 2.5 hour period and for cities over 5,000 persons the time requirement was established at 5.0 hours.

N.F.B.U. Emperical Formula

$$Q = 1{,}020 \quad P\ (1 - 0.01 \quad P)$$

Q is in gallons per minute
P is population in 1,000's

Utilizing the above formula the Fire Demands for Pendleton would be as shown in Table 33 below.

TABLE 33

FIRE DEMANDS BY N.F.B.U. FORMULA

Population Design	Flow Required In GPM	Duration In Minutes	Total Gallons Required In Millions
15,000	3,797	600	2.278
20,000	4,358	600	2.61
25,000	5,097	600	3.06

Several leading authorities over a period of time have taken exception with the fire flows recommended by the N.F.B.U. Emperical formulas developed by others result in 70 - 80% of the fire flows required by the N.F.B.U. formulas. Over the years several cities that have opposed providing of fire demands recommended by the N.F.B.U. have made their political weight felt. In recent years the N.F.B.U. was re-named the Insurance Services Office and a new fire demand schedule was adopted. The inability of a City to provide the recommended fire demand has its primary impact on the owners of large buildings seeking insurance coverage. The current fire demand schedule adopted by the Insurance Services Office (ISO) is shown on Table 34.

TABLE 34

ISO REQUIRED FIRE FLOWS AND DURATION

Required Fire Flow - GPM	Required Duration Minutes	Total Gallons Required In Millions
2,500 and less	120	0.30
3,000	180	0.54
3,500	180	0.63
4,000	240	0.96
4,500	240	1.08
5,000	300	1.50
5,500	300	1.65
6,000	360	2.16
6,500	360	2.34
7,000	420	2.94
7,500	420	3.12
8,000	480	3.84
8,500	480	4.08
9,000	540	4.86
9,500	540	5.13
10,000 or more	600	6.00

In Table 33 we find under the old formula that for a population of 20,000 persons a fire flow of 4,358 gpm was required for 10 hours or a total standby storage reserve of 2.61 million gallons. With the new Table 34 a slightly higher flow of 4,500 gpm is required but with a reduced duration of 5.0 hours the total storage reserve is only 1.08 million gallons. For a 5,097 gpm flow for a population of approximately 25,000 persons, we find on the new chart that a similar 5,000 gpm on the new chart requires a reserve storage of only 1.65 million gallons versus 3.06 million gallons of reserve under the old formula.

Early in the study the cost of constructing a water system to meet the fire flows recommended by the ISO was discussed with the City staff. Several structures with large fire demands are situated in the upper elevations of the low level system and in the high level systems with limited flow delivery capabilities. These structures were identified in Chapter II, page 25. We requested that as a matter of City policy the City staff determine what they felt would be a reasonable fire flow for the City to provide in these identifiable problem areas. The City staff decided that we incorporate a fire flow of 2,000 gallons per minute be provided above the maximum anticipated day demand conditions for these areas. In some of the lower areas of the City's low level system, available fire flows in excess of 15,000 gallons per minute have been recorded.

C. TOTAL DEMAND VERSUS TOTAL SUPPLY.

We contacted Mr. Jim Schwager of the Oregon Insurance Services Offices for the recommended fire flows for the City of Pendleton. We discussed the concept of the City providing 2,000 gpm fire flow in addition to maximum day demand flows for the problem structures as discussed earlier. He stated that other cities have taken a similar approach in the interest of economics and said the City of Pendleton may wish to consider adopting an ordinance covering the types of structures, their size, height, use, and sprinkler protection in the identifiable problem areas. Based on his 1973 survey of the City he felt that 4,000 gpm for four hours would be adequate for existing structures, and that a design fire flow of 5,000 gpm for five hours would be a generous amount for the foreseeable future. In computing the fire demand he suggested that fire demands be determined by selecting from three possible circumstances the one that would require the greatest reserve. These conditions were as follows:

1. The largest single well source being out of service for five days;

2. A break or rupture in the gravity line requiring three days to repair;

3. Or a localized power outage (control panel or a transformer) for a duration of two days and the effect of this condition on sources and booster stations.

The third condition would have its major impact on the high level service areas. The North Hill high level service area is serviced by a 300,000 gallon reservoir placed on high ground and is serviced by two booster pumping stations. The Airport is serviced by a booster station on the old airport road, a 500,000 gallon ground level reservoir at the Airport, and a booster station adjacent to the reservoir equipped with standby power to pressurize the system. The South, Southwest, Mt. Hebron, and the S. E. 20th Street and the S. E. Court Avenue areas are served by individual booster stations. A power outage of two days duration in any of these areas would place a substantial number of customers without water or a means of suppressing a major fire. A remedy for the South and Southwest areas is discussed later on in this chapter.

An analysis of supply requirements for conditions No. 1 and 2 recommended by the Oregon Insurance Offices and supply requirements for the maximum demand day are shown in Table 35 on page 132.

TABLE 35

TOTAL SUPPLY VERSUS TOTAL DEMAND IN MGD

	AVAILABLE SUPPLY						DESIGN DEMAND		
Design Population	Gravity Supply	Well Supply	60% of Storage	Loss of Largest Well	Loss of Gravity Line	Available Net Supply	Fire Demand	Maximum Day Demand	Total Combined Demand
15,000	2.582	12.312	2.28	3.024	--	14.150	0.96	9.906	10.866
20,000	5.250	12.744*	2.28	--	5.250	15.024	1.50	13.208	14.708
25,000	5.250	15.480**	2.28	--	5.250	17.760	1.50	16.509	18.009

* Assignment of Mission Well to serve Agency = loss of 1.152 MGD; plus the addition of the New Hospital Well at present capacity of 1.584 MGD. Net gain 0.432 MGD.

** Increasing yield of new Hospital Well to 3,000 gpm.

If the Brogoitti Well and the new Hospital Well were both reconstructed to be able to pump at the rate of 3,000 gpm (4.32 MGD) each and added to the supply system they would provide supply capabilities far in excess of the combined maximum day and the recommended fire demands for a population of 25,000 persons contemplated in this study.

With the exception of the Southwest area all the other existing high level systems supporting highly developed areas have direct access to a ground level reservoir for their pressurized systems or are served by a high level gravity reservoir. On this basis storage for fire demand in the Southwest high level area can be justified.

D. STORAGE REQUIREMENTS FOR SOUTHWEST HIGH LEVEL SERVICE.

Based on the prior established criteria that fire flows provided by the City shall be 2,000 gpm above the anticipated maximum day demand, the amount of required storage from Table 34 for a fire demand of 2,000 gpm is 300,000 gallons. Based on the premise that the reservoir is 60% full as was assumed in Table 35, the total storage required would be 500,000 gallons. The proposed reservoir would be constructed as a ground level reservoir at the present City owned site below the Community Hospital. The existing booster pumping equipment at S. W. 25th and S. W. Ladow Avenue, could be relocated to this site with the present pumping capacity increased to provide a fire flow of 2,000 gpm in addition to the estimated maximum day demand.

Computer analysis of the distribution system under a variety of loading conditions documented that by artificially raising the level of the Southwest Reservoir the amount of water withdrawn from the North Hill Low Level Reservoir during high demand periods was correspondingly reduced. Based on this information it would seem desirable to artificially raise the level of the South Hill Reservoir and construct the new proposed reservoir for the Southwest area as high as possible without imbalancing the distribution system or incurring excessive costs. This may, however, increase the pumping head of all the deep wells and increase the energy costs during the summer high demand periods.

If the design were to be based on 25,000 persons and the Brogoitti Well was not acquired and developed, a reservoir with a total capacity of 1,000,000 gallons that would serve both the low level and the high level area could be justified. This is based on incorporating the deficiency of 249,000 gallons shown in Table 35 for a population of 25,000 persons. Again assuming that 60% of the total storage is available for meeting demands the additional storage required would be 415,000 gallons. The providing of incremental storage from 500,000 to 1,000,000 gallons would cost far less than the redevelopment of the Brogoitti Well and the required mile of transmission line.

The Brogoitti Well with its high specific capacity (yield/ft of drawdown) is capable of withdrawing water from the underground reservoir economically and would also have the ability to accept artificial recharge water readily. The value of having a 16" waterline fronting the Industrial Park and ability to service additional tenants should be evaluated prior to arriving at a final decision on the alternatives of increasing the storage capacity of the proposed Southwest area reservoir versus acquisition and development of the Brogoitti Well.

E. RESERVOIR - HYDRAULIC BALANCING.

Presently the South Hill and North Hill Low Level Reservoirs provide service to the basic distribution system. Unfortunately, the North Hill Low Level Reservoir is only 0.66 feet (8") lower than the South Hill Reservoir which receives the incoming water from the Thornhollow Springs. This differential in elevation may have been adequate when these reservoirs were initially constructed and the North Hill water demands were low. To provide enough of an energy gradient to transfer an adequate amount of water by gravity from the South Hill Reservoir to the North Hill Low Level Reservoir to meet present water demands , the North Hill Low Level Reservoir would have to be approximately four feet lower than the South Hill Reservoir. When the present system demands become relatively high, water is withdrawn from both reservoirs but water in the South Hill Reservoir is constantly being replenished by the incoming gravity line. In various computer runs during periods of high demands which included fires at different locations, the North Hill Low Level Reservoir depleted rapidly.

To overcome this problem of being unable to fill the North Hill Low Level Reservoir, the City installed a booster pumping station at N.W. 5th Street and N.W. Horn Avenue over 40 years ago. The booster pump used to run only in the summer months between 10:00 p.m. and 8:00 a.m. to fill the reservoir. Now with the additional growth on the North Hill, the booster pump (original equipment) runs continuously in the summer months.

It is our recommendation if after the extension of the 16" transmission in N.W. 5th Street to N.E. Furnish does not materially relieve the need for pumping, that the City explore the feasibility of correcting the present hydraulic reservoir imbalance by:

1. Investigate closing some of the system valves during the summer months coupled with the use of the Byers and the Stillman Well.

2. By placing a short standpipe over the inflow pipe into the South Hill Reservoir during the winter months or whenever the deep wells are not required and restrict the flow out of the South Hill Reservoir to outflow only.

3. Experiment with the short standpipe at the South Hill Reservoir in the summer months and evaluate the hydraulic effect on the deep wells.

This arrangement would still permit the South Hill to accept excess water and overflow as it has in the past but during periods of high demands it would direct more water directly into the distribution system and during periods of moderate demand it would replenish the North Hill Reservoir. This would require a float operated valve in the North Hill Low Level Reservoir to prevent overflowing of this reservoir during periods of low demand. Should such hydraulic modifications prove to remedy this long-standing problem, the booster situation at N. W. 5th Street and N. W. Horn Avenue could be returned to standby service, and only be operational during periods of unusually high demands, or possibly eliminated.

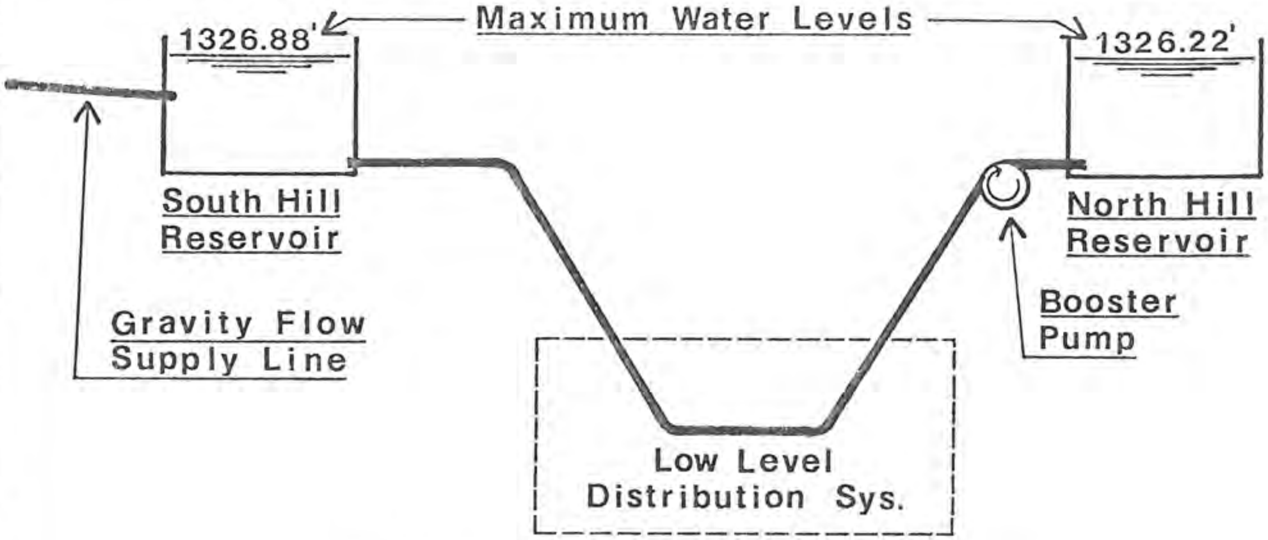
The supply to the North Hill Low Level Reservoir is entirely dependent upon the continuous operation of the booster pumping station at N. W. 5th Street and N. W. Gilliam Avenue. The filling of the North Hill High Level Reservoir is partially dependent on being supplied from booster pumps at the North Hill Low Level Reservoir. Any improvements to effect a better hydraulic delivery to the North side would enhance system reliability and minimize the risk of inadequate supply because of a failure at the present booster station at N. W. 5th Street and N. W. Gilliam Avenue. A schematic sketch on page 136 shows the existing and recommended hydraulic modification that could be implemented on a trial basis.

F. SERVICE TO THE SOUTH AND SOUTHWEST HIGH LEVEL AREAS.

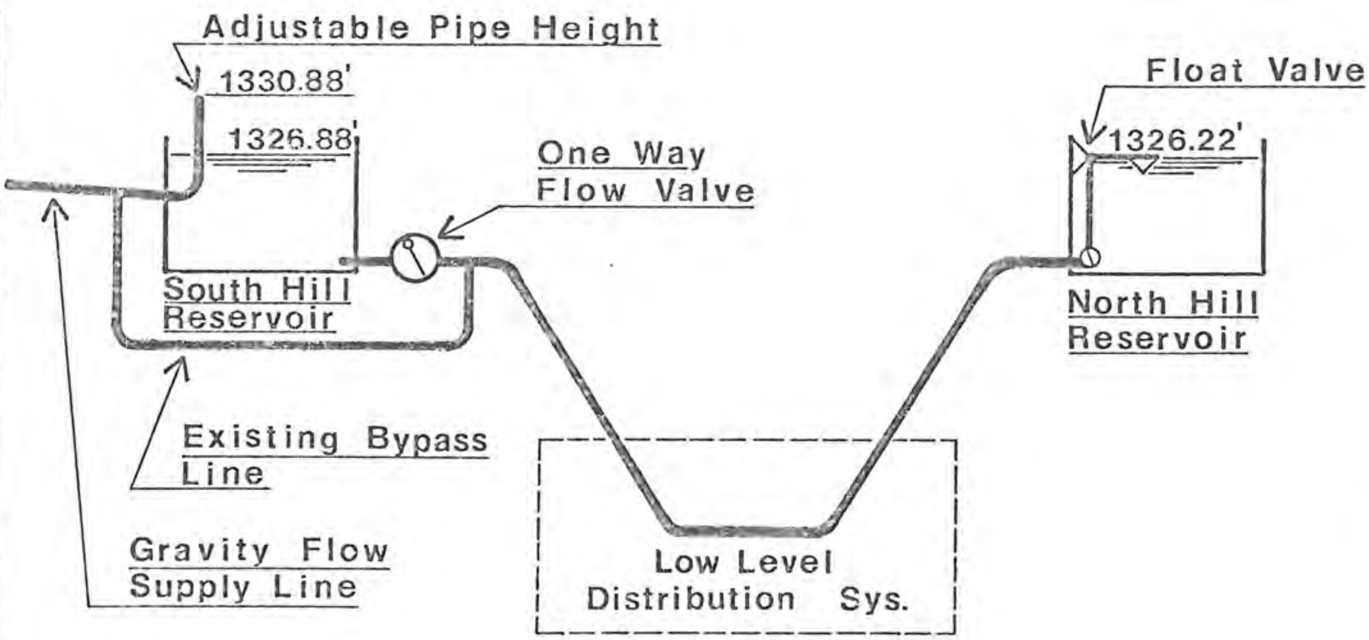
With most of the future growth being anticipated to occur in the South and Southwest areas, it would be desirable to interconnect the high levels with a 12" waterline. This would require the construction of approximately 3,000 feet of a high level service waterline through the low level system that would not be of use to the low level system because of excessive pressures. On the map on page 137 the areas requiring high level service have shaded backgrounds.

The benefits to be obtained from this intertie line would be that the one variable speed pump presently at S. W. 25th Street and S. W. LaDow could provide all of the low to moderate demands. This would eliminate the need of having a constant running pump in the booster station at S. E. 7th Street and S. E. Isaac Avenue. With this interconnection and the low probability of both stations being out of service at the same time, the reliability of water service to both of these areas would be materially enhanced. This would also provide the distribution system with the flexibility to respond to future significant growth that may occur in either the South or the Southwest section of the City. This solution is considerably more economical than constructing elevated storage facilities to provide comparable reliable fire protection in each area separately.

SCHEMATIC DIAGRAMS
of
LOW LEVEL RESERVOIR SYSTEM

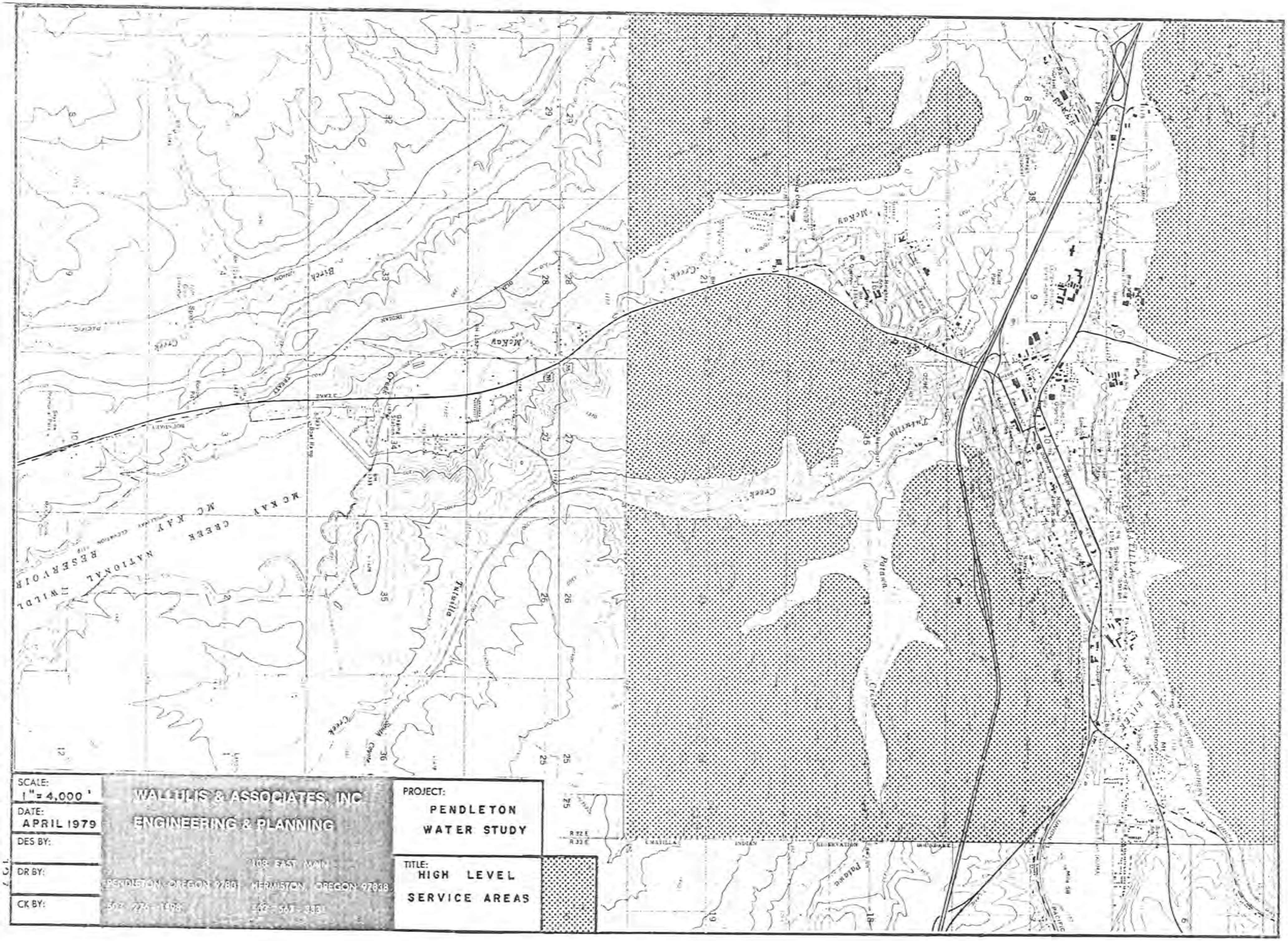


EXISTING SYSTEM



MODIFIED SYSTEM

PROJ. Pendleton Water Study	WALLULIS & ASSOCIATES INC ENGINEERING & PLANNING 5 S.E. NYE 108 E. MAIN PENDLETON HERMISTON 276-1498 OREGON 567-8331	SCALE NONE	NO. 136
TITLE Reservoir System		DATE April 1979	



SCALE: 1" = 4,000'

DATE: APRIL 1979

DES BY:

DR BY:

CK BY:

WALLULIS & ASSOCIATES, INC.
ENGINEERING & PLANNING

108 EAST MAIN

PENDLETON, OREGON 97801 HERMISTON, OREGON 97838

PROJECT: PENDLETON WATER STUDY

TITLE: HIGH LEVEL SERVICE AREAS

G. AIRPORT HIGH LEVEL SERVICE AREA.

The Airport Industrial Park study indicates the tenants that may choose this location would require little water for their normal operations but because of structure sizes may require unusually large fire flows. With the remoteness of the Airport from the rest of the system the only economical way of meeting these special fire needs would be the construction of additional storage facilities at the airport. The reconstruction of transmission lines and the booster station to meet this limited short term fire demand could not be economically justified on the basis of revenue through use.

An intertie line is proposed to be connected from the Airport high level system to the low level system directly above the Hill Furniture store. At this point of connection a pressure reducing valve would be installed to permit the flow of water from the upper system for fighting fires.

H. SUMMARY.

Even with the improvements suggested in this report there will be residential units that will still have to continue to use individual booster pumps to have adequate pressure for normal domestic uses and irrigation. It is difficult to economically justify the cost of a substantial booster pumping station for ten to twenty customers on the various fringes of the community. Should substantial growth occur in any of these areas, the need for the construction of one or more additional booster stations should be at that time evaluated.

CHAPTER VIII

WATER DISTRIBUTION SYSTEM

A. EXISTING SYSTEM.

Over the years the City has gradually converted the old pipelines made of various materials to cast iron pipe and more recently to ductile iron pipe which is an improved form of cast iron pipe. There remains approximately two blocks of old steel pipe in the present system.

Cast iron is an excellent pipe material well known for its strength and long service life. There are some soils which are highly corrosive and materially reduces the life of cast iron pipe. The Cast Iron Pipe Research Association (CIPRA) provides at no cost, soil evaluations for cities using or planning to use cast or ductile iron pipe. The City of Pendleton has had a soil survey performed by CIPRA and they have identified limited areas where special protection of the pipe will be required. In the areas identified as highly corrosive to ductile iron pipe, corrosion is arrested by encasing the pipe in a low cost light weight polyethylene wrap. The continued use of this pipe material, properly protected, should provide the City with an excellent water system with a long service life and reliability.

The City water department's revenue has remained relatively constant for several years while inflation was taking its toll in increased salaries, materials, power costs, and equipment replacement. With the shrinking purchasing power of the dollar, some of the manpower assigned to routine maintenance were reassigned to more pressing demands. This has resulted in the allowing of some of the system monitoring equipment to fall into disrepair and eliminating the progr am of annually exercising system valves. The exercising of system valves is important to keep them from corroding into a fixed position, thereby becoming inoperable. It is also desirable to periodically verify that they have not been accidentally left in the closed position. During this study, valves that should have been open were found closed. Valves left unattended for long periods of service can also develop substantial leaks around stuffing gland seal on the operating stem. The functional reliability of the valves are the very heart of the distribution system. Unfortunately the recent increase in water rates did not provide sufficient revenues to cover the City's cost participation in the oversizing of City mains in new developments, and reassignment of personnel back to the routing maintenance program. A good valve maintenance program also helps to lower the deficiency points assigned by the Oregon Insurance Services Office, which in turn affects the determination of fire insurance rates.

B. COMPUTER PROGRAM ANALYSIS OF WATER DISTRIBUTION SYSTEM.

The computer equipment utilized for the Pendleton water distribution system was an IBM 370148 system owned by the Clark County (Washington) PUD. The program has been used extensively and successfully by the Water Department Engineer, Claire Tittle, in the development and analysis of the Clark County water system over the past several years. The program was compiled by Professor Donald J. Wood, P.E., Department of Civil Engineering, University of Kentucky, and is being used extensively by municipalities and consultants for water distribution system analysis. This program provides a print-out showing the residual pressures at all pipe junctions and the velocity of the water in each individual line. Once set up, a typical system of 180 pipes takes only 12-15 seconds to analyze the entire distribution system under assigned specific load conditions.

C. AVAILABLE WATER SYSTEM DATA.

There are presently two pressure recorders in the distribution system, and both of them have been inoperative for several years. One of the pressure recorders is located in the Stillman Well No. 5 and the other is located at the booster station at S. W. 25th Street and S. W. Ladow Avenue. Without any information on actual system pressures it was necessary to estimate the distribution of water use throughout the entire system. As a part of this study the City has purchased three additional pressure recorders to be installed at the following locations:

1. Sherwood Heights School area

2. Northwest 12th Street and N. W. Horn Avenue booster station

3. The Airport Road and Old U. S. Highway No. 30

These additional recorders and the repair of the two existing recorders will provide some actual data to verify the accuracy of the estimated distribution of water use. It is our recommendation that after the data has been collected next summer that the computer analysis be re-run based on actual information gained from the pressure recorders.

D. ESTIMATED DISTRIBUTION OF PIPE FLOWS.

For the basis total flow loadings we utilized the highest monthly demand on record for the last 12 years. During July of 1970 the average total daily demand, including unaccounted for water, was 9.474 MGD and the population was 13,197 persons. Adjusting this rate of demand to a population of 15,000 persons results in an average daily demand of 10.77 MGD. This adjusted average day demand was multiplied by 1.2 to estimate the maximum demand day during the maximum month. This resulted in an estimated system demand of 13.00 MGD for a population of 15,000 persons, 17.95 MGD for 20,000 persons, and 22.41 MGD for 25,000 persons.

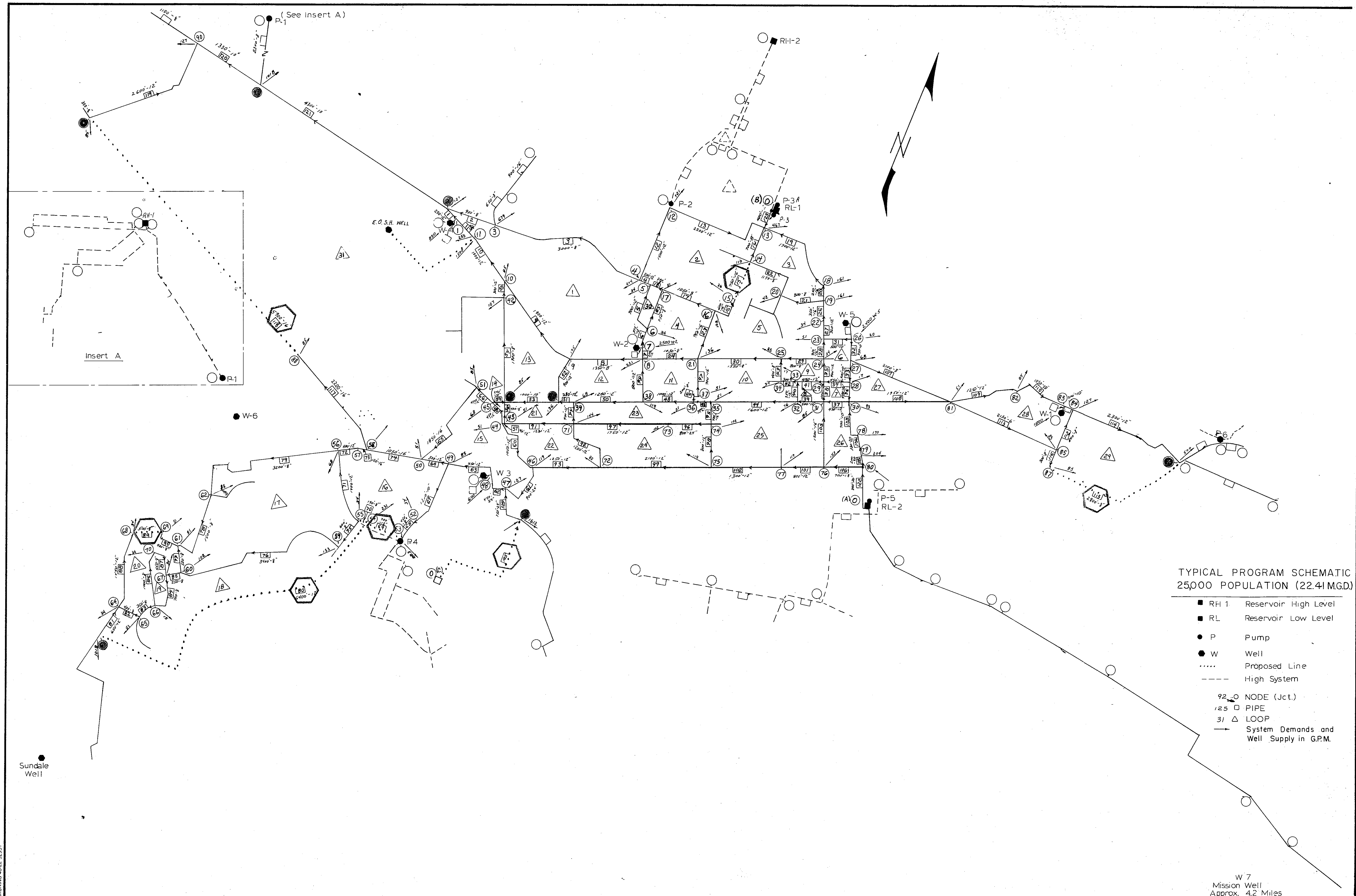
The distribution of the above determined maximum flows were heavily oriented to the land area served in each segment of the City. Summer irrigation places the heaviest hydraulic loadings on the distribution system, and the area and water consumption relationship should be reasonably proportional. Where large users were evident or major growth was anticipated to occur, the assigned flows were modified.

A schematic map of the distribution system on page 142 was especially prepared for the computer analysis. Pipe lines 6" and smaller were not included in the system analysis as their primary function is to meet localized demands and are of little value in the mass transfer of water. The information shown on the schematic map includes pipe sizes, lengths, pump stations, wells, reservoirs and the water demand at each pipe junction (node point).

The information on the schematic map and the elevation of each node point were placed into a computer program format for design populations of 15,000, 20,000 and 25,000 persons. The initial computer runs were based on the estimated maximum day demands and in successive computer runs additional fire demands of 2,000 gpm in different areas were imposed to determine the effect on the distribution system. The resulting printouts from the computer indicated present and future problem areas of low residual system pressures and excessive pipeline velocities. With several problem areas identified we prepared a proportional graphic map of the existing water system as an additional planning tool to determine the best approach to remedying system deficiencies.

E. PROPORTIONAL GRAPHIC MAPPING.

A Proportional Graphic Map of the existing water system was prepared to visually evaluate the relative capabilities of various pipelines to transmit water. Water lines smaller than 6" in diameter primarily service local demands and are of little value in mass transfers of water within the distribution system. The thickness of the water line shown on the map on page 143 relates to its relative ability to transmit water (e.g., lines twice as thick will carry twice as much water). With a proportional graphic map the bottlenecks can be readily found and areas that need reinforcement identified. This map also provides a check on the reasonableness of results obtained from a computer analysis. This map served as a basic planning tool in determining where the problem areas could be expected and how these system deficiencies might be corrected.



TYPICAL PROGRAM SCHEMATIC
25,000 POPULATION (22.41 M.G.D.)

■ RH 1	Reservoir High Level
■ RL	Reservoir Low Level
• P	Pump
● W	Well
·····	Proposed Line
– – – –	High System
92 ○	NODE (Jct.)
125 □	PIPE
31 △	LOOP
⟶	System Demands and Well Supply in G.P.M.

WALLULIS PROJECT NO: 78-107

AGENCY PROJECT NO:

PROJECT: PENDLETON OREGON WATER STUDY

WALLULIS & ASSOCIATES, INC.
ENGINEERING & PLANNING
108 EAST MAIN
PENDLETON, OREGON 97801 HERMISTON, OREGON 97838
503-276-1598 503-567-3331

SCALE: 1" = 1600'
DATE: Dec 1978
DES BY:
DR BY:
CK BY:

TITLE: COMPUTER ANALYSIS

REVISIONS

DRAWING: 142

SHEET OF

EQUIVALENCE TO 12" PIPE

DIAMETER	NO. OF PIPES
6"	0.25
8"	0.44
10"	0.69
12"	1.00
14"	1.36
16"	1.78
18"	2.25
20"	2.78
24"	4.00

LEGEND

- ■ RH 1 - Reservoir – High Level
- ■ RL 1 - Reservoir – Low Level
- ● P 1 - Pump
- ● W 1 - Well
- —— Low Level Pipe Lines
- - - - High Level Pipe Lines

WALLULIS PROJ. NO. 78-107

AGENCY PROJ. NO.

PROJECT

CITY OF PENDLETON, OREGON WATER STUDY

WALLULIS & ASSOCIATES, INC.
ENGINEERING & PLANNING

SCALE As Shown

DATE April 1979

DES BY

DR BY

CK BY

TITLE

Distribution System

REVISIONS

DRAWING 143

SHEET OF

With the aid of the proportional graphic map and the results of the computer analysis, a conference was held with the City staff. The initial problem areas identified from the computer analysis were as follows:

1. An 8" main on S. W. 6th at Emigrant, connecting 16" and 12" mains.

2. The need for connecting the 16 and 12" mains along N. W. 5th Street north of N. W. Carden.

3. The inability of the North Hill Low Level Reservoir to be filled by gravity from the South Hill Reservoir, due to the lack of elevation differential between the South Hill Reservoir (water elevation $1326\frac{9}{2}$) and the North Hill Reservoir (water elevation 1326^{2}).

4. That during periods of water demands on the North side of the distribution system the ability of the North Hill Low Level Reservoir to meet these demands and provide water to the North Hill High Level Reservoir was marginal even with the continuous running of the booster station at N. W. 5th Street and N. W. Horn Avenue. In summer months with the booster pump and the deep well pumps running, the present distribution system does allow the North Hill Reservoir to fill.

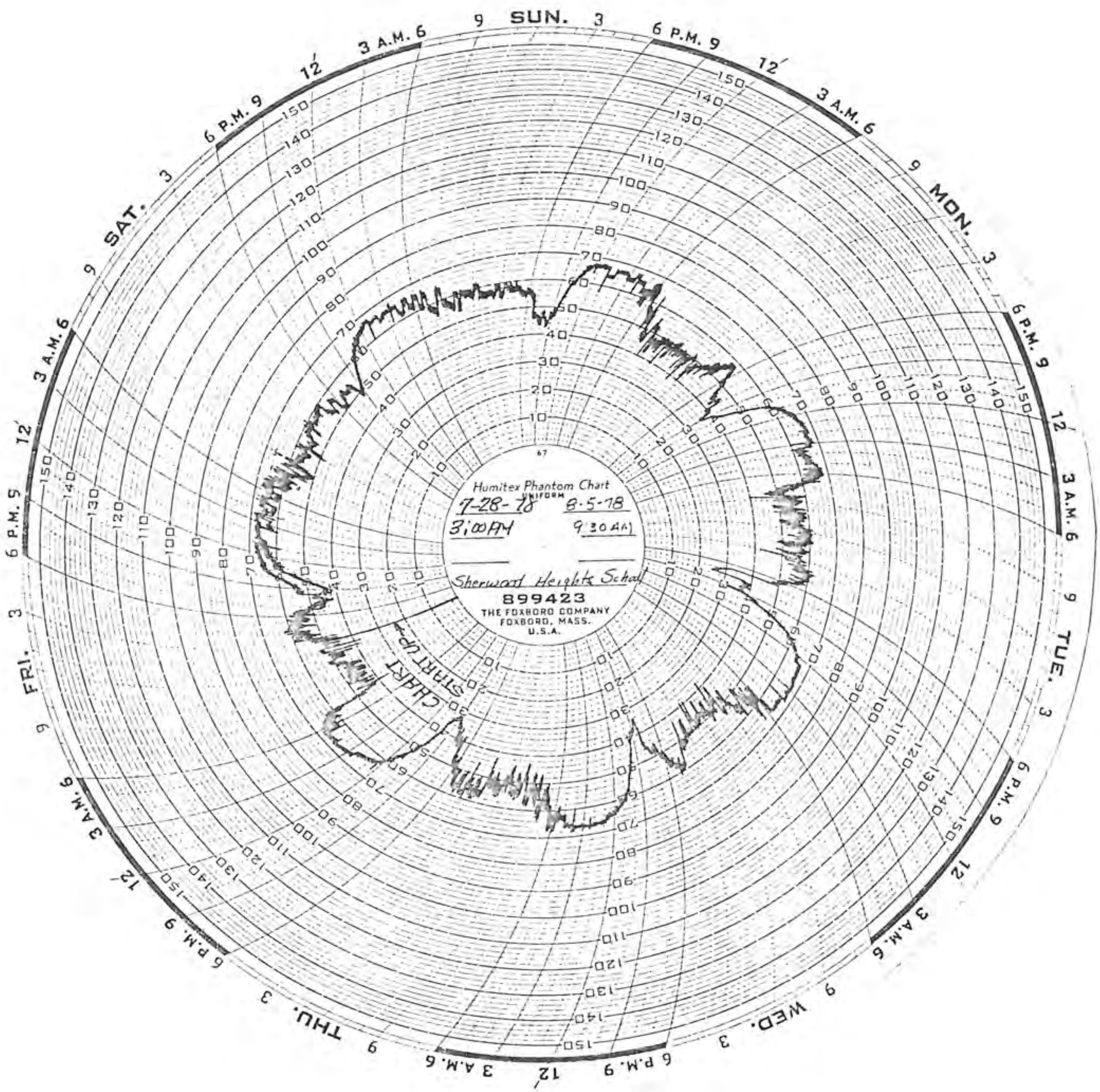
5. The inability of the present distribution system to provide the recommended fire demands in the vicinity of St. Anthony Hospital and the commercial area at the junction of the old Oregon Trail Highway and Oregon Highway 11 was identified as another area of concern.

6. Other areas with high fire demands and inadequate transmission capabilities in the present distribution system were:

   a) Airport
   b) Westgate - Airport interchange
   c) High School and Blue Mountain Community College
   d) Reith Industrial Park

7. Low pressures and inadequate fire flow capacities in the vicinity of S. W. 28th, S. W. 30th and S. W. Marshall. The results of one week's actual record of pressures is shown on the pressure chart on page 145.

8. The inability of the present distribution system to deliver an adequate supply of water to the Southwest section (Montee - Edwards) of the



RECORDED PRESSURES
at
SHERWOOD HEIGHTS SCHOOL

Note large pressure drops between 6:00 and 10:00 p.m. on Monday, Tuesday, and Wednesday.

City, without incurring excessive velocity and energy (pressure) losses. Correcting this deficiency would reduce the pressure losses occurring in the Sherwood Heights School vicinity.

Alternative remedies were discussed with the City staff to each of the above problem areas. Through these conferences with the City staff, the number of alternatives were reduced and the remaining alternatives were subjected to computer analysis.

The first computer run verified that to permit the mass transmission of water to the North side, the obvious bottleneck at S. W. 6th Street and Emigrant must be rectified by connecting the 16" main on S. W. 6th Street to the 12" main on S. W. Emigrant Avenue.

It was also apparent in the first computer run that to complete the mass transfer of water to the North Hill Low Level Reservoir that the existing 16" water main in N. W. 5th Street be initially extended northerly on N. W. 5th Street to the 8" main line at the intersection of N. W. 5th Street and N. W. Furnish Avenue. After this improvement has been completed the need for installing additional 16" line northward on N. W. 5th to N. W. Horn Avenue should be evaluated.

The third and fourth identifiable problems of providing additional supply to the North Hill Low Level Reservoir during high demand periods presented a more difficult situation with several possible alternatives available. At first, methods of reducing the losses by adding another cross town transmission water line from the South Hill Reservoir to the North Hill Low Level Reservoir was investigated. With the downtown corridor developed to the extent that it is, the undertaking of constructing another major inter-connecting pipe system was not considered to be an economically viable solution. Earlier studies recommended the abandonment of the present North Hill Low Level Reservoir and the construction of a new reservoir approximately four feet lower at the same location or on land nearby. A new North Hill Low Level Reservoir at a lower elevation would have the ability of being filled from the South Hill Reservoir. Because of the large capital expense in constructing a new reservoir we are recommending that the City explore what hydraulic improvements can be made by partial system valving during the use of Stillman and/or the Stillman Well coupled with the artificial raising of the level of the South Hill Reservoir with a standpipe as suggested in Chapter VII, page 134. Any improvements that could be implemented would increase system reliability by reducing the reliance on the booster station at N. W. 5th Street and N. W. Horn Avenue. Should such efforts fail to improve the present system hydralics, or prove to be cost effective, the existing booster station which is over 40 years old should be updated and equipped with standby power generating capabilities.

The fifth area of concern was the general area encompassed between St. Anthony Hospital and the junction of the Old Oregon Trail Highway and Oregon Highway 11. It was generally recognized that the potential for additional development to the east of this area is limited because of the Indian Reservation boundaries. With this under consideration, the use of extensive source development in this area or the construction of major pipe networks were eliminated as a solution. The addition of a loop 8" water line (#115) along S. E. Court Place was added to the existing system and its effect analyzed. This completed loop greatly relieved the load on the existing 12" main (#114) along S. E. Byers Avenue and provided a second main serving the Mt. Hebron - Riverside area. To provide a minimum fire flow at 2,000 gpm to the commercial establishments at the junction of the Old Oregon Trail Highway and State Highway No. 11 requires the construction of the 12" loop line shown on the map on page 148. Also, if the water treatment plant is constructed at Site A, this end of the distribution system could be connected to the gravity line and provide an additional feed line into the low level system.

The Westerly and Northwesterly areas of the City were previously identified as the sixth area of general concern. The effect of adding the newly constructed well at the State Hospital was analyzed for the beneficial effect it would have upon remedying the flow deficiencies in this area. The existing 10" diameter water line along U. S. Highway 30 proved to be totally inadequate to receive or transmit additional flows to the areas of need. To provide delivery capabilities to this general area and the Southwest growth area would require the construction of the pipelines as shown on the map on page 148. These lines are tentatively sized large enough to transmit 3,000 gpm from both the new Hospital Well and the Brogoitti Well into the local area, the Southwest area, and the City low level system. The routing shown for the 16" water line from the Brogoitti Well to the distribution system eliminates the need of a freeway bore by installing the water line under the overhead bridges spanning the Old Oregon Trail Highway to Reith. This alignment would also be of benefit to future tenants at the Reith Industrial Park. For the short term solution we are recommending that the Airport high level system be connected to the low level system through a pressure reducing valve immediately north of Hill Furniture. With the construction of the proposed new lines the present pressure reducing valve servicing the Reith Industrial Park would have to be relocated to a position south of the Old Oregon Trail Highway to Reith. It is recommended that the tie line between the new State Hospital Well and the Brogoitti Well not be constructed until the population exceeds 20,000 persons and only if the Brogoitti Well can be reasonably obtained. It would also be prudent to defer construction until more accurate demand data is collected from a continuous pressure recorder to be installed in this area.

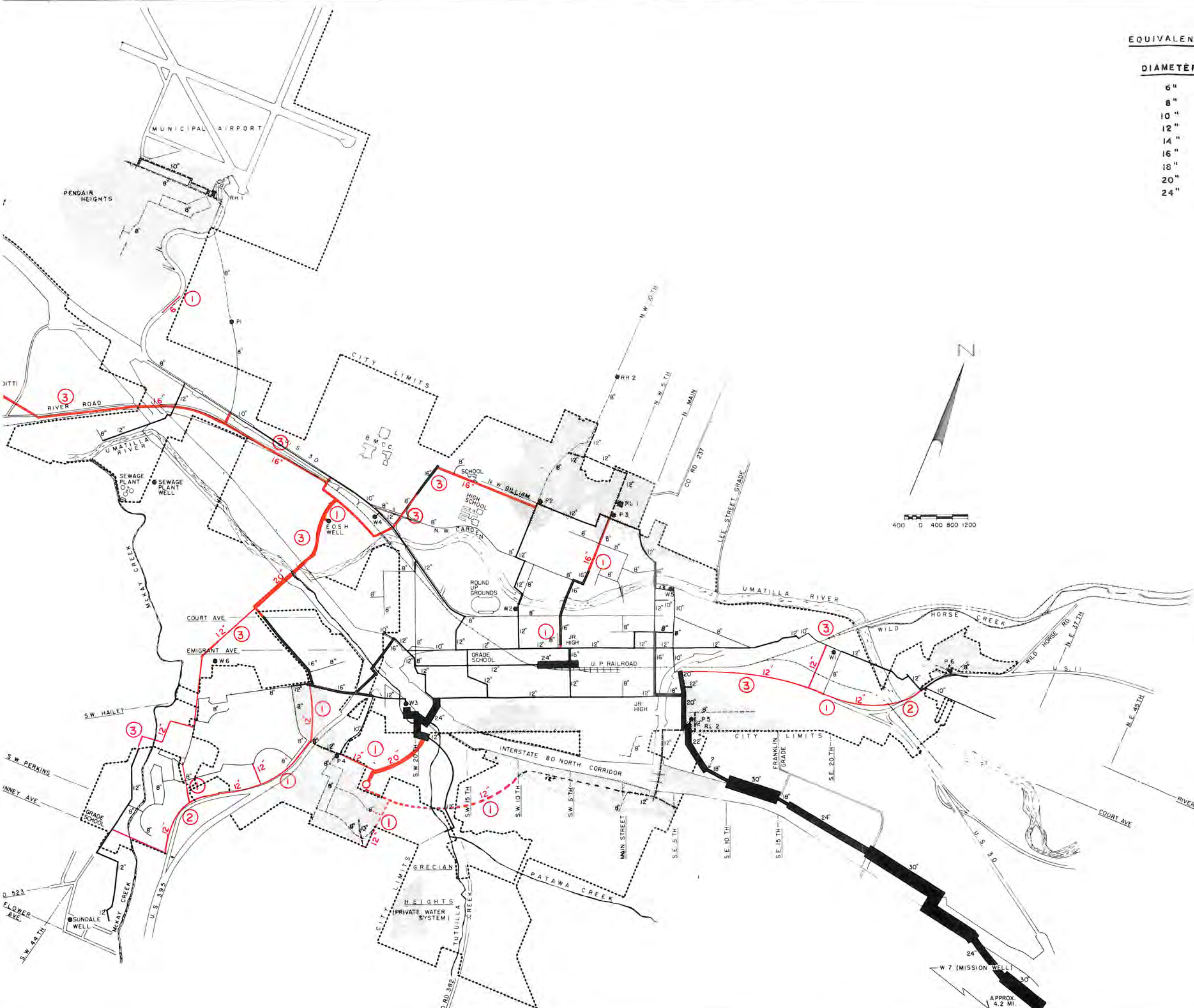
The last two areas of concern were improving the minimum pressure levels on the upper bench of Sherwood Heights in the vicinity of Sherwood Heights School and the supplying of an adequate water supply to the rapidly developing area along the McKay Creek Valley floor. These problem areas proved to be inseparable. On July 20, 1977 the Pendleton Fire Department conducted a hydrant test at the Sherwood Heights School and data showed a static pressure of 49 psi. With a fire flow of 1,110 gpm the residual pressure in the system was reduced to 20 psi. The recommended fire flow for the Sherwood Heights School is 2,500 gpm or 1,490 gpm more than was available. Based on a one week record of the actual pressure available at this location last year there were four days out of seven during the early evening hours when the available pressure was less than 40 psi and on one occasion during the week when only 25 psi of pressure would have been available. The location of the pressure recorder that recorded these pressures was installed in a janitor's closet on the main floor of the Sherwood Heights School. The pressure recorder at this location was 9 to 12 feet below the elevation of neighboring residences which means that the pressure in the water main in front of these residences was four to five psi less than that being recorded at the Sherwood Heights School. Considering the total pressure losses resulting from friction losses in the service line from the main to the water meter, losses through the water meter, and the friction losses in the service line from the water meter to the residence it is understandable why there have been complaints primarily from the men in this area not being able to have a decent shower in the evenings after work. It is also noteworthy that the consumption in the following month was substantially greater than the period observed. The need of the Sherwood Heights School to utilize the wash basin to which the pressure recorder was connected, limited the time of duration which pressures could be recorded. There is an obvious need for the placement of a continuous pressure recorder in the Sherwood Heights School area.

On October 24, 1973 the City Fire Department conducted a hydrant test at the McKay Creek School. The data from this test shows that the static pressure in the 12" water main was 110 psi, and with a fire flow of 2,010 gpm the residual pressure in the system dropped to 20 psi. The recommended fire flow for the McKay Creek School is 3,000 gpm or 990 gpm more than was available at the minimum pressure of 20 psi. With the additional growth that has occurred in this area in the last six years, the available fire flow would now probably be significantly less and particularly so if the test was conducted during the yard irrigation season. For the basis of a comparison, a test was conducted on the same day at the intersection of Main and Byers Avenue with the following results: the static pressure in the 6" water main was 109 psi; at a flow of 15,900 gpm the residual pressure would be 20 psi. This contrast illustrates the inability of the distribution system in the Southwest area to transmit appreciable quantities of water without incurring excessive pressure losses.

The addition of a new 12" water line (#80) to the distribution system servicing the McKay Creek area and computer analysis indicated the present

EQUIVALENCE TO 12" PIPE

DIAMETER	NO. OF PIPES
6"	0.25
8"	0.44
10"	0.69
12"	1.00
14"	1.36
16"	1.78
18"	2.25
20"	2.78
24"	4.00



LEGEND

- RH 1 - Reservoir - High Level
- RL 1 - Reservoir - Low Level
- P 1 - Pump
- W 1 - Well
- Low Level Pipe Lines
- High Level Pipe Lines
- Proposed S.W. Reservoir
- Project Priority

WALLULIS PROJ. NO. 78-107
AGENCY PROJ. NO.

PROJECT
CITY OF PENDLETON, OREGON WATER STUDY

WALLULIS & ASSOCIATES, INC
ENGINEERING & PLANNING

SCALE As Shown
DATE April 1979
DES BY
DR BY
CK BY

TITLE
Distribution System

REVISIONS

DRAWING
148

SHEET OF

deficiencies would be materially relieved but with additional growth would only be marginal. Unlike other areas of the community, there is an urgent need to provide a 12" diameter transmission water line to this area through one of the two routes shown on the map on page 148.

The alternative of increasing the pipeline to the Montee - Edwards area above 12 inches was considered but, based on data presently available, this would be difficult to justify. It is also desirable to divide the load to major areas with two smaller lines so that in the event one of the two lines by design or accident is removed from service, there still remains a reasonable level of protection for the residents of the area. The addition of the new Hospital Well into the water system, the proposed 20" intertie line (State Hospital to S. W. 28th Drive), and the connecting 12" water line (S. W. 28th Drive to S. W. 37th Street) as shown on page 148, would also provide a direct feed into this high demand area and materially improve pressure deficiencies in the Sherwood Heights area. The selected routing of the above 20" intertie line eliminates the need of a freeway boring by making the required river crossing underneath the existing freeway bridges.

F. PROPOSED SOUTHWEST RESERVOIR.

The need for the construction of a reservoir to service both the low and high level service areas in the Southwest area was established in the preceding chapter. The reservoir would be constructed on a site situated adjacent to the Community Hospital which is currently owned by the City. This reservoir would be constructed at ground level with booster pumps to provide pressure to the high level service area. The effect of placing this proposed new reservoir at different elevations was analyzed in the distribution system analysis. The computer analysis indicated that by placing the new reservoir at higher elevations the drain on the North Hill Reservoir was correspondingly reduced. A differential in elevation of approximately four feet is required between the South Hill Reservoir and the proposed Southwest reservoir to provide the energy to transfer the water.

G. SUMMARY.

The area in the Sherwood Heights area along S. W. 28th Street and S. W. 29th Street is presently being served by 6" water lines connected to the Southwest high pressure system. With the construction of the proposed improvements for this area shown on page 148 and the conversion of the present 12" water line (#69) from the high pressure system to the low level, this area should be capable of being served off of the low level system. The service pressure would be considerably less than presently available but adequate for normal domestic uses. These system modifications would eliminate the need for the pressure reducing valve presently situated at the intersection of S. W. 28th Street and S. W. 30th Streets.

It is our recommendation that as a follow-up to this study that funds be authorized to consider alternatives to the suggested routings of new water lines along with the preparation of detailed cost estimates. The City should also, as soon as possible, initiate steps to acquire the new Hospital Well by transferring the present agreement from the existing well to the new well. The yield from the old Hospital Well more closely fits the individual needs of the Hospital for their own use. A new agreement should also incorporate the right to purchase any available surplus water from the old State Hospital Well. Immediately after the City has a new agreement with the State Hospital the needed interconnecting transmission lines north of the new Hospital Well to the Old Oregon Trail Highway should be installed.

CHAPTER IX

FINANCING AND REVENUES

A. REVENUE HISTORY.

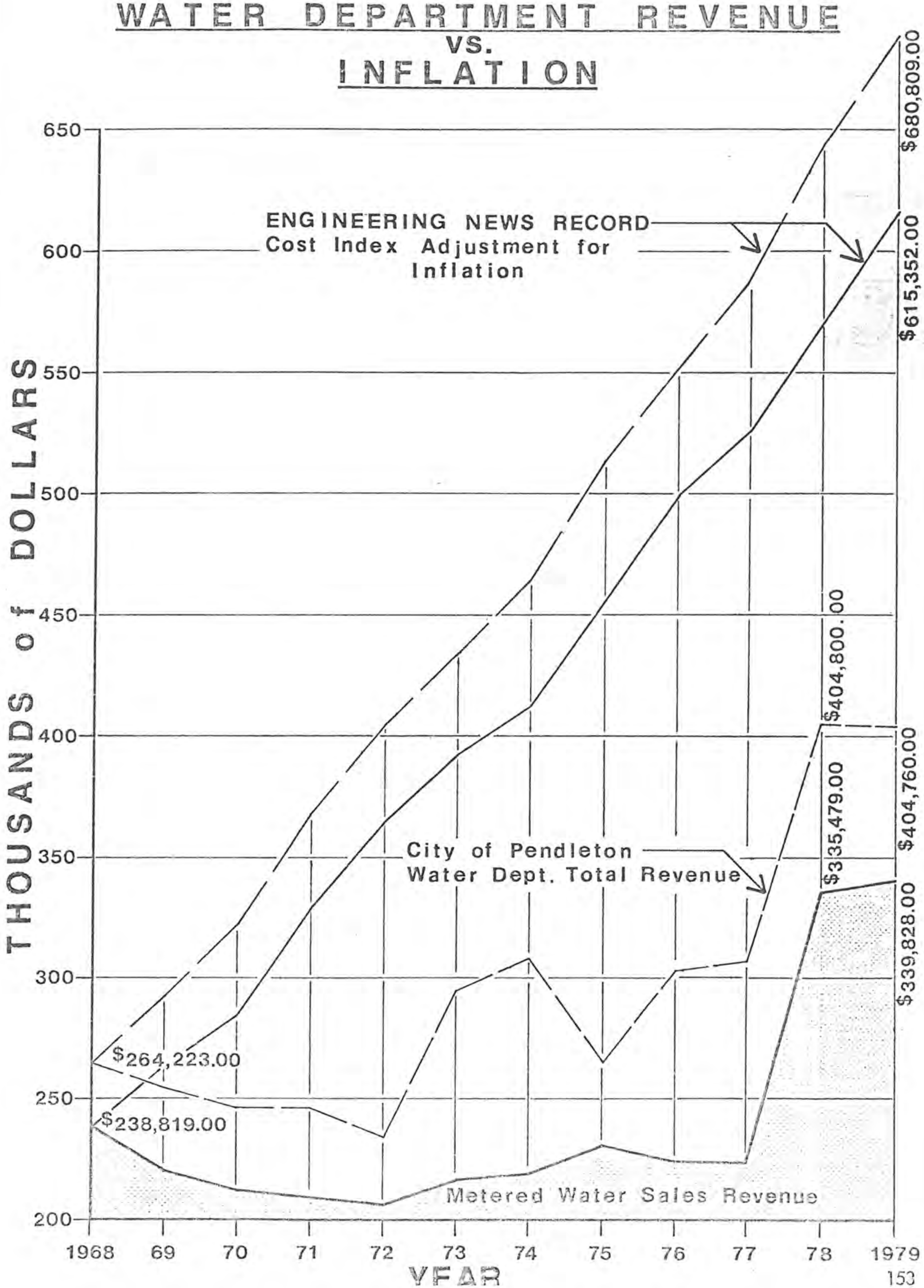
Over the years the City of Pendleton has endeavored to provide the residents of the City with an economical supply of water. Even through periods of inflation the water rates were consistently held low. For the time period of 1968 through 1978 we have prepared a graph that illustrates the effect of maintaining the unrealistically low rates have had on the Water Department. This graph on page 153 shows periods of declining revenues during times of high inflation. In the time period of 1973 and 1974 the City was a recipient of a federal grant for the installation of a water system for Mt. Hebron. The population in 1968 was 14,600 persons and in 1978 was 15,000 persons. An adjustment for the population increase over the year 1968 would have added $17,635 to the ENR Construction Cost Index Curve shown for the year 1978.

The ENR (Engineering News Record) construction cost index has a long established record of accurately reflecting construction costs. This cost index was initiated in 1921 and has gained broad acceptance by the professional community, and agencies of the state and federal government administering federal grants on construction projects. The Water Department functions of construction, repair, and replacement are compatible with the costs reflected in this index.

The City Water Department, suffering from both reduced revenues and inflation, had to curtail several routine but important maintenance functions. Even with the water rate increase in May of 1977 the revenue generated in 1978 is only 56.1% of the equivalent revenue in 1968. The additional revenue raised by the rate increase in 1977 has been to a large degree used for oversizing of water lines in new growth areas. Even without any of the major improvements contemplated in this study, the present water rates should be significantly increased.

B. WATER RATES.

Water rates should be based on the financial needs of each city. The quality of the raw water, population density, energy costs, proximity of water source, per capita income, extent and type of industry, climate, alternative irrigation water sources, maintenance programs and the level of service provided to the water customer are all factors that effect water rates which vary considerably from city to city. The yardstick of what neighboring communities charge for water should not enter into the establishment of a rate for any particular community.



WATER DEPARTMENT REVENUE
VS.
INFLATION
THOUSANDS of DOLLARS
650
600
550
500
450
400
350
300
250
200
ENGINEERING NEWS RECORD
Cost Index Adjustment for
Inflation
$680,809.00
$615,352.00
$404,800.00
$404,760.00
$335,479.00
$339,828.00
City of Pendleton
Water Dept. Total Revenue
$264,223.00
$238,819.00
Metered Water Sales Revenue
1968
69
70
71
72
73
74
75
76
77
78
1979
YEAR

To illustrate the diversity of water rates of different cities, we have prepared a chart on page 155. The two water districts shown on the chart purchase their water from the city of Portland, Oregon, and serve large suburban areas. All other cities have their own water systems. A water study is presently in progress for the city of Baker and the findings of this study will determine the amount of increase in their water rates. Because cities as a group tend to only increase water rates when an overwhelming need can be demonstrated, their relative position on a rate structure comparison chart can change dramatically when that need is finally recognized.

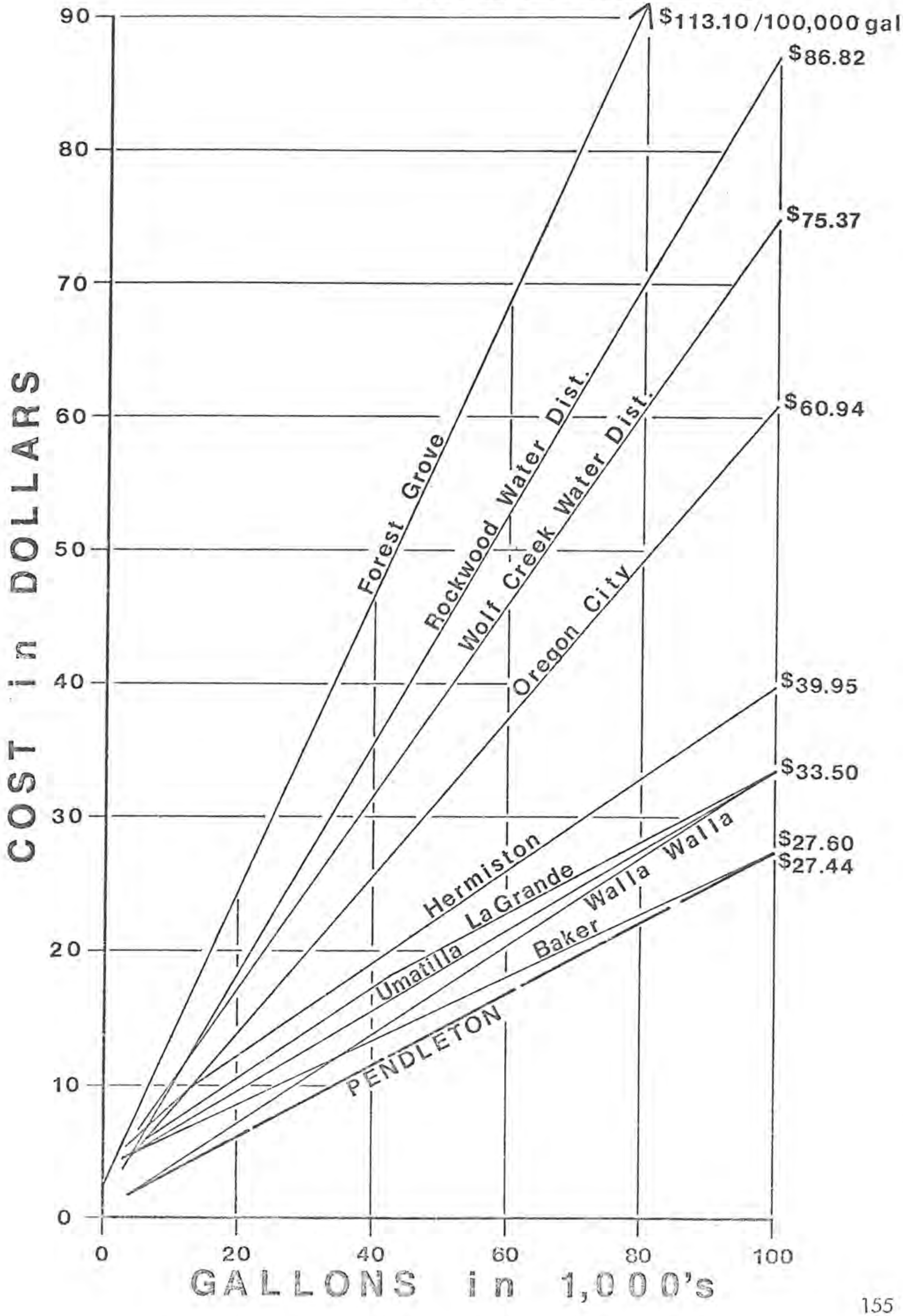
After the depression years of the thirties the providing of low cost water by cities to attract both industry and residents gained popularity. The concept of maintaining low water rates has "hung-on" and when practiced to extremes it is normally accomplished by exhausting whatever remaining life is available in the physical plant. Communities that practice this philosophy are faced with an overwhelming financial burden to completely restore these facilities.

Annual adjustment of water rates reflecting any change in the purchasing power of the dollar would enable the water department to sustain a uniform level of adequate maintenance, maintain a schedule of needed replacements, and incorporate those new technological advances that were cost effective.

Even though there are many favorable aspects to the concept of maintaining a uniform level of real income for a water department, it is rarely practiced. The only real barrier to the implementation of annually adjusting the water rate schedule is the effort put forth to maintain an on-going public information program to document the need, and support from the elected officials of the community.

The City of Pendleton is fortunate in that it has not fallen behind to the degree that a large expenditure is required to bring the basic facilities up to date. Increases in the water budget for the maintenance of an annual valve program, improved metering program, grounds maintenance at the springs, records keeping, individual well monitoring of the water table, maintenance of miscellaneous equipment and a more frequest analysis of total system performance are areas that need improvement. Restoration of the annual hydrant testing program should also be resumed as soon as possible.

WATER RATES COMPARISON



C. REVENUE REQUIREMENTS

A substantial increase in the water rates will be required for new debt service for the various capital improvement components, operational treatment costs, and restoration of funding to 1968 levels for normal maintenance. This information is presented herein without the benefit of an in-depth evaluation of the present physical plant, equipment, and personnel requirements.

To estimate the revenue required for annual debt service it is necessary to assume the interest rate, the number of years to retire the debt, and the gallonage sold. For the basis of this report we are assuming that bonds will be sold with a 25-year maturity schedule at an annual interest rate of 6%. The gallonage will be based on the projections developed in prior Chapters.

The capacity of the existing Thornhollow gravity line will provide a base flow of 5.25 MGD and with the proposed improvements will serve a population of 22,550 persons. Several other future improvements proposed in this report would not be required until this population level was exceeded. The basis of determining annual revenues developed in this Chapter will therefore be limited to populations up to 22,550 persons. Above a population level of 22,550 persons would require: additional surface storage; the expansion of the water treatment plant; increasing the capacity of the gravity waterline from the treatment plant to the distribution system; the construction of a river intake on the Umatilla River; a pumping plant at the River and a transmission line to the proposed treatment plant.

Once the amount of debt and the interest rate has been set, the annual repayment obligation remains fixed. During inflationary periods this has the advantage of debt repayment with the devalued dollars which progressively reduces the impact of debt on the rate structure, and the effect of debt is further diminished by an increase in the population. We have prepared in Table 36 below the amount of revenue needed in cents and decimals of cents per 1,000 gallons or 100 cubic feet of water sold for each million dollars of debt for different population levels.

TABLE 36

DEBT SERVICE REQUIREMENTS
25 year term @ 6% interest

Population	For each $1,000,000 of Debt Increase in Revenue Required	
	cents/1,000 gal.	cents/100 cu.ft.
15,000	6.07	4.55
20,000	4.55	3.41
22,550	4.04	3.03

In 1977-78 the City Water Department produced 1,558,351,000 gallons (includes unaccounted for water) and generated a total revenue of $335,479.00 from the sale of water. The revenue received amounts to $0.215/1,000 gallons ($0.161/100 cubic feet) produced. In 1977-78 the unaccounted for water was 22% of the total production. With the adjustment for the unaccounted for water the overall average rate to the customers was $0.276/1,000 gallons ($0.207/100 cubic feet).

The Water Department generated an additional $69,273.00 in 1977-78 from meter rents; new main and service line installations; sale of materials; interest and other miscellaneous charges. These income producing activities should be at least self-sustaining and as such are excluded from consideration in developing the required revenue from the sale of water to cover the costs of producing and delivering to the customer.

As stated earlier in the report, we are projecting a 20% reduction in the water production requirements for the future population projections. This was premised on at least a 10% drop in the amount of unaccounted for water and an additional 10% primarily because of the increased price of the water. If the amount of unaccounted for water was reduced by 10% this would either increase revenues or reduce the production costs. For a revenue base we are assuming that the revenue of $0.276/1,000 gallons received in 1977-78 would have remained relatively unchanged if a rate increase reduced use by 10% and reduced system losses would increase metered sales by 10%.

Some of the line items shown on Table 37 are, as stated earlier in the report, based on some rough "shotgun" estimates and that additional detail engineering estimates should be funded and authorized to study selected alternative solutions in detail prior to implementation. It is felt, however, that the estimated costs of the improvements shown in Table 37 are reasonably representative and are adequate for the Council to determine the relative financial impact of each of the proposed water system components.

There is not expected to be any capital investment required in the acquisition of the new State Hospital Well except for connecting transmission lines and modifications to the two Hospital wells. The option of providing on-site disposal of Wastewater from the water treatment plant should have little effect on the rates if the wastewater is properly utilized to enhance crop productivity.

In the interest of consistency with the balance of this report which has expressed demands and consumption rates in terms of gallons, the revenue requirements shown in Table 37 are expressed in terms of thousands of gallons. The City, however, bills to customers in terms of cubic feet. We have therefore shown the revenue requirements both in cents per 1,000 gallons and cents per 100 cubic feet.

TABLE 37

WATER SYSTEM IMPROVEMENT
COSTS AND REVENUE REQUIREMENTS

CATEGORY I
RECOMMENDED IMMEDIATE IMPROVEMENTS

Item	Cost In Millions	Revenue Required cents/1000 gallons and in cents/100 cubic feet, for populations of:		
		15,000	20,000	22,250
1. New S.W. Reservoir 1.0 MG & Re-located booster Station	0.580	3.52 - 2.64	2.64 - 1.98	2.34 - 1.76
2. Connecting piping for above reservoir	0.235	1.43 - 1.07	1.07 - 0.80	0.95 - 0.71
3. South - SW High pressure Level Intertie Line	0.099	0.60 - 0.45	0.45 - 0.34	0.40 - 0.30
4. Water Treatment Plant - Debt Service	2.118	12.85 - 9.63	9.63 - 7.22	8.56 - 6.42
5. Water Treatment Plant O & M	N.A.	10.00 - 7.50	10.00 - 7.50	10.00 - 7.50
6. Artificial recharge of underground aquifers	0.247	1.50 - 1.12	1.12 - 0.84	1.00 - 0.75
7. 12" Transmission line to McKay Creek area	0.100	0.60 - 0.46	0.46 - 0.34	0.40 - 0.30

CATEGORY I
(Continued)

8. Connecting Piping to utilize new State Hospital Well	0.086	0.52 - 0.39	0.39 - 0.29	0.35 - 0.26
9. 12" Water line S.W. 28th & S.W. 29th. Sherwood Heights	0.041	0.25 - 0.19	0.19 - 0.14	0.17 - 0.12
10. SE Court Avenue 12" Water line	0.036	0.22 - 0.16	0.16 - 0.12	0.15 - 0.11
11. 16" Water line connecting at S.W. 6th & S.W. Emigrant Ave.	0.006	0.04 - 0.03	0.03 - 0.02	0.02 - 0.02
12. 16" Water line on N.W. 5th Despain to Furnish	0.016	0.10 - 0.07	0.07 - 0.06	0.06 - 0.05
13. 6" Water line from airport system to airport interchange	0.010	0.06 - 0.05	0.05 - 0.04	0.04 - 0.03
14. 8" Intertie line through Cammie Addition	0.016	0.10 - 0.07	0.07 - 0.05	0.06 - 0.05
Subtotal	3.590	31.79/23.83	26.33/19.74	24.50/18.38

CATEGORY II

FOR POPULATION ABOVE 16,250 PERSONS

Item	Capital Cost	15,000	20,000	22,250
1. Construct 1,200 Acre ft. Impoundment	3.000	18.21 - 13.66	13.65 - 10.24	12.12 - 9.09
2. 16" Water line on N.W. 5th - Furnish to Horn	0.023	0.14 - 0.10	0.10 - 0.07	0.09 - 0.07
3. 12" Water line SE Court Avenue to Riverside Ave.	0.035	0.21 - 0.16	0.16 - 0.12	0.14 - 0.11
4. 16" Water line from Hwy. 30 North on Hwy. 37	0.031	0.19 - 0.14	0.14 - 0.11	0.13 - 0.09
5. 16" Water line on NW Gilliam NW 12th to Hwy. 37	0.085	0.52 - 0.39	0.39 - 0.29	0.34 - 0.26
6. 20" Water line from new State Hospital Well to SW 28th Drive.	0.140	0.85 - 0.64	0.64 - 0.48	0.57 - 0.42
	3.314	20.12/15.09	15.08/11.31	13.39/10.04

CATEGORY III

FOR A POPULATION OF 22,500 PERSONS

Item	Capital Cost	15,000 Persons	20,000 Persons	22,500 Persons
1. Increase Surface Storage 700 Acre ft.	1.34	8.13 - 6.10	6.10 - 4.57	5.41 - 4. 06
2. Increase the Capacity of new State Hospital Well	0.350	2.13 - 1.58	1.58 - 1.18	1.41 - 1.06
3. Second 12" feed line to Montee Addition	0.140	0.85 - 0.64	0.64 - 0.48	0.57 - 0.43
4. 12" Water line on SE Frazer	0.110	0.67 - 0.50	0.50 - 0.38	0.44 - 0.33
5. 16" Water line from State Hospital to 12" line to Industrial Park.	0.175	1.06 - 0.80	0.80 - 0.60	0.71 - 0.53
	2.115	12.84 - 9.62	9.62 - 7.21	8.54 - 6.41

It is our recommendation that the proposed improvements listed in Category I be considered as a single unit. It is felt that all of these components are urgently needed and therefore were not prioritized. The components in Categories II and III were prioritized with the first component having the highest priority. Depending on subsequent developments within the community the priorities may need rearranging to be responsive to what are now unforeseen needs.

In addition to the above improvements and the revenue requirements to pay for these improvements should be added an adjustment in the present rate structure to reflect the diminished purchasing power of our currency. To restore the water rates to the equivalent purchasing power of the dollar in 1968 to July 1979 would require increasing the present revenue 81% or $0.224/1,000 gallons ($0.168/100 cubic feet). As stated earlier in this report, the failure of revenue to keep pace with inflation has been accomplished by a gradual reduction of general operation and maintenance activities of the water department.

In light of the magnitude of the capital improvement program recommended in this study and their impact on water rates, the City Council may wish to:

1. Increase rates for operation and maintenance in annual increments above the inflation rate.

2. Delete some of the capital improvements recommended in this study, in categories II and III.

3. Rearrange or defer some of the proposed capital improvements.

D. ADDITIONAL REVENUE SOURCES

Another means of raising revenue used by some cities is a "buy-in" charge. The justification for a buy-in charge is that new developing areas should pay for some portion of the physical plant they are "hooking-up" to. The underlying objective is to minimize the impact of new growth upon existing customers of the water system. This source of revenue is keyed to growth and can contribute significant amounts as long as the City continues to experience populations gains.

E. SUMMARY

Adjusting City water rates to the 1968 level would in itself add significantly to the water rates. Restoration to that level would, however, provide for funding of needed maintenance and the gradual installation of needed feeder mains in the distribution system.

Developing a new rate structure will materially affect the extent of the volume of water sold. A rate structure with a low minimum charge will have the most adverse impact on revenue. Water sales in the summer months would drop off substantially, materially reducing needed revenue. A larger minimum monthly charge would materially reduce the impact on sales of water and keep the yards green.

The Water Department is required to construct facilities large enough to meet fire demands, seasonal irrigation demands, in addition to the normal domestic demands. The "readiness to serve" or "standby" charge incorporated into the monthly minimum charge can be easily justified on this oversizing and the real savings customers enjoy on fire insurance coverage.

It is important to also recognize that a substantial investment is required to provide a water of high quality for drinking, cooking, and industrial uses. These uses represent only a small percentage of the total annual water consumption. For other uses such as washing, waste disposal, irrigation, and the fighting of fires, a water of considerably lesser quality would be adequate. In short, a substantial portion of the capital investment is to satisfy a relatively small demand for human consumption. A "people demand" related charge can therefore be justified on the quality of the water required for this limited demand and also the costs incurred in preserving the integrity of quality of the water supplied throughout the entire distribution system.

The per capita cost for water service on an annual basis for the year 1978 was \$1.86/month. As this includes sales to industry and commercial customers, the actual per capita cost to residential customers was actually closer to \$1.30 or \$3.52 per residential unit per month.

The City of Hermiston has a provision in their ordinance for a lower monthly minimum for those residents that qualify as hardship cases. Raising of the monthly minimum would also help stabilize the annual base income by minimizing the adverse impact on revenues from wet and cool summers.

CHAPTER X

RECOMMENDATIONS

A. SYSTEM IMPROVEMENTS

1. Water Rates and Grant Funding

Several capital improvements have been recommended in this study. Some of the recommended improvements may be eligible for funding from various state and federal agencies. Those improvements that may have any possibility of funding should be investigated as soon as possible. Long delays in obtaining grant funding and the strings that are sometimes attached negate any real benefits. With the present rate of inflation it is extremely difficult to project very far in the future the cost of construction projects. The cliche that "today's high costs are tomorrow's bargains" was never more true than in our current economy. Preliminary checking with East Oregon Association of Counties, state and federal agencies has not been encouraging on Pendleton receiving any grant assistance in the foreseeable future.

2. Water Treatment Mandated by EPA

The proposed water treatment plant will have the most impact on water rates. The Oregon office of the EPA has never been staffed at an adequate level to supervise and enforce the provisions of the Safe Drinking Water Act of 1974. The apparent unresponsiveness of this agency does not necessarily mean a moderation on their position about their requiring that the City treat the Thornhollow Springs supply or stop using this source. This firm, consultants for the City of Echo, Oregon, was personally involved in the negotiation of Compliance Agreement with EPA. The alternatives presented to Echo were to either abandon the City Water System and let residents develop their own well systems or to improve the system in accordance with an acceptable Compliance Agreement. The Compliance Agreement mandated some immediate improvements and the more significant improvements were tied to a construction schedule. More recently the EPA has been involved in an enforcement action against Neskowin.

The opposition of EPA in a "no-win" contest would draw a lot of unfavorable press in papers with statewide circulation. Unfavorable press could have an adverse effect on the local tourist and convention center oriented businesses. The City should begin negotiations with EPA for both an exemption and a compliance agreement as stated in EPA's most recent correspondence.

3. Artificial Recharge of Underground Aquifer

We are recommending that an experimental artificial recharge to the deep underground aquifer be conducted at the Round-Up Well (No. 2). This well has a relatively high specific capacity (yield/foot of drawdown), and is situated on ample

City owned land where additional components can be easily accommodated. A minimum of piping modifications are required at this well and a nearby monitoring well could be located within Round-Up Park. Monitoring equipment would probably require an expansion or addition to the existing well house. As a part of this artificial recharge experiment, continuous depth recorders would be installed in each of the City Wells to monitor the elevation of the water table at each well. State law requires that each well be equipped so the depth to the water table can be measured. The depths to the underlying water table cannot be presently determined in four out of six of the present City Wells within the City itself.

4. Sherwood Heights Low Pressure Problems and Supply to McKay Creek Area

Providing a relief water supply line to the Montee Addition, Edwards Addition area is a high priority need. A single 12" diameter feed line to these areas should provide substantial relief to the low water pressure problems presently encountered in the Sherwood Heights School area. The westerly alignment of a 12" waterline from S. W. 28th Drive requires a longer length of line to effect any relief and going through undeveloped property would receive little if any support from land owners. The easterly alignment of a 12" waterline along the new and old U. S. Highway #395 would cost more per foot but should receive more participating funding from abutting properties and cost considerably less along this route.

5. Proposed Reservoir for Southwest Area

The proposed new reservoir to serve both the low and high level distribution systems and the 12" diameter waterline to serve as an intertie between the South Hill high level system and the Southwest high level system should be considered as a single unit. The construction of 1,000,000 gallon reservoir is recommended to permit the City to grow to a population of 25,000 persons without acquiring the Brogoitti Well.

6. Water Rights: Evaluation, Additional, and New Legislation

The City should have legal counsel determine the status and value of the current water rights on the Umatilla River, water rights on the North Fork of the Umatilla River, and the Johnson Well. Legal counsel should also evaluate the merits of requesting the State Water Resources Department to investigate our geological area on the assumption that a Critical Groundwater Area Declaration may be desirable.

Water right applications should be filed as soon as possible on: winter surplus waters in the Umatilla River; logical upstream surface storage sites; and the Columbia River. The Columbia River water right may not be needed but to be prudent the right should be established until proven otherwise.

Enabling legislation by the State for the creation of artificial recharge improvement districts should also be explored. This would assure participation in recharge costs from all water right holders determined to be withdrawing water from a common source.

7. New Hospital Well and 20" Diameter Transmission Line

The new State Hospital Well has the good fortune of having the best specific capacity (yield/foot of drawdown) of any known well in the area. Test data indicates that 3,000 gpm could be withdrawn from this well with only 15.9 feet of drawdown. To obtain a comparable flow from the other City wells would require the following drawdowns at the other City Wells:

Well No. 1 (Byers)	31.2 feet
Well No. 2 (Round-Up)	32.3 feet
Well No. 5 (Stillman)	106.7 feet
Well No. 3 (S. W. 21st Street)	493.4 feet
Well No. 4 (Hospital)	659.3 feet

The new Hospital Well offers an opportunity to withdraw large quantities of water during high demand periods with a minimum amount of electrical energy. This well should also be an excellent well for artificial recharging of the deep groundwater. The reconstruction and enlargement of this well would provide the first real opportunity to properly design a well for recharging, overcoming many of the drawbacks previously encountered in using existing wells not specifically designed for artificial recharge.

In conjunction with acquiring the new Hospital Well, the proposed 20" diameter line from the new Hospital Well to the existing 16" water line in S. W. 28th Drive will provide a means of meeting high summer irrigation demands experienced in the Southwest section of the City. This well should also be connected to the 12" line along old U. S. Highway #30. This would permit placing the two most inefficient City wells (No. 3 and No. 4) on a standby basis and reduce the electrical energy costs for pumping of well water.

8. Sewage Plant Well

It is our recommendation that from current available water department funds that a yield test be performed on this well this fall or winter. The conflicting data on this well has removed this well from serious consideration in this study.

9. Proposed Improvement Schedule

Our recommendations for a proposed capital improvement program will be limited to the time requirements for placing a water treatment plant "on-line" to maintain the gravity supply as a useable source. All other contemplated improvements, with the exception of the upstream surface impoundments, could be accomplished within the time frame of the proposed water treatment plant.

The scheduling shown below commences on the day the City Council elects to proceed with the proposed water treatment plant.

Action or Activity	Running Total of Time Required in Months
Council decision to proceed	Start
Public hearings, information, negotiations with EPA	$1\frac{1}{2}$
Hold Bond Election and sale of bonds	3
Acquire site and authorize design	4
Sample raw water and design of facility	14
Advertise for construction bids	15
Award of bids and construction	30
Plant start-up and "shakedown"	31
Plant operational	32

At the City's option, four months' time could be saved by authorizing design to commence prior to the holding of a bond election and the sale of bonds.

10. Coordination With Other Entities

The Thornhollow gravity supply is situated along the Union Pacific Railroad right-of-way and the County Road to Thornhollow and Gibbon. While this location is convenient and easily accessible for maintenance, it does expose this source to considerable risk of contamination from oil, gas, and chemical spills as a result of a train derailment or an accident involving a transport truck. On two occasions during this study, livestock were found present on the land over the springs. Residences with septic tanks are sited in reasonably close proximity to the Wenix and Simons Springs. A feed lot located southeast of Simon Springs often surface floods towards Simons in the spring months. Improved fencing, clearing of vegetation, and control over access to these sites need to be improved. Perhaps after clearing a leasing of this property may provide an economical solution.

It is our recommendation that an on-going coordination be developed between the City Water Department staff and the staff of the County Sheriff's Department and the Union Pacific Railroad. The City should be immediately notified of any accident or derailment in the area of, or above the buried collection galleries. This source should either be continuously monitored or bypassed whenever there is any risk of any foreign pollutants impairing the water quality.

11. Future Studies

This study is the first comprehensive study of the total water system in 15 years. Water systems are very capital intensive and to assure that expensive capital improvements are cost effective, the water system should be re-evaluated at least every five years.

The conclusions and information presented in this report were based on an exhaustive search of the City's records. The hydrant flow testing program has been practically nonexistent for a number of years. The lack of periodic checking of valves in recent years has resulted in some of the system valves being left accidentally closed. Existing pressure recorders are not functionable and flow recorders for the high level systems are inoperable. Working within the limited amount of available data we had to hypothocate residual pressures and system loadings.

We are therefore recommending that the present monitoring equipment be repaired and the three recommended continuous pressure recorders be installed as soon as possible. Data should be collected through the summers of 1980 and 1981 and a new computer analysis of the system be performed in the fall of 1981. It would also be desirable to have all the system valves checked prior to the summer of 1981 and hydrant flows at selected locations be performed during the summers of 1980 and 1981.

12. Conclusion

The magnitude of improvements contemplated in this study for the City of Pendleton by itself seems enormous. Other cities, however, are facing similar programs in upgrading and improving their water supply capabilities. Ontario, Oregon, with a population of 8,520 persons, is embarking on a $3,650,000 water improvement program. The City of Hermiston has filed for water rights on the Columbia River as their ultimate water source. This firm prepared the study for a Regional Water Supply for the City of Hermiston in September of 1976 and the costs were then estimated to be $5,300,000. The Regional supply for the City of Hermiston was predicated on a Ranney Well being constructed, thereby eliminating the cost of building and operating a water treatment plant.

The water table at the City of Athena dropped 40 feet this year and 60 feet in 1978. The rapid depletion of the underlying groundwater source at Athena is considerably more dramatic than the 5.1 feet lowering of the water table encountered at the Stillman Well last year. The era of cheap and plentiful water in our area is rapidly becoming history. The future water supply for the City of Pendleton is keyed to the treatment, development, and expansion of the Thornhollow gravity supply. For additional water supply above and beyond that available from the Thornhollow supply, the Columbia River at this writing appears to be the only other viable alternative.

AREA GEOLOGY

1. COLUMBIA RIVER BASALTIC LAVA FLOWS.

The Columbia River basaltic lava flows encompass an area of approximately 50,000 square miles as shown on the map on page A-2. The depth of the lava flows varies considerably and is known to be in the central part of the flow as much as 5,000 feet thick. The total thickness of the basalt is the result of a series of individual sequential flows of lava ranging from 50 to 150 feet thick. During the cooling process the lava formed vertical columnar joints normally 5 or 6 sides similar in appearance to the surface of drying mud flats. The sides of these columns normally range in inches with a few measured in feet. As the surface of flows cooled very rapidly there were often left horizontal zones where scoriaceous material ("honey-comb" like rock) were formed by entrapped gases. When two horizontal scoriaceous zones are separated by shallow lava flows of say ten feet and also interconnected by vertical columnar joints the result is a single permeable zone that can yield considerable quantities of confined deep ground water.

Individual lava flows have been identified for distances up to ten miles and in our immediate area the total combined thickness ranges from zero to more than 2,500 feet. In practically all of the Umatilla River basin which covers some 2,700 square miles, the basalt either underlies the surface at a shallow depth or crops out above the surface.

The series of lava flows that formed the Columbia basalts apparently occurred over a relatively short period in terms of geologic time. The basis for this are that the flows are layered concordantly (succeeding flow layers paralleled prior flow layers) without much evidence of weathering or deposition of wind blown material between the succeeding layers. There is also a general lack of water transported materials between the successive flows which would have been deposited by the formation of several dams created if deformation of the earth's surface had taken place during the period of these lava flows. As tracement of single flows have been limited to ten miles in length, it is believed that the basalt issued out upon the surface quietly from several fissures.

An examination was made at the base of the basalts with a sea level datum of 1,300 feet near Gibbon. The flows at this location are about 2,500 feet thick and revealed that no evidence of interflow marine sediments, zeolite mineralization or pillow structure which one might expect with a submarine extrusion of the basalt. It is therefore believed that the basalt was originally deposited at least 1,000 feet above sea level and that after the lava flows ceased, deformations, warping, folding, subsidence, faulting and uplifting occurred to provide the topography that we have today. If any lava flows had occurred during the reshaping of the topography this would have been evidenced by the successive layers of lava flows being at a considerable angle to the preceding flow.

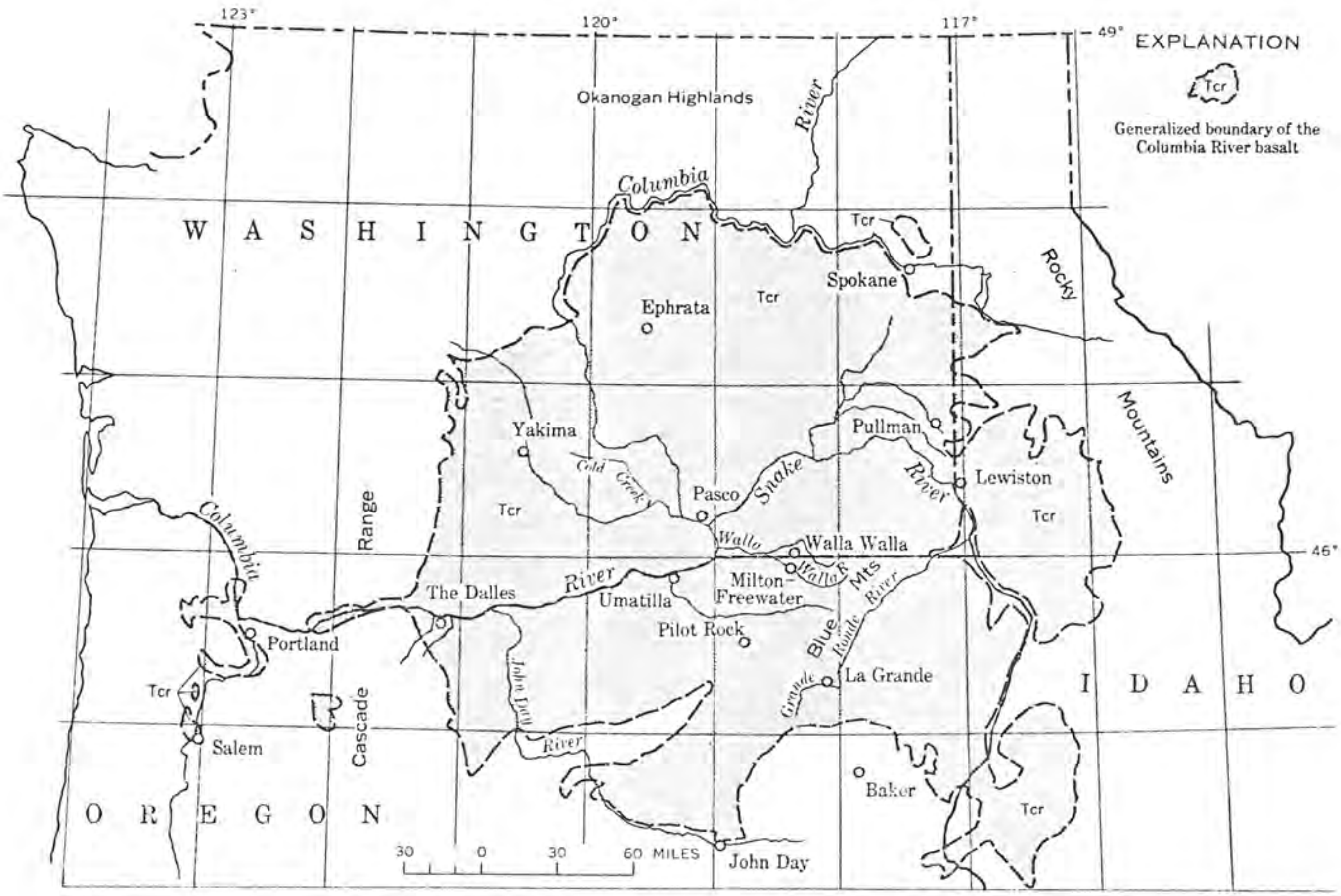


FIGURE 1.—Map showing the main area underlain by the Columbia River basalt, Washington, Oregon, and Idaho.

Reproduced from:

"Hydrology of Volcanic-Rock Terranes", a Geological Survey Professional Paper 383-A by R. C. Newcomb. U. S. Government Printing Office, Washington: 1961.

The lava flows on Blue Mountain upland above the Blue Mountain monocline (fold line of the basaltic rock from a level plane to a sloping plane) form a nearly horizontal platform-like crest. Precipitation that occurs in the Blue Mountain uplands percolates to the top of the impermeable basalts and becomes a perched water body on top of the basalts. This water then flows nearly horizontally to the nearest water course or appears as a spring on the side hills. Natural recharge of the deep ground water occurs where the original layered flows have been tilted upwards and the scoriaceous ("honey-combed") rocky materials are exposed to, and have an opportunity to intercept either surface or shallow ground water. Therefore, little if any, of the precipitation over this large area becomes a part of the deep ground waters.

The typical basaltic flow in our area ranged from 30 to 50 feet with a few individual flows of more than 300 feet thick. Because the lava cooled rapidly the lava at the lower part of each flow which were under considerable weight consolidated the lava into a tight fine-grained mass, making identification of the individual mineral grains indistinguishable without the aid of magnification. In the upper part of each flow there were gas bubbles trapped during the rapid cooling which are called vesicles and give the rock the "honey-combed" appearance. Often the density of the gas bubbles was quite high near the top of a particular flow, giving it a rubbly, slaglike, broken rusty red appearance. Unfortunately, successive flows of lava often completely obliterated this porous zone which permits the rapid transmissability of water. During the individual lava flows that make up the Columbia River basalts there is some evidence of volcanic ash, tuft (disintegrated rock) and river deposited sediments in limited localized areas.

2. DEFORMATION OF BASALTS.

During the formative years following the establishment of the Columbia River basalts, geologists are of the opinion that there were at least two stages of deformation. Investigation of stream canyons such as the south wall of East Birch Creek provides documentation of at least two different occasions when the erosive action of water had cut through two distinct periods of uplifted basalts that had in effect created surface impoundments.

During the deformation period of the Columbia River, basalts were gradually bent from both subsidence and uplifting and depending on stresses, some vertical and horizontal shearing (faulting) occurred. In areas where large flows sheared either vertically (uplift and subsidence) or horizontally, the result was at these interfaces (faults) there was tons of pressure that in effect ground a homogenious mass of lava into fine to coarse rubble. This gouging along vertical and horizontal shear lines (faults) accelerated the decomposition of the mineral constituents into clay-like materials which form impermeable barriers to the transmission of ground water. These fault zones create in effect subsurface dams impounding essentially dead water.

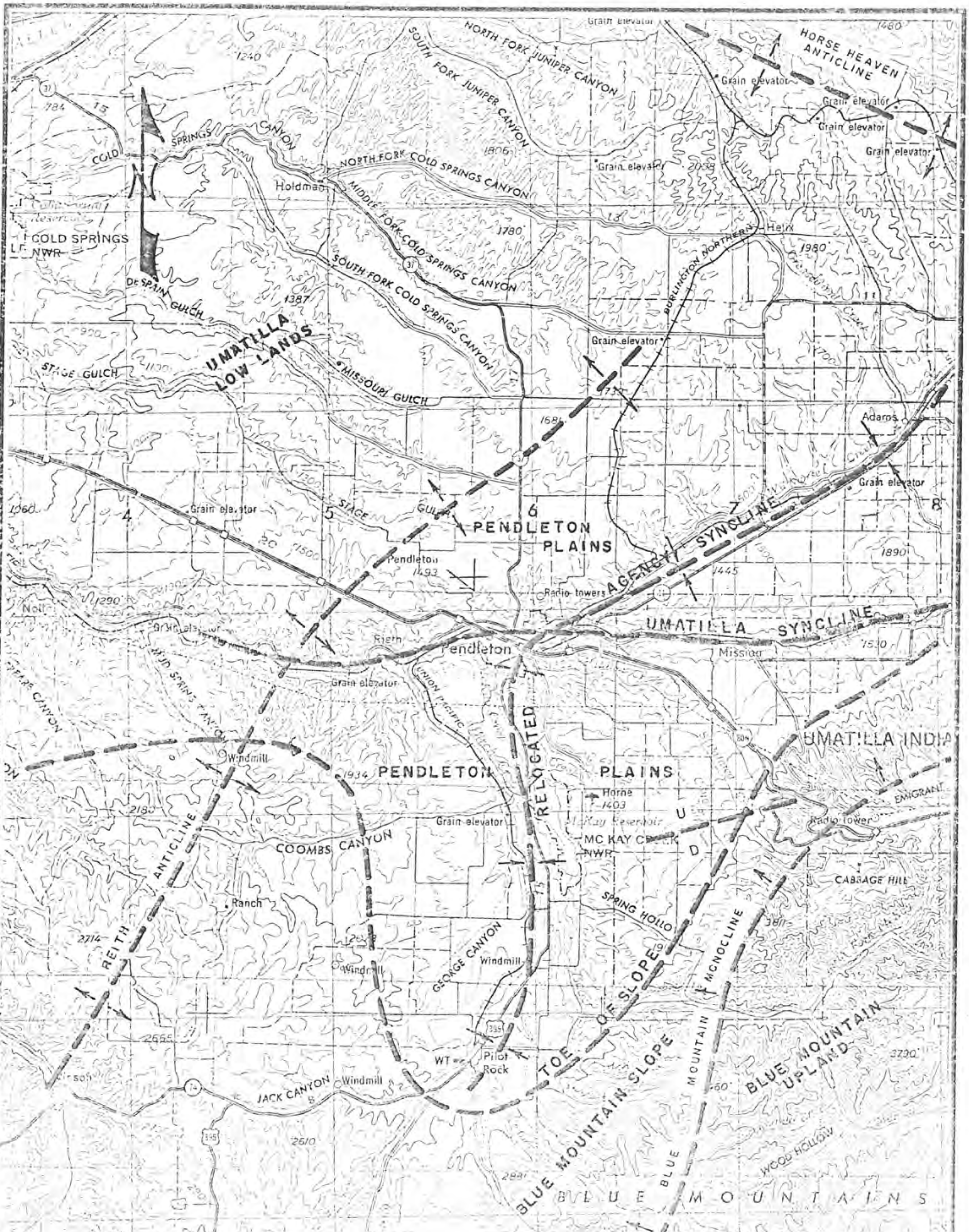
In some areas where the deformation was not as severe, anticlines (upward arching bands of the lava sloping downward both directions from the apex) were formed and synclines (troughs or downward dips) were formed without faulting occurring. Large masses of rock formations have a more fluid-like character and under weight over extended periods of time can bend similar to more pliable materials without breaking. The primary anticlines and synclines are shown on the geologic map on page A-5. Within this area some local, but incomplete, mapping of faults have been established in prior geological studies. The only mapped fault of record within the boundaries of the anticlines and the monocline is shown on the geologic map shown on page A-5 with "U" denoting uplift and "D" denoting subsidence. This is not to say that there may exist several other local subordinate areas that have not at this time been identified. As man and mother nature through erosion cut into the face of hills and mountains, geologists are provided with additional evidence to refine their prior observations.

Based on telephone conferences with Joe Gonthier, co-author of one of the more recently published geological papers which were based on field investigations conducted in 1975, the position of the Agency Syncline has been relocated to the vicinity of the Emigrant Avenue interchange (Harris Pine Mills). This geological study further indicated that the potentiometric surface (static level) of deep ground waters underlying the City of Pendleton proper area are relatively level and that the hydraulic gradient increases (upward slope of the water table) at the rate of 120 feet per mile to Mission (300 feet in 2.5 miles) and from this point eastward the water table is within a few feet of the valley floor. This report further cites that shallow basalt wells closely spaced on the Reservation for irrigation purposes causes mutual interference of water levels in these wells. Also noted in this report was that the continual lowering of the water table under the City proper is attributable to the high rate of withdrawal by the City.

3. LOCAL GEOLOGICAL SUB-AREAS.

On the geologic map shown on page A-5, local sub-areas have been given geologic titles. The Blue Mountain upland area as discussed earlier is underlain with a nearly horizontal or platformlike layer of basaltic rock which provides little if any natural recharge to the deep ground water supply. The border of the Blue Mountain upland is shown on the map as the Blue Mountain Monocline. Along the monocline the basalts tip downward to the Northwest on a downward ramp with grades ranging from 2% to 60%. At the base of the foothills another alignment has been established and the land in between these areas has been named the Blue Mountain Slope. Based on an examination of well logs for this area, they indicate there is a shallow overlayment of 1 to 23 feet of soil, gravel, hardpan, and claystone over the basalts. Wells drilled in the basalts in this area typically yield from 20 to 60 gpm with one well yielding 100 gpm.

The Umatilla syncline (dip or troughlike bottom) and the relocated Agency Syncline intersect in the vicinity of the Southgate interchange (Harris Pine Mills). The low point at the intersection of these synclines is bounded on the west by the Reith anticline (upward arch or fold) from which basalts are tilted backwards to the Southeast; to the North by the Horse Heaven anticline; to the East by the Blue Mountain Anticline and to the Southeast by the Blue Mountain Monocline.



PROJ. PENDLETON WATER STUDY	WALLULIS & ASSOCIATES INC ENGINEERING & PLANNING 100 E. MAIN PENDLETON HERMISTON OREGON	SCALE 1 : 250,000	NO.
TITLE GEOLOGICAL SUB AREAS		DATE APRIL 1979	

BRUNING 40-22 23381

From this is is obvious that if all of the locally deposited flows were all hydraulically interconnected, the deep ground waters would flow to the geologic intersection of the synclines in the general vicinity of Harris Pine Mills. Prior studies have not been extensive enough to determine the actual extent of hydraulic interconnection between the individual local lava flows or identify all the faults which may exist below the surface.

As stated before, if all the lava flows were hydraulically interconnected, it would mean that all wells outward from Pendleton would have the first opportunity to intercept the deep ground waters before they reached the bottom of the basin. It is also possible that there are several faults resulting from the vertical or horizontal shearing of the mineralized lava preventing the movement of deep ground waters towards Pendleton. The possibility of subsurface unmapped faults, intermixing and overlapping of localized lava flows which may seal off porous aquifers make the selection of a site for a well highly speculative and to a large degree purely a matter of chance.

Upon reviewing the wide variation of specific capacities for wells in the Pendleton area, it would seem only prudent to incorporate the best producing wells in the area into the system to meet peak demands. Both the new State Hospital Well and the Brogoitti Well should be considered as a means of meeting future summer peak demands. Besides having high specific capacities (which reduces pumping costs), they are located in areas of need and future growth.

4. GEOLOGICAL FORMATIONS PREDATING THE COLUMBIA RIVER BASALTIC FLOW.

The Columbia River basalts which issued forth from localized fissures were deposited over older geologic rock structures such as the CLARNO (sandstone, shale) which were in turn deposited over INTRUSIVE rocks (quartz, diorite with some noride), all of which rest over METAMORPHIC rocks (amphibole schist, gneiss, scattered bodies of hornblendite and coarse granite).

5. GEOLOGIC DEPOSITS AFTER FORMATION OF COLUMBIA RIVER BASALTS.

Following the formation of the Columbia River basalts and during and after their deformation (uplifting, subsidence, and faulting) there occurred a period of time in which surface materials were transported by streams or ice flows and deposited in the lower elevations. This material has been given the geologic term of Fanglomerate and consists of basaltic sands and gravel, gravel, silt, sand, clay and sandstone. These deposits normally range from 30 to 200 feet thick. These deposits normally have low permeability and may consist of gravels cemented with calcium carbonate. Clean layers of sand and gravel are rare and the deposition as a unit is generally impermeable. These deposits overlying the basalts generally do not produce adequate water from wells except for stock watering or limited domestic use. Precipitation that falls on these deposits usually finds its way to abutting minor and major water courses that cut through these deposits.

Following the above deposits of Fanglomerate there was a deposition of wind-blown brown silts (loess) called the Palouse Formation. The opinion of geologists is that the Palouse Formation of windblown silts originated from sediments in an ancient glacial lake formed by the damming of the Columbia River northwest of the City of Pendleton. The lenses (Layers) of this formation are generally less than four feet thick and in localized areas may be as much as ten feet thick. Wells that withdraw water from this formation have very low yields and generally where these deposits are found, annual precipitation is low.

On top of the Palouse Formation there is in localized areas deposits of volcanic ash. These localized areas are normally less than one acre in size and consist of a grayish white, very fine grained, uniformly textured ash less than five feet thick. The origin of the ash has been identified as the result of volcanic emission in the Cascade Range. Overland flows from high intensity rainfalls has concentrated the ash within small alluvial fans (water transported) near the edges of principal water courses.

In more recent geological history the alluvial deposits (Alluvium) along most intermittent and continuous water courses have evolved from the combined reworking of loess and Fanglomerate deposits. This reworked material has formed deposits that are both porous and permeable, along with formations that are relatively tight and impermeable. Normally, wells in these deposits are limited to stock wells or domestic wells to serve ranches. According to two of the geological reports, the City of Pendleton's spring sources which consists of buried open joint and perforated pipe systems, induces a flow from the more permeable alluvial gravels in the upper reaches of the Umatilla River flood plain.

In these alluvial deposits over the basalts in the Pendleton plains south of the Umatilla River and west of McKay Creek, it is estimated that 40% of this area is probably saturated with shallow ground water perched above the underlying basalts. These deposits south of the Umatilla River may be as much as 100 feet thick and where lenses of sand and gravel exist, shallow wells in this area may be expected to yield up to 80 gpm. On the remainder of the Pendleton plains there is not believed to be any, or very little, saturated soil with water perched above the underlying basalts.

WASHINGTON STATE UNIVERSITY

PULLMAN, WASHINGTON 99164

DEPARTMENT OF CHEMICAL ENGINEERING

Department No. (509) 335-4332

Air Pollution & Resources (509) 335-1526

Nuclear Engineering, Radioisotopes & Radiocarbon Dating (509) 335-4731

Chemical Energy & Processing (509) 335-4332 or (509) 335-4333

RECEIVED

DEC 11 1978

WALLULIS & ASSOCIATES

December 5, 1978

Mr. Stanley G. Wallulis, P.E.
Wallulis & Associates, Inc.
Engineering-Planning-Surveying
5 SE Nye (Box 398)
Pendleton, OR 97801

Dear Mr. Wallulis:

Here are the results of our ^{14}C dating of your water samples. To date these samples we passed the water through an anion exchange resin to absorb any bicarbonate ion present. The resin was acidified with phosphoric acid to generate carbon dioxide. Subsequently, the carbon dioxide was converted to methane and the ^{14}C disintegration rate determined with our gas proportional counter.

Relative to water that we have dated in this region (mostly Hanford, Horse Heaven Hills, and the Columbia Basin) these samples are rather young. Most of the samples that we have dated recently have ages in the 10,000 to 20,000 range. However, ages this young have been observed for water in the Columbia Basin.

There are several explanations for these ^{14}C ages. 1) the ages are correct. This means that the water has been out of contact with the environment for somewhere between 2570 and 5840 years. In this case we are assuming no complicating factors. 2) the water is not recent recharge water because it does not have a value approaching contemporary ^{14}C. If this water was totally "modern", it would have a ^{14}C counting rate approaching 140 percent of our pre-H-bomb reference material (NBS Oxalic acid). 3) the age may reflect water mixed from two or more aquifers or with some contemporary water. If the ^{14}C in these water samples is derived from a mixture of "old" and "contemporary" water, the "mission" and "State hospital" water samples contain 34 and 73 percent modern, past-H-bomb ^{14}C, respectively. 4) these water samples could be younger than the reported values because of chemical reactions of the type

$$Ca^{12}CO_3(\text{limestone}) + H_2O + {}^{14}CO_2 = Ca^{2+} + H^{14}CO_3^- + H^{12}CO_3^-$$

Reactions of this type essentially dilute the ^{14}C in the water making the radiocarbon age falsely old. The maximum effect would be on ^{14}C half-life or 5740 years.

Thus, there are several complicating factors to the intrpretation of ^{14}C ages of water. If the water had ^{14}C ages greater than ten thousand years, there would be little doubt that the water was truly "old".

Sincerely,

John C. Sheppard

John C. Sheppard
Professor Chemical Engineering
and Anthropology

RADIOCARBON DATING LABORATORY

WASHINGTON STATE UNIVERSITY
Pullman, Wash. 99163

SAMPLE REPORTING FORM

WSU sample number: | Submitter's sample number: | WSU age date determination:

SEE REMARKS BELOW

Date received:

6-19-78

Date reported:

12-5-78

Name of submitter:

Stanley G. Wallulis, P.E. Wallulis and Assoc.
Engineering-Planning-Surveying, Pendleton, OR 97801

Requisition number:

Description of sample:

WSU work order number:

Submitter's estimate of age date:

Remarks:

WSU #	Your #	^{14}C date, years B.P.
2037	Mission Well	5840 ± 120 years
2038	State Hospital Well	2570 ± 135

See attached letter for comments about ^{14}C dating of water.

Sample processed by:

Y. Welter

Sample calculated by:

Y. Welter/J. Sheppard

Sample reported by:

J. Sheppard

Note: All analyses are based upon the Libby half-life (5570 ± 30 years) for radiocarbon. To convert ages to the half-life of 5730 years, multiply the age given above by 1.03. Zero age date is A.D. 1950. (Reference: Editorial Comment, RADIOCARBON, Vol. 7, 1965).

cc: W. Bonnington
S. Dixon
Y. Welter

WALLULIS & ASSOCIATES, INC.

ENGINEERING & PLANNING

PRES: STANLEY G. WALLULIS, P.E.

- Water & Sewage Treatment, Transmission & Distribution
- Streets & Storm Drainage
- Municipal Planning
- Rate Studies & Financing
- Subdivisions
- Surveying
- Solid Waste

* 213 S. W. Emigrant Ave.
Pendleton, Oregon 97801
(503) 276-1598

108 East Main Street
Hermiston, Oregon 97838
(503) 567-3331

* Address replies to this office.

May 2, 1978 78-107

Mr. Robert Brown
Project Engineer
Central Snake Projects Office
Bureau of Reclamation
214 Broadway
Boise, ID 83702

Re: City of Pendleton Water Study

Dear Bob,

We have been requested by the City to investigate the possibility of utilizing water stored at McKay dam as a potential source for residential and industrial use. To evaluate this as a potential source, we would need the following information:

1. Amount of water currently being rented on an annual basis and not secured by firm contracts.
2. Cost of acquisition of stored water if the City is determined to be eligible for such water.
   (a) Capital costs.
      (1) Lump sum.
      (2) Long term purchase agreement with annual payments.
   (b) Annual operation and maintenance costs.
3. When such water will be available for firm contracts.

I appreciate the courtesy and cooperation you have conveyed in prior telephone discussions and would appreciate any additional comments you may feel pertinent or beneficial. I hope we can get together when you are in this area the first of next week.

Best regards,

Stanley G. Wallulis, P.E.
President

SGW:jgp

cc: Jerry Odman, Public Works Director

United States Department of the Interior
BUREAU OF RECLAMATION
CENTRAL SNAKE PROJECTS OFFICE
214 BROADWAY AVENUE
BOISE, IDAHO 83702

IN REPLY REFER TO: 100
563.

June 7, 1978

Mr. Stanley G. Wallulis, President
Wallulis & Associates, Inc.
213 S. W. Emigrant Avenue
Pendleton, Oregon 97801

Dear Stan:

This is in reply to your letter of May 2, 1978, regarding the possibility of using water stored in McKay Reservoir for residential and industrial use for the City of Pendleton, Oregon.

The McKay Reservoir was authorized as a single-purpose irrigation project. While it is true that, under Section 9(c)(2) of the Reclamation Project Act of 1939, the Secretary of the Interior has authority to market M&I water from a project where that function is not an authorized function, such an arrangement can be made only if it does not impair the efficiency of the project for irrigation purposes. In view of the long standing list of individuals who have requested permanent contracts for storage space in McKay Reservoir to irrigate their lands and the interest expressed by the Westland and Stanfield Irrigation Districts in obtaining all unobligated space available following the recent reauthorization of McKay Dam and Reservoir, it would not be possible to demonstrate that the sale of M&I would not interfere with irrigation.

If we can be of any further help to you in this matter, please let us know.

Sincerely,

Bob Brown

Project Superintendent

cc: Jerry Odman, Public Works Director
City of Pendleton, City Hall
34 S E Dorion, Pendleton, Oregon 97801

UMATILLA RIVER BASIN

STATE WATER RESOURCES BOARD

JUNE 1963

FINDINGS AND CONCLUSIONS

28. Flood damage benefits are not great enough to justify large single-purpose structures. Multipurpose structures are needed and are more easily justified.

29. Small reservoirs on important tributaries could reduce local flooding and erosion and provide late season water for irrigation, livestock, and fish life.

45. Storable surface flows on the upper Umatilla River (above Pendleton) amount to about 180,000 acre-feet, most of which can be stored at either of two proposed sites, Mission and Thornhollow. Reservoir construction plans at these sites have been hampered by problems encountered with Indian lands, railroad rights-of-way and anadromous fish. Development of a site in this area will be necessary, even with upstream storage, to fully develop the water resource potential of the upper Umatilla River.

50. Summer flows recommended by the Oregon State Game Commission for the Umatilla River system are considerably in excess of presently available supplies. Future storage and groundwater development plans should give consideration to enhancement of fish life as one of the ten beneficial uses of water.

FEB 15 1978

Source Evaluation, City of Pendleton

Bill Titus, Project Officer

Don Gipe, Water Supply Coordinator

1. Background

On Tuesday, January 31, 1978, I met with John Molsness, Director of Public Works, City of Pendleton. The purpose of the meeting was to discuss the classification of the source works which the City maintains in the Thornhollow area and to perform a field survey to determine whether or not the current classification is correct (see correspondence file).

During the course of the meeting, I reviewed the as-built plans of the source works, interviewed Mr. Molsness and several members of his staff regarding the construction, operation and maintenance of the works, the general quantity and quality of water produced, the ownership, use and control of the land in the immediate vicinity of the works and the ultimate origin of the water produced by the works. Following the meeting, I visited the source works and control facilities, accompanied by Mr. Molsness and a member of his staff.

2. Source Description

The source works are located in the valley of the Umatilla River about 15 miles northeast of Pendleton, in the Umatilla Indian Reservation. At that location, the valley is a broad, u-shaped canyon cut from the extensive lava flows which blanket the area for many miles around. The canyon is approximately 6,000 feet wide at this point, and 600-800 feet deep, with steeply sloping sides and a flat bottom composed of coarse sand and gravel. The river meanders back and forth across the valley floor, but is mainly confined to the northern side of the valley.

The source works consist of an extensive network of infiltration lines of concrete half pipe, concrete "A" frame pipe, open joint vitrified clay and concrete pipe and perforated steel pipe of varying diameters. These lines are buried 5-15 feet deep in the gravel of the south side of the valley floor at varying distances from the present-day river channel. The infiltration lines are concentrated at four separated areas, known locally as "Wenix", "Simons", "Chaplish" and "Longhair". The lines in these areas are connected by a tight-jointed transmission main which conveys the water by gravity to the city reservoirs. Each area is equipped with one or more valved drain lines which can be used to divert the water to waste when it is not needed or is of poor quality. An enclosed weir, located downstream from the infiltration lines, is used to control and gauge flows and is the turbidity monitoring point.

The land immediately surrounding the infiltration lines at the four sites is owned by the City and land use is controlled by fences. The surrounding area is owned by the Umatilla Indian tribe and land use does not appear to be well controlled. Portions of the valley floor in areas adjacent to the City infiltration lines are used for agricultural purposes, including the pasturing of cattle. Scattered farm buildings and houses occupy the higher elevations of the valley floor. The hilltops and gentle slopes on both sides of the valley are currently planted to winter wheat every other year. During alternate years these fields are ploughed and left fallow.

Production from the source works varies from approximately 3.5 mgd during the summer months to 5.25 mgd during the winter months. Actual production capacity of the source works during the winter months may be considerably higher, but only 5.25 mgd can be accommodated by the existing transmission line. The quality of the water produced by the source works varies with the seasons. Turbidity is highest during the winter months and lowest during the late summer. The bacteriological quality of the raw water is not monitored, but routine monitoring of the water after chlorination has been excellent. The water produced by the source works is very soft in comparison to the water produced by local wells.

During normal operations, only those lines which are necessary to meet the current demand are used. The others are diverted and run to waste. In general, the infiltration lines farthest downstream are in continuous use, the upstream lines are brought "on-line" as needed. Flows are normally controlled by manual operation of the main weir. To control flows during periods of high turbidity, a surface-scatter turbidimeter, operated continuously at the main weir, is set to automatically divert all flows when the turbidity exceeds 5 TU.

Maintenance of the source works is a relatively simple matter. The operation of the weir and the associated equipment is checked daily (Monday-Friday) and repairs are made as needed. Each spring, the infiltration lines are cleaned of roots and debris with rods and augers. An ongoing program of upgrading the existing transmission line has resulted in replacement of much of the line in the source works area. Recently, several stream crossings have been replaced.

3. Source Evaluation

Specific information relating to the local geomorphological and hydrological conditions is not available. Without this information, conclusions as to the ultimate origin of the water produced by the source works are speculative. Mr. Molsness maintains that the water originates from "springs" fed by confined aquifers located in contact seams between the various flows of the local Columbia River Basalt formation.

It is my opinion that the water originates from local rainfall and snow-pack and is carried into the gravels of the valley floor by the well-developed local pattern of streams and creeks and by the Umatilla River itself. In support of this opinion, I list the following circumstantial evidence:

1) the flow from the source works is seasonally variable, the peaks correspond perfectly to local precipitation and snowmelt patterns.

2) the water is relatively soft, a general characteristic of surface water which has had little or no contact with underground rock formations.

3) the turbidity is seasonally variable, with peaks corresponding to peak runoff periods.

Regardless of the ultimate source of the water, however, the flow from the ultimate source is being stored in the naturally-occurring fluvial gravel deposits which comprise the valley floor. These gravel deposits are not protected by an impervious layer (or even a restrictive layer) and are subject to contamination by surface runoff. Since protection from surface runoff is the essential criterion which distinguishes between ground water and surface water, it is my opinion that the source works in the Thornhollow area are surface water sources and it is my recommendation that they be classified as such.

Insofar as the City's request for an adjustment to the turbidity MCL is concerned, I do not believe that the watershed which supplies the source works (the fluvial gravel deposits up-gradient from the source works and the streams which contribute flow to those deposits) is protected to any reasonable degree from contamination. Since a protected watershed is an essential prerequisite for the MCL adjustment (see EPA DWPB Procedural Criteria 77-3), I do not feel that the City of Pendleton is eligible for the adjustment.

I do feel, however, in light of the degree of flexibility which is built into the system and the current practice of chlorination with adequate contact time, that an exemption might be granted to allow the City to install the treatment equipment which will be necessary to meet the turbidity MCL. A compliance schedule with interim control measures could be drafted to minimize the health hazard which the current turbidity levels represent.

BTitus/de 2/15/78
DGipe

UNITED STATES ENVIRONMENTAL PROTECTION AGENCY

OREGON OPERATIONS OFFICE
522 S.W. 5TH AVENUE
YEON BUILDING, 2ND FLOOR
PORTLAND, OREGON 97204

Rec'd 2-15-79

REPLY TO ATTN OF: 10000

Mr. Gerald L. Odman, Public Works Director
City of Pendleton
P. O. Box 190
Pendleton, Oregon 97801

Dear Mr. Odman:

This is to follow-up our past conversations about the turbidity problems at Pendleton and to provide a summary of the options available to Pendleton in this matter. I apologize for the delay in providing this summary and hope that it has not caused you any undue inconvenience.

As you know, the City's water system routinely violates the Federal maximum contaminant level for turbidity in two ways: first, the monthly average of daily turbidity measurements frequently exceeds one turbidity unit, and second, the average turbidity on two consecutive days has, on several recent occassions, exceeded five turbidity units. It is our opinion that those violations are an unavoidable result of the use of unfiltered surface water and can be eliminated only by the addition of filtration to the existing source or the use of an alternate source of water. Of course, the actual method for eliminating the turbidity violation must be determined by the city.

The Safe Drinking Water Act provides for the issuance of "exemptions" from the turbidity maximum contaminant level for water systems which are working toward compliance with the regulations. An exemption must show a final compliance date not later than January 1, 1981, unless the system is becoming a part of a new regional water system, in which case, the compliance date may be extended to January 1, 1983. On March 9, 1978, we received a request for an exemption from the City of Pendleton. However, because the city showed a compliance date beyond January 1, 1981. We were unable to issue the exemption at that time. As we noted in an April 11, 1978, letter, because the city was already part of an existing regional water system, the extended exemption compliance date is not available.

If you are able to provide us with a construction schedule leading to compliance by the statutory deadline of January 1, 1981, we can process your exemption. If it is documented that you cannot complete the necessary modifications by January 1, 1981, we have recently determined that we can still issue an exemption for the period prior to January 1, 1981. The exemption will indicate a valid construction schedule up to December 31, 1980, followed by a compressed schedule showing compliance by January 1, 1981. By that time, it is hoped that legislative extension of the 1981 statutory deadline will have occurred and EPA can simply extend the date. Otherwise, EPA could then issue a "compliance agreement" to be effective from January 1, 1981, through the completion of necessary modifications.

A compliance agreement is a negotiated agreement between a community and EPA. In essence, through the mechanism of the compliance agreement the EPA agrees to defer enforcement action provided that the city accomplishes some prearranged tasks resulting in necessary system modifications. The agreement would also include some interim control measures to assure that public health was adequately protected until the final system improvements are made. The compliance agreement would be in a different format than the exemption, but it is anticipated it would contain essentially the same requirements.

The advantage of an exemption during the period prior to January 1, 1981, is that the city is protected from private citizen suits (provided, of course, that the terms of the exemption are met). While the compliance agreement protects you from federal enforcement action, it does not provide this legal protection from citizen suit. It is for this reason, we recommend the issuance of the compliance agreement only after the exemption is no longer available to you.

Early last year (January 1978) it was requested that EPA evaluate your water system to see if a relaxation of the turbidity maximum contaminant level in accordance with Section 141.13(a) of the National Interim Primary Drinking Water Regulations was justified. As a result of that evaluation, we determined that the relaxation was not justified. For your convience, I have attached a copy of our letter dated February 15, 1978, notifying you of our determination.

In summary, upon your request, and with adequate documentation (see subpart F of the attached regulations), we are willing to consider the issuance of an exemption followed by a compliance agreement, if necessary. This resubmittal of information is necessary because of the time that has passed since our denial of your original request. Additionally, the original request was deficient in some of the areas outlined in the regulations.

I hope this letter clarifies the matter regarding resolution of the city's turbidity violations. Again, I am sorry for our delay and will attempt to avoid subsequent, unnecessary EPA delays.

If you have any questions, or need additional information, please contact me at 221-3250.

Sincerely,

Donald C. Gipe

Donald C. Gipe, Coordinator
Safe Drinking Water Program

Enclosures

cc: Mayor Joe McLaughlin, Pendleton

federal register

TUESDAY, JANUARY 20, 1976

PART II:

ENVIRONMENTAL PROTECTION AGENCY

National Interim Primary Drinking Water Regulations

Implementation

Subpart E—Variances Issued by the Administrator

Subpart F—Exemptions Issued by the Administrator

Subpart E—Variances Issued by the Administrator

§ 142.40 Requirements for a Variance.

(a) The Administrator may grant one or more variances to any public water system within a State that does not have primary enforcement responsibility from any requirement respecting a maximum contaminant level of an applicable national primary drinking water regulation upon a finding that:

(1) Because of characteristics of the raw water sources which are reasonably available to the system, the system cannot meet the requirements respecting the maximum contaminant levels of such drinking water regulations despite application of the best technology, treatment techniques, or other means, which the Administrator finds are generally available (taking costs into consideration); and

(2) The granting of a variance will not result in an unreasonable risk to the health of persons served by the system.

(b) The Administrator may grant one or more variances to any public water system within a State that does not have primary enforcement responsibility from any requirement of a specified treatment technique of an applicable national primary drinking water regulation upon a finding that the public water system applying for the variance has demonstrated that such treatment technique is not necessary to protect the health of persons because of the nature of the raw water source of such system.

§ 142.41 Variance Request.

A supplier of water may request the granting of a variance pursuant to this subpart for a public water system within a State that does not have primary enforcement responsibility by submitting a request for a variance in writing to the Administrator. Suppliers of water may submit a joint request for variances when they seek similar variances under similar circumstances. Any written request for a variance or variances shall include the following information:

(a) The nature and duration of variance requested.

(b) Relevant analytical results of water quality sampling of the system, including results of relevant tests conducted pursuant to the requirements of the national primary drinking water regulations.

(c) For any request made under § 142.40(a):

(1) Explanation in full and evidence of the best available treatment technology and techniques.

(2) Economic and legal factors relevant to ability to comply.

(3) Analytical results of raw water quality relevant to the variance request.

(4) A proposed compliance schedule, including the date each step toward compliance will be achieved. Such schedule shall include as a minimum the following dates:

(i) Date by which arrangement for alternative raw water source or improvement of existing raw water source will be completed.

(ii) Date of initiation of the connection of the alternative raw water source or improvement of existing raw water source.

(iii) Date by which final compliance is to be achieved.

(5) A plan for the provision of safe drinking water in the case of an excessive rise in the contaminant level for which the variance is requested.

(6) A plan for interim control measures during the effective period of variance.

(d) For any request made under § 142.40(b), a statement that the system will perform monitoring and other reasonable requirements prescribed by the Administrator as a condition to the variance.

(e) Other information, if any, believed to be pertinent by the applicant.

(f) Such other information as the Administrator may require.

§ 142.42 Consideration of Variance Request.

(a) The Administrator shall act on any variance request submitted pursuant to § 142.41 within 90 days of receipt of the request.

(b) In his consideration of whether the public water system is unable to comply with a contaminant level required by the national primary drinking water regulations because of the nature of the raw water source, the Administrator shall consider such factors as the following:

(1) The availability and effectiveness of treatment methods for the contaminant for which the variance is requested.

(2) Cost and other economic considerations such as implementing treatment, improving the quality of the source water or using an alternate source.

(c) In his consideration of whether a public water system should be granted a variance to a required treatment technique because such treatment is unnecessary to protect the public health, the Administrator shall consider such factors as the following:

(1) Quality of the water source including water quality data and pertinent sources of pollution.

(2) Source protection measures employed by the public water system.

§ 142.43 Disposition of a Variance Request.

(a) If the Administrator decides to deny the application for a variance, he shall notify the applicant of his intention to issue a denial. Such notice shall include a statement of reasons for the proposed denial, and shall offer the applicant an opportunity to present, within 30 days of receipt of the notice, additional information or argument to the Administrator. The Administrator shall make a final determination on the request within 30 days after receiving any such additional information or argument. If no additional information or argument is submitted by the applicant the application shall be denied.

(b) If the Administrator proposes to grant a variance request submitted pursuant to § 142.41, he shall notify the applicant of his decision in writing. Such notice shall identify the variance, the facility covered, and shall specify the period of time for which the variance will be effective.

(1) For the type of variance specified in § 142.40(a) such notice shall provide that the variance will be terminated when the system comes into compliance with the applicable regulation, and may be terminated upon a finding by the Administrator that the system has failed to comply with any requirements of a final schedule issued pursuant to § 142.44.

(2) For the type of variance specified in § 142.40(b) such notice shall provide that the variance may be terminated at any time upon a finding that the nature of the raw water source is such that the specified treatment technique for which the variance was granted is necessary to protect the health of persons or upon a finding that the public water system has failed to comply with monitoring and other requirements prescribed by the Administrator as a condition to the granting of the variance.

(c) For a variance specified in § 142.40 (a)(1) the Administrator shall propose a schedule for:

(1) Compliance (including increments of progress) by the public water system with each contaminant level requirement covered by the variance; and,

(2) Implementation by the public water system of such control measures as the Administrator may require for each contaminant covered by the variance.

(d) The proposed schedule for compliance shall specify dates by which steps towards compliance are to be taken, including at the minimum, where applicable:

(1) Date by which arrangement for an alternative raw water source or improvement of existing raw water source will be completed.

(2) Date of initiation of the connection for the alternative raw water source or improvement of the existing raw water source.

(3) Date by which final compliance is to be achieved.

(e) The proposed schedule may, if the public water system has no access to an alternative raw water source, and can effect or anticipate no adequate improvement of the existing raw water source, specify an indefinite time period for compliance until a new and effective treatment technology is developed at which time a new compliance schedule shall be prescribed by the Administrator.

(f) The proposed schedule for implementation of interim control measures during the period of variance shall specify interim treatment techniques, methods and equipment, and dates by which steps toward meeting the interim control measures are to be met.

(g) The schedule shall be prescribed by the Administrator within one year after the granting of the variance, subsequent to provision of opportunity for hearing pursuant to § 142.44.

§ 142.44 Public Hearings on Variances and Schedules.

(a) Before a variance or a schedule proposed by the Administrator pursuant to § 142.43 may take effect, the Administrator shall provide notice and opportunity for public hearing on the variance or schedule. A notice given pursuant to the preceding sentence may cover the granting of more than one variance and a hearing held pursuant to such notice shall include each of the variances covered by the notice.

(b) Public notice of an opportunity for hearing on a variance or schedule shall be circulated in a manner designed to inform interested and potentially interested persons of the proposed variance or schedule, and shall include at least the following:

(1) Posting of a notice in the principal post office of each municipality or area served by the public water system, and publishing of a notice in a newspaper or newspapers of general circulation in the area served by the public water system: and

(2) Mailing of a notice to the agency of the State in which the system is located which is responsible for the State's water supply program, and to other appropriate State or local agencies at the Administrator's discretion.

(3) Such notice shall include a summary of the proposed variance or schedule and shall inform interested persons that they may request a public hearing on the proposed variance or schedule.

(c) Requests for hearing may be submitted by any interested person other than a Federal agency. Frivolous or insubstantial requests for hearing may be denied by the Administrator. Requests must be submitted to the Administrator within 30 days after issuance of the public notices provided for in paragraph (b). Such requests shall include the following information:

(1) The name, address and telephone number of the individual, organization or other entity requesting a hearing;

(2) A brief statement of the interest of the person making the request in the proposed variance or schedule and of information that the requesting person intends to submit at such hearing;

(3) The signature of the individual making the request, or, if the request is made on behalf of an organization or other entity, the signature of a responsible official of the organization or other entity.

(d) The Administrator shall give notice in the manner set forth in paragraph (b) of this section of any hearing to be held pursuant to a request submitted by an interested person or on his own motion. Notice of the hearing shall also be sent to the persons requesting the hearing, if any. Notice of the hearing shall include a statement of the purpose of the hearing, information regarding the time and location for the hearing, and the address and telephone number of an office at which interested persons may obtain further information concerning the hearing. At least one hearing location specified in the public notice shall be within the involved State. Notice of hearing shall be given not less than 15 days prior to the time scheduled for the hearing.

(e) A hearing convened pursuant to paragraph (d) of this section shall be conducted before a hearing officer to be designated by the Administrator. The hearing shall be conducted by the hearing officer in an informal, orderly and expeditious manner. The hearing officer shall have authority to call witnesses, receive oral and written testimony and take such other action as may be necessary to assure the fair and efficient conduct of the hearing. Following the conclusion of the hearing, the hearing officer shall forward the record of the hearing to the Administrator.

(f) The variance or schedule shall become effective 30 days after notice of opportunity for hearing is given pursuant to paragraph (b) if no timely request for hearing is submitted and the Administrator does not determine to hold a public hearing on his own motion.

§ 142.45 Action After Hearing.

Within 30 days after the termination of the public hearing held pursuant to §142.44, the Administrator shall, taking into consideration information obtained during such hearing and other relevant information, confirm, revise or rescind the proposed variance or schedule.

§ 142.46 Alternative Treatment Techniques.

The Administrator may grant a variance from any treatment technique requirement of a national primary drinking water regulation to a supplier of water, whether or not the public water system for which the variance is requested is located in a State which has primary enforcement responsibility, upon a showing from any person that an alternative treatment technique not included in such requirement is at least as efficient in lowering the level of the contaminant with respect to which such requirements was prescribed. A variance under this paragraph shall be conditioned on the use of the alternative treatment technique which is the basis of the variance.

Subpart F—Exemptions Issued by the Administrator

§ 142.50 Requirements for an Exemption.

The Administrator may exempt any public water system within a State that does not have primary enforcement responsibility from any requirement respecting a maximum contaminant level or any treatment technique requirement, or from both, of an applicable national primary drinking water regulation upon a finding that:

(a) Due to compelling factors (which may include economic factors), the public water system is unable to comply with such contaminant level or treatment technique requirement;

(b) The public water system was in operation on the effective date of such contaminant level or treatment technique requirement; and

(c) The granting of the exemption will not result in an unreasonable risk to health.

§ 142.51 Exemption Request.

A supplier of water may request the granting of an exemption pursuant to this subpart for a public water system within a State that does not have primary enforcement responsibility by submitting a request for exemption in writing to the Administrator. Suppliers of water may submit a joint request for exemptions when they seek similar exemptions under similar circumstances. Any written request for an exemption or exemptions shall include the following information:

(a) The nature and duration of exemption requested.

(b) Relevant analytical results of water quality sampling of the system, including results of relevant tests conducted pursuant to the requirements of the national primary drinking water regulations.

(c) Explanation of the compelling factors such as time or economic factors which prevent such system from achieving compliance.

(d) Other information, if any, believed by the applicant to be pertinent to the application.

(e) A proposed compliance schedule, including the date when each step toward compliance will be achieved.

(f) Such other information as the Administrator may require.

§ 142.52 Consideration of an Exemption Request.

(a) The Administrator shall act on any exemption request submitted pursuant to § 142.51 within 90 days of receipt of the request.

(b) In his consideration of whether the public water system is unable to comply due to compelling factors, the Administrator shall consider such factors as the following:

(1) Construction, installation, or modification of treatment equipment or systems.

(2) The time needed to put into operation a new treatment facility to replace an existing system which is not in compliance.

(3) Economic feasibility of compliance.

§ 142.53 Disposition of an Exemption Request.

(a) If the Administrator decides to deny the application for an exemption, he shall notify the applicant of his intention to issue a denial. Such notice shall include a statement of reasons for the proposed denial, and shall offer the applicant an opportunity to present, within 30 days of receipt of the notice, additional information or argument to the Administrator. The Administrator shall make a final determination on the request within 30 days after receiving any such additional information or argument. If no additional information or argument is submitted by the applicant, the application shall be denied.

(b) If the Administrator grants an exemption request submitted pursuant to § 142.51, he shall notify the applicant of his decision in writing. Such notice shall identify the facility covered, and shall specify the termination date of the exemption. Such notice shall provide that the exemption will be terminated when the system comes into compliance with the applicable regulation, and may be terminated upon a finding by the Administrator that the system has failed to comply with any requirements of a final schedule issued pursuant to § 142.55.

(c) The Administrator shall propose a schedule for:

(1) Compliance (including increments of progress) by the public water system with each contaminant level requirement and treatment technique requirement covered by the exemption; and

(2) Implementation by the public water system of such control measures as the Administrator may require for each contaminant covered by the exemption.

(d) The schedule shall be prescribed by the Administrator within one year after the granting of the exemption, subsequent to provision of opportunity for hearing pursuant to § 142.54.

§ 142.54 Public Hearings on Exemption Schedules.

(a) Before a schedule proposed by the Administrator pursuant to § 142.53 may take effect, the Administrator shall provide notice and opportunity for public hearing on the schedule. A notice given pursuant to the preceding sentence may cover the proposal of more than one such schedule and a hearing held pursuant to such notice shall include each of the schedules covered by the notice.

(b) Public notice of an opportunity for hearing on an exemption schedule shall be circulated in a manner designed to inform interested and potentially interested persons of the proposed schedule, and shall include at least the following:

(1) Posting of a notice in the principal post office of each municipality or area served by the public water system, and publishing of a notice in a newspaper or newspapers of general circulation in the area served by the public water system.

(2) Mailing of a notice to the agency of the State in which the system is located which is responsible for the State's water supply program and to other appropriate State or local agencies at the Administrator's discretion.

(3) Such notices shall include a summary of the proposed schedule and shall inform interested persons that they may request a public hearing on the proposed schedule.

(c) Requests for hearing may be submitted by any interested person other than a Federal agency. Frivolous or insubstantial requests for hearing may be denied by the Administrator. Requests must be submitted to the Administrator within 30 days after issuance of the public notices provided for in paragraph (b). Such requests shall include the following information:

(1) The name, address and telephone number of the individual, organization or other entity requesting a hearing;

(2) A brief statement of the interest of the person making the request in the proposed schedule and of information that the requesting person intends to submit at such hearing; and

(3) The signature of the individual making the request, or, if the request is made on behalf of an organization or other entity, the signature of a responsible official of the organization or other entity.

(d) The Administrator shall give notice in the manner set forth in paragraph (b) of this section of any hearing to be held pursuant to a request submitted by an interested person or on his own motion. Notice of the hearing shall also be sent to the person requesting the hearing, if any. Notice of the hearing shall include a statement of the purpose of the hearing, information regarding the time and location for the hearing, and the address and telephone number of an office at which interested persons may obtain further information concerning the hearing. At least one hearing location specified in the public notice shall be within the involved State. Notice of hearing shall be given not less than 15 days prior to the time scheduled for the hearing.

(e) A hearing convened pursuant to paragraph (d) of this section shall be conducted before a hearing officer to be designated by the Administrator. The hearing shall be conducted by the hearing officer in an informal, orderly and expeditious manner. The hearing officer shall have authority to call witnesses, receive oral and written testimony and take such action as may be necessary to assure the fair and efficient conduct of the hearing. Following the conclusion of the hearing, the hearing officer shall forward the record of the hearing to the Administrator.

§ 142.55 Final Schedule.

(a) Within 30 days after the termination of the public hearing pursuant to § 142.54, the Administrator shall, taking into consideration information obtained during such hearing, revise the proposed schedule as necessary and prescribe the final schedule for compliance and interim measures for the public water system granted an exemption under § 142.52.

(b) Such schedule shall require compliance by the public water system with each contaminant level and treatment technique requirement prescribed by:

(1) Interim national primary drinking water regulations pursuant to Part 141 of this chapter, by no later than January 1, 1981; and

(2) Revised national primary drinking water regulations pursuant to Part 141 of this chapter, by no later than seven years after the effective date of such regulations.

(c) If the public water system has entered into an enforceable agreement to become a part of a regional public water system, as determined by the Administrator, such schedule shall require compliance by the public water system with each contaminant level and treatment technique requirement prescribed by:

(1) Interim national primary drinking water regulations pursuant to Part 141 of this chapter, by no later than January 1, 1983; and

(2) Revised national primary drinking water regulations pursuant to Part 141 of this chapter, by no later than nine years after the effective date of such regulations.

UNITED STATES ENVIRONMENTAL PROTECTION AGENCY

OREGON OPERATIONS OFFICE
522 S.W. 5TH AVENUE
YEON BUILDING, 2ND FLOOR
PORTLAND, OREGON 97204

REPLY TO ATTN OF: 10000

FEB 1? ?8

John R. Molsness, Director of Public Works
City of Pendleton
P. O. Box 190
Pendleton, Oregon 97801

Dear Mr. Molsness:

In my letter of January 20, I indicated that I would review the classification of your source facilities based on a survey of those facilities by my staff. On January 31, Bill Titus met with you and conducted the survey. A copy of his report is enclosed.

Based on Mr. Titus' findings, I have classified your source facilities at Thornhollow as a surface source. It is clear that the water passes through and is, to some extent, stored in an unconsolidated deposit of coarse sand and gravel before it enters your source facilities. This deposit of sand and gravel is not protected from contamination by surface runoff and the water contained in it must be considered to be surface water.

As I explained in my letter of December 9, 1977, an adjustment to the maximum contaminant level for turbidity is available to public water systems which can demonstrate that turbidity levels higher than the maximum contaminant level do not interfere with the disinfection process or the microbiological analysis procedures. The degree to which this can be demonstrated is a function of both the nature of the particles which cause the turbidity and the degree with which that nature can be predicted. The watershed and your source facilities are vulnerable to contamination from surface runoff. This runoff is subject to contamination by residential, agricultural and recreational users within the watershed over which you have little control. Under these conditions, the nature of the turbidity particles cannot be predicted with any reasonable certainty and the probability that they include the organic debris which is known to interfere with disinfection and microbiological analyses is high. Therefore, I cannot justify an adjustment to the maximum contaminant level for turbidity for your system.

The National Interim Primary Drinking Water Regulations (copy enclosed) establish monitoring requirements and maximum contaminant levels for public water systems. You should begin a monitoring program designed to meet the requirements established for systems which utilize surface water sources. These requirements involve daily turbidity measurements, annual organic and inorganic chemical monitoring and radiological monitoring every four years, in addition to the routine bacteriological monitoring which you are currently conducting. Please forward the results of your monitoring program, on a monthly basis, to:

Environmental Protection Agency
Mail Stop 412
1200 Sixth Avenue
Seattle, Washington 98101

Thank you for your cooperation during the recent survey. If you have any questions or comments, or if you require any assistance, please do not hesitate to contact Mr. Titus at the address listed on the letterhead, or at 221-3250.

Sincerely,

John Vlastelicia

John Vlastelicia, Director
Oregon Operations Office

cc: Umatilla County Health Department

REFERENCES

CITED SOURCES

1. Water System Feasibility Study, Confederated Tribes of Umatilla Indian Reservation, Anderson-Perry & Associates, Inc., July, 1979.

2. Artificial recharge through wells in the Salem Heights area, Salem, Oregon, March 1 to October 17, 1977, letter report of October 26, 1977, C & G Engineering, Salem, Oregon.

3. Geomorphology, Hydrology, Stream Flow Management in Upper Umatilla River Basin, Oregon, by Howard D. Kopp, R. L. Albrook Hydraulics Laboratory, Department of Civil and Environmental Engineering, Washington State University, Pullman, Washington, September 30, 1977.

4. Water Resources of the Umatilla Indian Reservation, by J. B. Gonthier and D. D. Harris, U. S. Geological Survey Water Resources Investigation 77-3. 1977.

5. Water Needs Study, Umatilla Indian Reservation, by CH_2M Hill, Incorporated, May, 1976.

6. Plan for Growth, Overall Economic Development Program, Confederated Tribes of the Umatilla Indian Reservation, by Program Planning Committee and Tribal Development Office, July, 1974.

7. Planning for the Umatilla Indian Reservation, Initial Comprehensive Planning Investigation, by CH_2M Hill, Incorporated, December, 1973.

8. Geology and Groundwater Resources of the Walla Walla River Basin, Washington-Oregon, by R. C. Newcomb, State of Washington Division of Water Resources, 1965.

9. Geology and Groundwater of the Umatilla River Basin, Oregon, by G. M. Hogenson, Geological Survey Water Supply Paper 1620, 1964.

10. Umatilla River Basin, State Water Resources Board, June, 1963.

11. Storage of Groundwater Behind Subsurface Dams in the Columbia River Basalt, Washington, Oregon and Idaho, by R. C. Newcomb, Geological Survey Professional Paper 383-A, 1961.

12. Occurrence of Groundwater in the Columbia River Basalt Near Pilot Rock, Oregon, by Stuart G. Brown, Geological Survey Open-File Report, 1955.

13. City of Pendleton Water Studies.

November, 1971	Wallulis & Associates, Inc.	South McKay Creek
January, 1964	CH_2M Hill	Report Part III
October, 1963	CH_2M Hill	Report Part II
July, 1962	CH_2M Hill	Report Part I
May, 1960	CH_2M Hill	South and Southwest Area
February, 1958	CH_2M Hill	Water Supply Investigation
November, 1954	CH_2M Hill	Extensions to Southwest Area
March, 1952	CH_2M Hill	Water Supply Investigation
September, 1950	CH_2M Hill	Water Supply Investigation

14. Artificial Recharge of a Well Tapping Basalt Aquifers, Walla Walla Area, Washington, by Charles E. Price, State of Washington Division of Water Resources, 1960.

15. Artificial Recharge Through a Well Tapping Basalt Aquifers, Walla Walla Area, Washington, by Charles E. Price, Geological Survey Water Paper 1594-A, 1961.

16. Artificial Recharge Through a Well Tapping Basalt Aquifers at The Dalles, Oregon, by B. L. Foxworthy and C. T. Bryant, Geological Survey Water Supply Paper 1594-E, 1967.

17. Hydrologic Conditions and Artificial Recharge Through a Well in the Salem Heights Area of Salem, Oregon, by Bruce Foxworthy, Geological Survey Water Supply Paper 1594-F, 1970.

18. Oregon Water Resources Department, data on area wells.

19. U. S. Department of Commerce, climatological data.