city of grants pass

ENGINEERING REPORT
OF THE

# WATER DISTRIBUTION SYSTEM

GRANTS PASS, OREGON

CH2M HILL

FEBRUARY 1979

CH2M
HILL
engineers
planners
economists
scientists

7 February 1979
C12227.A0

City of Grants Pass
101 N.W. "A" Street
Grants Pass, Oregon 97526

Ladies and Gentlemen:

Subject: Engineering Report of the
Water Distribution System

CH2M HILL is pleased to submit our engineering report which summarizes the findings and recommendations for the Grants Pass water distribution system in accordance with your authorization of 2 October 1978.

This report proposes improvements to the existing water distribution system and future expansions to serve the urbanizing areas outside the present city limits. The investigation has determined the water requirements and developed a plan to serve the future population to year 2000 residing within both the service areas contained by the existing city limits and the "draft urban growth boundary". The service area, land uses and occupancy, and the projected population conforms with the proposed policies and guidelines of the "draft" comprehensive plan.

We wish to express our appreciation for the opportunity to furnish this engineering service and to thank Mr. Ron Bergman and many other City employees for their cooperation and assistance.

Respectfully submitted,

Charles R. Meek

Charles R. Meek
Project Administrator

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# ENGINEERING REPORT
# OF THE
# WATER DISTRIBUTION SYSTEM

CITY OF GRANTS PASS
OREGON

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Date: February 1979

Project No. C12227.A0

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# I

# INTRODUCTION

# Chapter 1
# INTRODUCTION

## GENERAL

The Comprehensive Plan including the establishment of the urban growth boundary and land uses will provide for the orderly development of the Grants Pass area. In conjunction with this planning process, the City of Grants Pass has recognized the need to study the water distribution system to plan for its most efficient use and expansion, and to consider the technical aspects of providing water for the urbanizing area outside the present city limits. In October 1978, CH2M HILL was authorized to complete a study of the water distribution system.

This report summarizes the findings and recommendations of the investigation which includes population projections, estimates of water requirements, evaluation of the existing water system and the recommended system improvements. The study includes growth projections within the urban growth boundary to the year 2000. The proposed new facilities, including the pump stations, reservoirs and pipelines, are scheduled for development in phases as needed to accommodate growth. The area within the urban growth boundary taken from the draft document includes a land vacancy factor; however, some areas may develop to full occupancy during the 20-year planning period.

The water distribution system improvements presented herein will give the City a water system that will provide adequate and reliable service to all of the users throughout the service area. An assessment of the proposed improvements will determine the impact of extension into the urbanizing fringe areas outside the city limits but within the "draft urban growth boundary".

A product of this study is the model of the water distribution system. The model preparation required an inventory of the distribution system, including pipe locations, size, length, type, and age. In addition, land-use data was reviewed to determine the allocation of water demand and fire protection requirements throughout the service area. This information was used to produce the model, followed by field tests to verify the model's accuracy. The verified model was used to test various flow conditions, such as maximum hour demand and reservoir refill, and fire flows were imposed on the system model. The pipeline flows and pressures shown by the simulation tests were used to evaluate the existing water distribution system. Additional computer runs were used to determine the distribution pipeline and reservoir requirements for the projected growth conditions.

The water distribution system model can be used to evaluate phasing proposals, new developments, and changes in the area served by simulating the appropriate condition.

## PREVIOUS STUDIES

Previous engineering studies of the water system are:

Cornell, Howland, Hayes and Merryfield, Corvallis, Oregon, An Engineering Report on a Water System Investigation, May 1960.

Cornell, Howland, Hayes and Merryfield, Corvallis, Oregon, A Report on a Preliminary Engineering Study of a High Level Water System, July 1966.

Brown and Caldwell, Eugene, Oregon, Water System Study, May 1974.

## DOCUMENTS AND REPORTS

The following documents and reports were reviewed in the preparation of this study.

Longford and Steward, A General Plan for the Urbanizing Area and the City of Grants Pass, September 1969.

Grants Pass Urbanizing Area Population Projections, prepared jointly by the planning staffs of the City of Grants Pass and Josephine County, March 1978.

Draft...Grants Pass Urban Growth Boundary and Urban Services Policies, prepared jointly by the planning staffs of the City of Grants Pass and Josephine County, July 1978.

## ACKNOWLEDGEMENT

We appreciate the assistance furnished by many individuals on the City of Grants Pass staff in the preparation of this report. Their suggestions and contributions were essential, and their help is gratefully acknowledged.

# II

# WATER REQUIREMENTS

## Chapter II
## WATER REQUIREMENTS

### STUDY AREA

The area studied is shown on Figure II-1. This area corresponds with the urban growth boundary as presented in the Draft...Grants Pass Urban Growth Boundary and Urban Service Policies prepared by the planning staffs of the City of Grants Pass and Josephine County. The area within this "draft urban growth boundary" contains 7820 developable acres. The present water distribution service area, shown in Map No. 1, covers approximately 3,900 acres.

### WATER USAGE

The land uses for the study area were reviewed to determine the water needs. The "draft" comprehensive plan document presents the zoning within the city limits and anticipated zoning within the urbanizing area. Current zoning is shown on Map No. 2. Unit water consumption values developed in the 1966 and 1974 referenced studies were used to predict the future water requirements. The water use characteristics developed are summarized in Table II-1.

Table II-1
Water Use - Average Daily Unit Consumption Values

| | |
|---|---|
| Residential Users | 148 gallons per capita per day |
| Commercial Users | 3000 gallons per acre per day |
| Low Water Use Industrial Land | 670 gallons per acre per day |
| High Water Use Industrial Land | Actual water use, 1973-1978 |
| Institutions and Public Lands | 1000 gallons per acre per day |

### POPULATION ESTIMATES

The "draft urban services policy" of the comprehensive plan has determined the long-range population growth within the urban growth boundary by the year 2000 to be 36,600, (low-range target population).

The full site development for the area within the "draft urban growth boundary" (UGB) would yield a population of approximately 46,000 people. In this study, a 72 percent development factor for that land presently undeveloped was used for establishing demands within the service area. This factor corresponds with the additional acreage provided by the "draft urban services policy" to give developers and buyers the option of market selection. The full site development population by service area was used to size facilities and structures which are not built in phases because of economic or other factors; these facilities include transmission pipelines, storage reservoirs, and pump stations. The mid-range controlled growth target population of 39,000 as presented in the "Urbanizing Area Population Projections" document was used to size facilities which can be constructed in phases. This mid-range target population was used rather than the low-range target population to provide a degree of safety in the sizing of facilities.

## BASIS OF DESIGN

Three water usage rate variations are generally used in the design of water system facilities. These are the average annual demand, maximum day demand, and peak hour demand.

The seasonal variation of water demand is plotted on Figure II-2. The monthly water use ranges from a low of 42 percent to a high of 193 percent of the average annual demand. The monthly maximum day demands, also plotted on Figure II-2, range from 244 to 270 percent of average annual demand. The peak hour demand was determined to be 400 percent of the average annual demand. The present average annual demand is 253 gallons per day per capita.

The average annual day water production and the average annual day water metered are also plotted in Figure II-2. The shaded difference in these values is the amount of unmetered water, which consists of system losses such as leakage, overflow at reservoirs, park irrigation, hydrant flushing and other unaccounted for uses.

The amount of unmetered water is presently near 10 percent of the water produced. This amount of unaccounted water is within the normal range for cities in the Pacific Northwest, and further reduction may not be economically justified.

Figure II-3 illustrates the generalized hourly variation of a typical system demand as a function of maximum day demand. As shown, the maximum hour demand is approximately 150 percent of maximum day demand. This is thought to be a reasonable representation of the Grants Pass daily pattern of water usage.

The average annual per capita per day water usage and the water usage rates discussed above were used in conjunction with the design population projections to develop total system water requirements. These requirements are presented

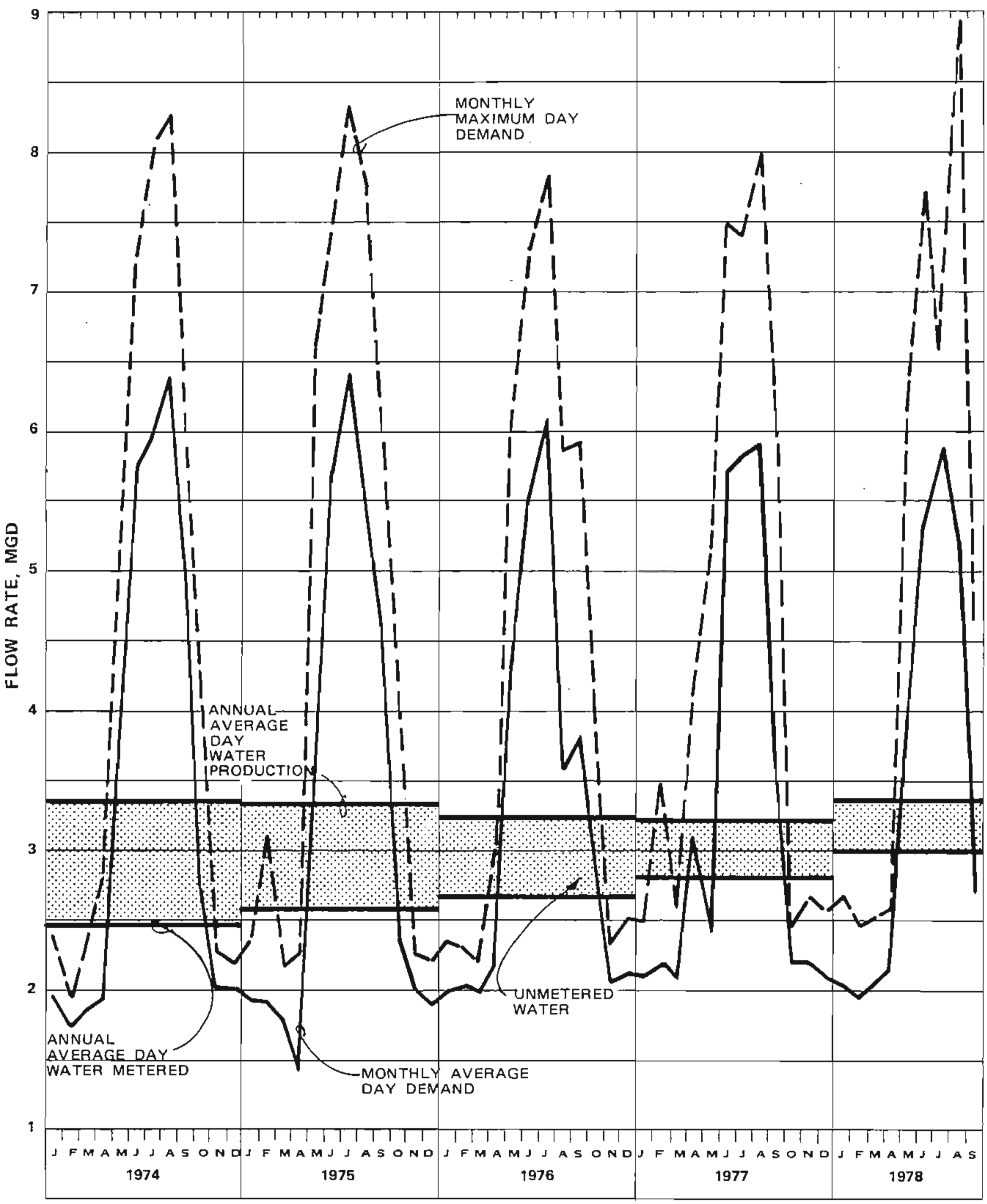


FIGURE II - 2

WATER PRODUCTION AND USAGE
(1974 THROUGH 1978)

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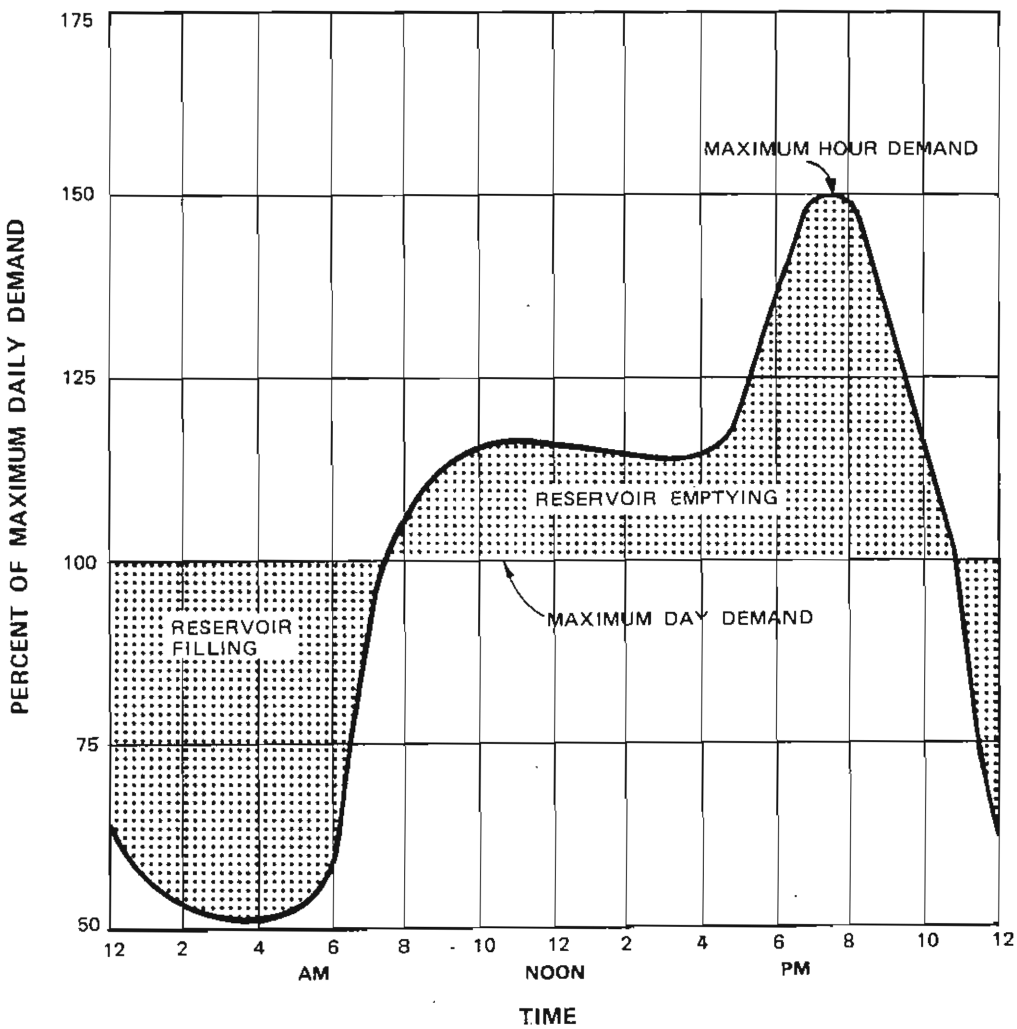


FIGURE II-3

TYPICAL HOURLY VARIATION OF SYSTEM DEMAND
DURING DAY OF MAXIMUM USE

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in Table II-2. The water demands in Columns 1 and 2, are based on the low- and mid-range target populations, together with the corresponding per capita use demands. The water demand for a population of 46,000, estimated for full site development, is shown in Column 3. Column 4 shows the water demand based upon full site development with per capita consumption values.

For this study the year 2000 average annual day demand is 10 mgd, and the full site development average annual day demand is 11.6 mgd. Tables II-3 and II-4 show the incremental average annual day demand through year 2000 for the service areas of the "draft urban growth boundary" and present city limits, respectively. The projections assume a uniform increase in population.

Figure II-4 shows the present and projected water requirements for the users within the "draft urban growth boundary" service area and also the service area contained within the present Grants Pass city limits. An expansion of the system demands based upon a uniform increase is also shown. This expansion will develop as the City extends water service to the urbanizing area outside the present city limits.

TABLE II-2
PROJECTED WATER DEMANDS
Year 2000, Urban Growth Boundary Service Area
(in million gallons per day)

| DESCRIPTION | COLUMN NO. | | | |
|---|---|---|---|---|
| | (1) | (2) | (3) | (4) |
| Average annual demand | 9.26 | 9.87 | 11.64 | 11.63 |
| Average day demand during maximum month | 17.87 | 19.05 | 22.47 | 22.45 |
| Maximum day demand | 25.00 | 26.65 | 31.43 | 31.40 |
| Peak hour demand | 37.04 | 39.48 | 46.56 | 46.52 |

1. Urban Growth Boundary low-range target population year 2000, 36,600 with per capita consumption value.
2. Urban Growth Boundary mid-range target population year 2000, 39,000 with per capita consumption values.
3. Full site development (present land-use ratios), population 46,000, with per capita consumption values.
4. Full site development (present land-use ratios), population 46,000, with unit consumption values.

TABLE II-3
PROJECTED WATER DEMANDS
Five-Year Increments
Urban Growth Boundary Service Area
(in million gallons per day)

| | 1980 | 1985 | 1990 | 1995 | 2000 |
|---|---|---|---|---|---|
| Average annual demand | 7.29 | 8.10 | 8.73 | 9.34 | 9.87 |
| Average day demand during maximum month | 14.07 | 15.63 | 16.85 | 18.02 | 19.05 |
| Maximum day demand | 19.68 | 21.87 | 23.58 | 25.22 | 26.65 |
| Peak hour demand | 29.15 | 32.39 | 34.92 | 37.35 | 39.48 |
| MID-RANGE TARGET POPULATION | 28,000 | 32,000 | 34,500 | 36,900 | 39,000 |

TABLE II-4
PROJECTED WATER DEMANDS
Five-Year Increments
Present Grants Pass City Limits Service Area
(in million gallons per day)

| | 1980 | 1985 | 1990 | 1995 | 2000 |
|---|---|---|---|---|---|
| Average annual demand | 3.89 | 4.25 | 4.61 | 4.97 | 5.33 |
| Average day demand during maximum month | 7.51 | 8.20 | 8.90 | 9.59 | 10.29 |
| Maximum day demand | 10.50 | 11.48 | 12.45 | 13.42 | 14.39 |
| Peak hour demand | 15.56 | 17.00 | 18.44 | 19.88 | 21.32 |
| MID-RANGE TARGET POPULATION | 15,358 | 16,786 | 18,215 | 19,643 | 21,071 |

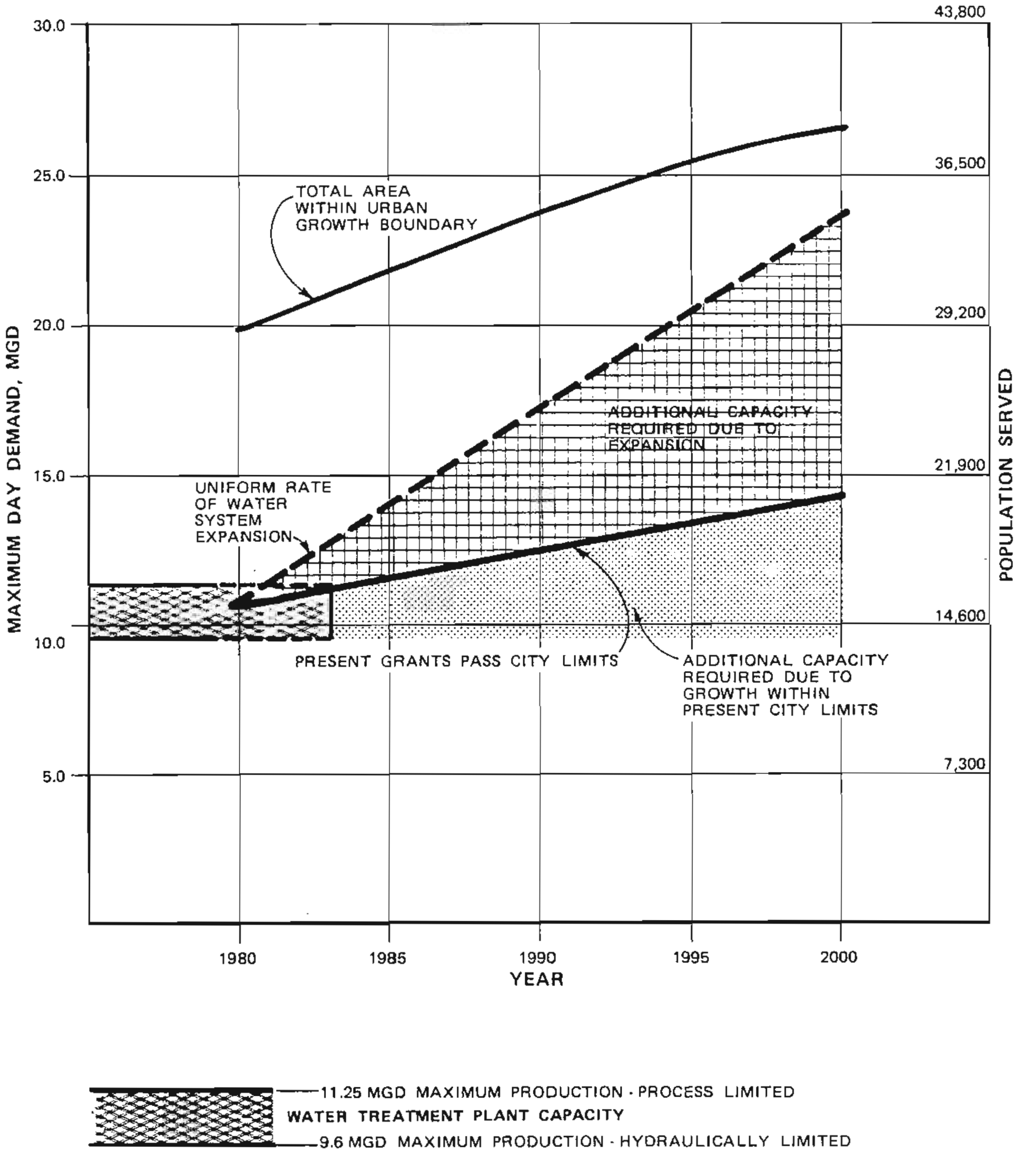


FIGURE II - 4

PROJECTED WATER REQUIREMENTS

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# III

# DISTRIBUTION SYSTEM

## Chapter III
## DISTRIBUTION SYSTEM

### EXISTING SYSTEM

### SERVICE LEVELS

The existing distribution system has three service levels. The first level, Zone 1, elevations 900 through 1020, serves the largest area and has the greatest water demand. The service pressures throughout the first level usually range from 32 psi to 90 psi; however, pressures up to 108 psi do occur during reservoir refill periods. The first level is supplied from the "M" Street Water Treatment Plant at a maximum rate of approximately 6800 gpm.

The second level, Zone 2, serves an area north of Birch Street, Evelyn Avenue, and Madrone Street from elevations 1020 through 1140. Zone 2A is a small area located, within the second level, north of Evelyn Avenue between Prospect Avenue and Memorial Drive. It serves elevations 960-1035. This zone is fed through two pressure-reducing valves, one located near the intersection of Manzanita Avenue and Hawthorne Street and the other near the intersection of Savage and Ninth streets. Zones 2 and 2A are supplied by the Lawnridge and Madrone pump stations and Reservoir Number 4.

The third level, Zone 3, is served by three hydropneumatic pump systems, i.e. Hefley, Starlight, and Woodson Pump Stations. Also, two fire pump stations, Hawthorne Pump Station and Champion Pump Station supply fire flows to Zone 2 service area.

Table III-1 lists the existing pump stations, the service level served, rated capacity, and operating information.

### STORAGE

Four storage reservoirs with a total capacity of 5.45 million gallons provide the system storage. Three of these reservoirs, located on the west side near Woodson Park, serve the first level with 4.7 million gallons. The overflow elevation of these reservoirs is 1108.5.

Reservoir Number 4, with an overflow elevation of 1240.0, is located east of the city near the intersection of Interstate Highway 5 and Savage Street. This 750,000-gallon reservoir serves the second level.

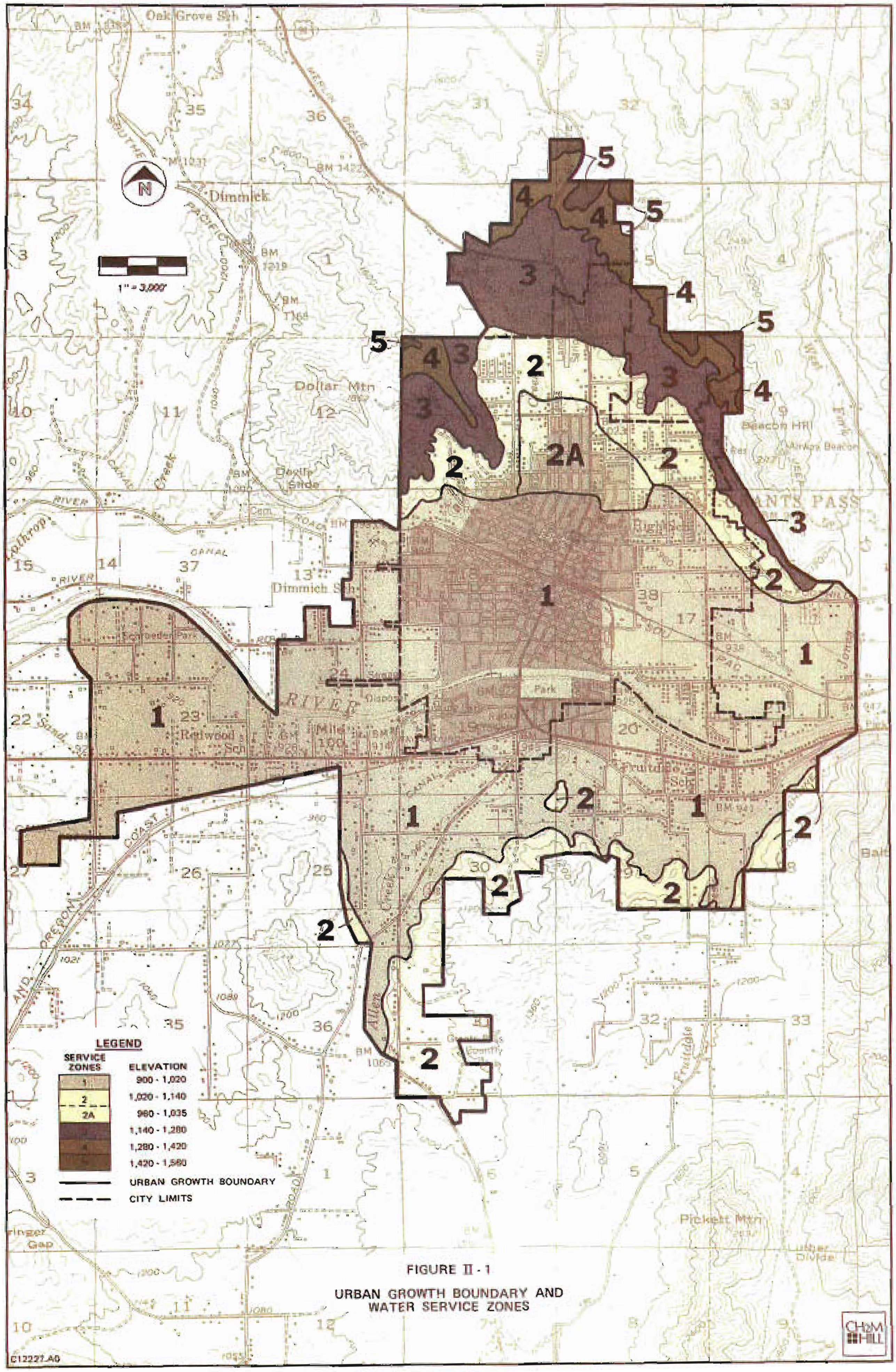



FIGURE II - 1

URBAN GROWTH BOUNDARY AND WATER SERVICE ZONES

Table III-I
EXISTING PUMP STATIONS

| Pump Station | Zone Service Level | Rated Capacity | Pump Discharge Head(Ft) | Operating Information |
|---|---|---|---|---|
| Lawnridge | 2 | 2 pumps, 1000 gpm ea.<br>1 pump, 400 gpm | 120 | Manual control at "M" St. WTP; override controls at pump station; |
| Madrone | 2 | 1 pump, 600 gpm<br>1 pump, 300 gpm | 140 | Controlled by differential pressure, manual override at "M" St. WTP; space for third pump |
| Hefley | 3,4 | 1 pump, 300 gpm | 140 | 1000-gal. hydropneumatic tank; controlled by differential pressure |
| Starlight | 3 | 1 pump, 200 gpm | 160 | 2000-gal. hydropneumatic tank; controlled by differential pressure |
| Woodson | 2 | 1 pump, 150 gpm | 140 | 500-gal. hydropneumatic tank; controlled by differential pressure |
| Hawthrone | 2 | 1 pump, 1500 gpm | 175 | Fire pump for Grants Pass Bowl; activated by flow switch |
| Champion | 2 | 1 pump, 1500 gpm | 185 | Fire pump for Champion Knitware Co.; activated by flow switch |

## PRIVATE WATER SYSTEMS

Several small private water systems presently exist within the "draft urban growth boundary". Some of these systems are shown on Map 1. These systems rely upon wells for their supply, and their distribution piping systems consist of small diameter pipe. The system piping is undersized and cannot be used effectively in the Grants Pass water system.

## FUTURE REQUIREMENTS

As the population and service area grows, the existing supply, distribution, and storage facilities must be expanded to meet the increased water requirements. The future system requirements are summarized in the following paragraphs. Supply demands during the summer are near the capacity of the "M" Street Water Treatment Plant. The proposed Parkdale Drive Water Treatment Plant will be required by about 1982, depending upon service policies and the rate of expansion.

### Distribution

The existing distribution system has been fully described in the referenced engineering studies. As reported, refilling the first-level reservoirs causes excessive pressure at the water treatment plant. Figure III-1 shows the pressure and pumping rates with respect to the existing distribution system capacity. The system resistance increases the pressure to 108-109 psi at a 6600-gpm pumping rate. Higher pressures cannot be imposed on the distribution system; therefore the plant capacity is limited. During high summer demands the reservoirs may not be refilled each day. Because of this limitation, low pressures and unsatisfactory service occur during subsequent days. Low pressures are now experienced along Morgan Lane, Crescent Drive, North Hawthorne Avenue, and Vine Street.

### Storage

The existing 5.45 million gallon storage is inadequate. The recommended required storage total is 8.1 million gallons: 4.4 million gallons for emergency reserve, 2.2 million gallons for equalizing, and 1.5 million gallons for fire fighting. Under the present conditions, the reserve storage would only serve the maximum day demands for about 2 1/2 hours.

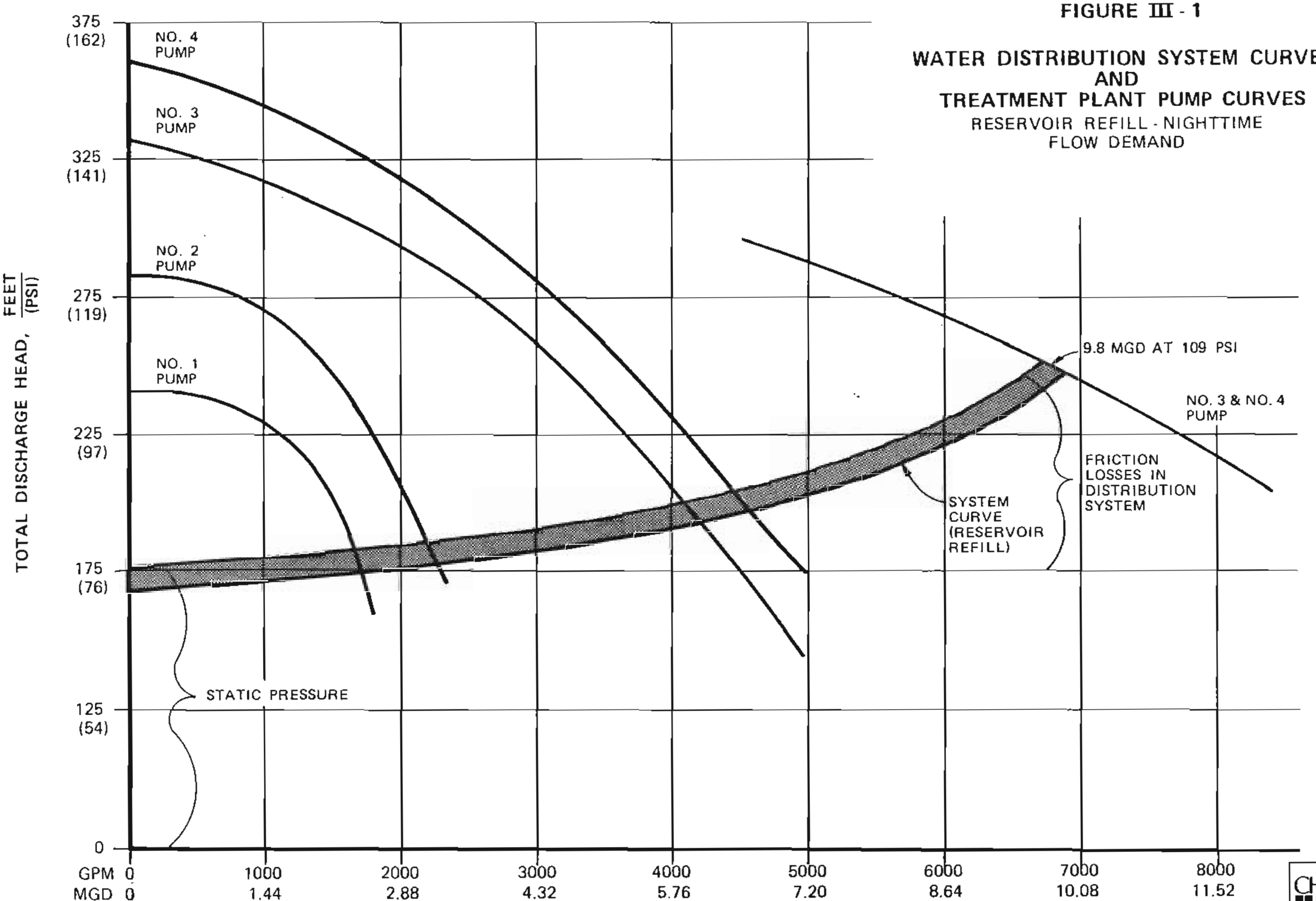
## RECOMMENDED SYSTEM

To meet the present service area requirements and to provide service to future users, a plan was developed using the computer model to simulate the system. The plan assumes the

## FIGURE III - 1

### WATER DISTRIBUTION SYSTEM CURVE AND TREATMENT PLANT PUMP CURVES

RESERVOIR REFILL - NIGHTTIME FLOW DEMAND

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"M" Street Water Treatment Plant and the proposed Parkdale Drive Water Treatment plant as shown on Map 1 would be in service. The "M" Street Water Treatment Plant is capable of producing 8.65 mgd with the largest filter out of service and has a maximum hydraulic rate of 11.25 mgd. These maximum rates account for 2 percent backwash flow but do not allow for any maintenance downtime. The "M" Street Plant has two large high service supply pumps. The future Parkdale Drive Water Treatment Plant, planned for an initial capacity of 18.0 mgd, together with the "M" Street Water Treatment Plant, will meet the projected maximum day demand of 26.65 mgd in Year 2000.

## WATER DISTRIBUTION SYSTEM

### Distribution Design Criteria

The capacity of a distribution system depends on the location and size of all components including pipelines, storage reservoirs, and supply facilities. A pipeline of a given size has a capacity that varies depending upon the pressure available to cause the water to flow. Generally, ample water and pressure is available at locations close to pump stations and reservoirs. As the distance from reservoirs and pump stations increases, the resistance to the flow reduces the available service pressure; thus pipeline diameters must vary depending upon the flow rate, the network of piping, and the distance from storage reservoirs or pumps.

A properly designed and operated distribution system should be capable of supplying the necessary quantity of water to any given location under all service conditions. Generally, three service conditions are reviewed to determine the basis for the design of distribution systems. These service conditions are:

- Supplying the peak-hour demand to the entire service area while maintaining minimum pressures at the upper elevations of the service level. Normally, reservoir levels should not be more than 10 feet below overflow during the peak-hour demand.

- Furnishing the required fire flows to any given location at the same time the maximum daily demand is supplied to the entire service area. Minimum pressure at the site of the assumed fire flow is 20 psi. Reservoir levels should not be more than 10 feet below overflow. Design criteria assumes only one fire at a time. Table III-2 is a list of the recommended fire flows.

- Refilling the reservoirs during low demand periods. Pipelines between pump stations and storage reservoirs are sized to avoid excessive pressures while the pumps are refilling the reservoirs. Pump discharge pressures should not exceed 100 psi.

Table III-2 Recommended Fire Flows

| Area classification | Fire flow, gpm | Duration of flow, hours | Volume of water req'd mg |
|---|---|---|---|
| Residential | | | |
| Suburban[a)] | 1000 | 2 | 0.12 |
| Urban, low density | 1000 | 2 | 0.12 |
| Urban, medium density[b)] | 2500 | 2 | 0.30 |
| Urban, high density[c)] | 3500 | 3 | 0.63 |
| Commercial | | | |
| Central business district | 4500 | 4 | 1.08 |
| Tourist | 3000 | 3 | 0.54 |
| Limited commercial | 3000 | 3 | 0.54 |
| Strip commercial | 2500 | 2 | 0.30 |
| Industrial[d)] | 5000 | 5 | 1.50 |
| Institutional | | | |
| Schools | 3500 | 3 | 0.63 |
| Hospitals | 4500 | 4 | 1.08 |

a) Minimum design flow for study.

b) For a two-story, four-unit, wood frame building.

c) For a three-story, twelve-unit, wood frame building in close proximity to a similar building.

d) For typical mill found in the industrial area.

Figure III-2 is a typical distribution grid which illustrates the recommended sizes for local water distribution pipelines, and it provides an example to follow in laying out water piping grids in street layouts. Pipeline extensions should follow this typical distribution to ensure adequate capacity. Where new developments use short dead-end streets, the waterline serving the dead-end street should be extended to an existing pipeline, forming a loop. Specific pipeline sizes proposed for a particular street are sized to provide sufficient capacity to satisfy the users served. The pipeline size and route may be changed if the demand can be satisfied without sacrificing adequate system pressures.

When considering combinations of pipelines to serve users, the following equivalent pipe relationships generally will provide approximately equal carrying capacities:

| | |
|---|---|
| One 16-inch pipe | = two 12-inch pipes |
| | = four 10-inch pipes |
| | = six 8-inch pipes |
| One 12-inch pipe | = two 10-inch pipes |
| | = three 8-inch pipes |
| | = six 6-inch pipes |
| One 10-inch pipe | = two 8-inch pipes |
| | = four 6-inch pipes |
| One 8-inch pipe | = two 6-inch pipes |
| | = six 4-inch pipes |
| One 6-inch pipe | = three 4-inch pipes |

These ratios are based on approximately equal head loss characteristics. Small pipelines constructed in parallel can be effective for supplying a particular location with a specific amount of water. However, this may not be the most economical use of funds. The installed cost of the several small pipelines necessary to equal the carrying capacity of the single large pipeline will usually exceed the cost of the larger pipeline. If the small pipelines are integrated into the distribution system of the service level rather than used merely to convey water to a specific area, the higher construction cost may be justified. For system reliability, it is preferable to have parallel smaller pipelines, assuming that they are not too small, rather than a single, large pipeline.

Valves should generally be spaced no further than 600 feet apart in a looped distribution system. Each pipeline intersection should have two or three valves, corresponding to a tee or cross connection, to minimize the areas out of service

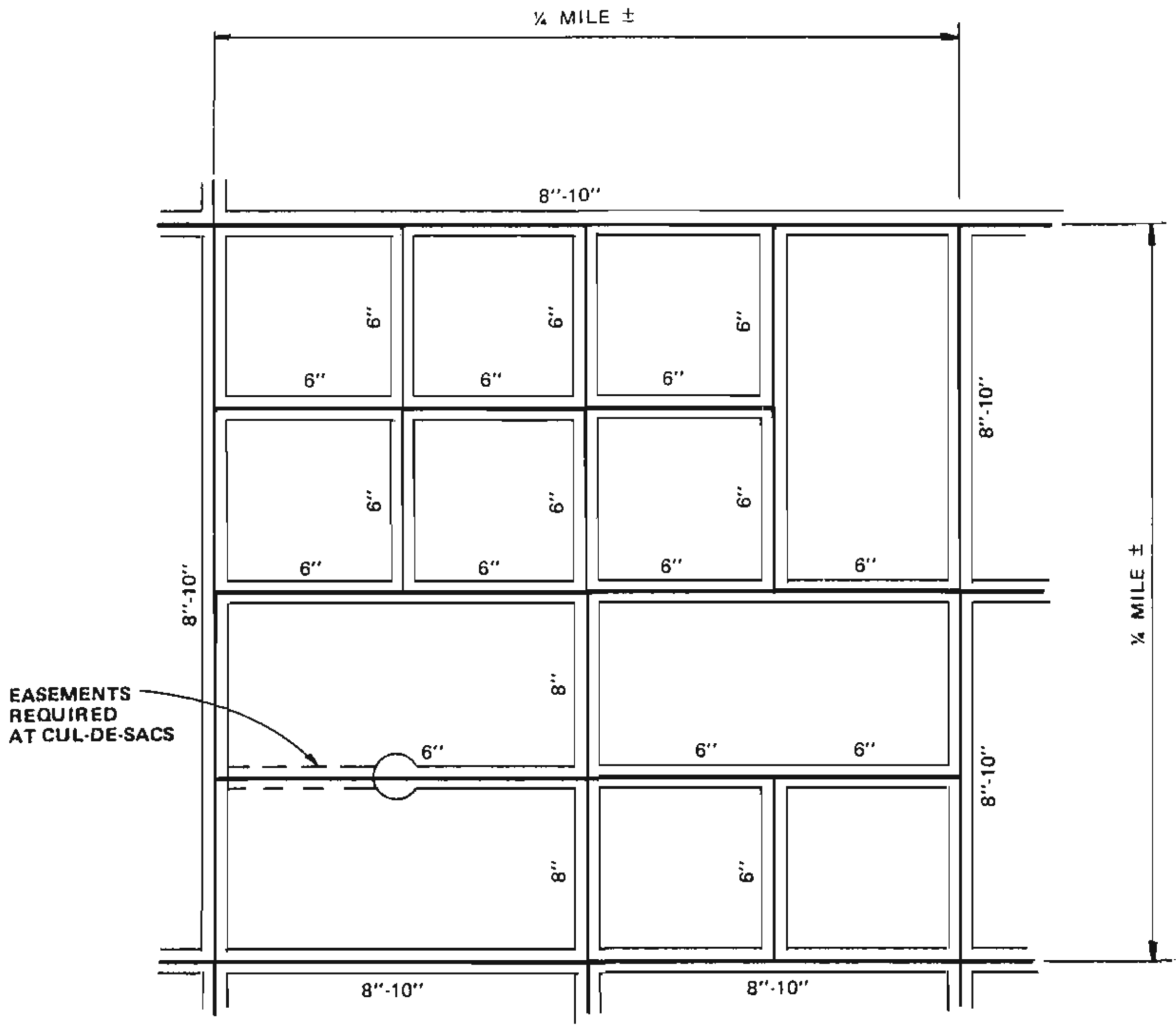


FIGURE III - 2

TYPICAL DISTRIBUTION GRID

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during a pipeline failure, repairs, or extensions. Fire hydrants should be spaced to meet the criteria recommended by the Insurance Services Office. Their letter of 4 October 1977 recommended that the high value business districts have a hydrant for each 104,000 square feet and for residential districts one hydrant for each 160,000 square feet. This indicates that fire hydrants should be spaced at about 300-350 feet apart in the high value business districts and 450-500 feet apart in the residential districts.

The Insurance Service Office of Oregon also recommends that distribution mains within residential districts be 8-inch or larger where dead ends and poor gridiron spacing exist, and a minimum of 6-inch when good gridiron spacing is available.

In commercial districts, the minimum main size should be 8-inch where good gridiron spacing is available and 12-inch or larger on principal streets or for long lines not connected to other mains at intervals close enough for mutual support.

Recommended Distribution System Improvements

The proposed water distribution system is shown on Map 1. The pipelines were located and sized using the model under various simulated conditions. A detailed description of these conditions is in Appendix A.

The proposed new pipelines and construction cost estimates are listed in Chapter IV.

These distribution system improvements include:

- To correct the reservoir refill problem and to provide water transmission capability to the Lawnridge Pump Station, a new pipeline is proposed extending north along Mill St. from the "M" Street Water Treatment Plant to "J" Street, then west along "J" Street across the center of the City to 3rd Street, thence north on 3rd Street to "E" Street, and west on "E" Street to the three first-level reservoirs

- Extending the transmission line north along Mill Street and Ninth Street to Madrone Street and Evelyn Avenue to strengthen flow to the upper first level and provide water supply to the second-level pump stations

- Installing pipelines along Ninth Street, Hillcrest Street, and Hawthorne Avenue to strengthen the northern second-level grid system

- Installing a new 12-inch pipeline along Savage and Hawthorne streets to improve flow within Zone 2A

- Extending the Highland Avenue pipeline to Reservoir Number 6

- Constructing a service line to Reservoir Number 5 along Beacon Drive.

Future Expansions

The development of the urbanizing areas adjacent to the present city limit service area will require extension and expansion of the water system. The plan to provide this service was developed using the design criteria discussed in preceding sections. The recommended pipeline improvements are shown on Map 1. The sequencing of water line construction is dependent upon rate of growth within the specific areas and demand for water service. Preliminary cost estimates for the recommended improvements within specific areas are listed in Chapter IV.

SERVICE LEVELS

To maintain adequate pressures at higher elevations and to prevent excessive pressures at lower elevations, distribution and storage systems must be separated into service levels. A system's maximum static pressures should not exceed 100 psi. Pressures above 100 psi tend to cause consumer plumbing problems, especially when unavoidable system surges occur. Such surges may be caused by fire hydrants being closed too quickly. The desired maximum pressure will set the lower elevation boundaries of a service level as a specific elevation difference between the reservoir's maximum water surface and the lowest customer's service connection. Since 100 psi is the pressure exerted at the base of the column of water 231 feet high, the elevation of the lowest service connection should not be more than 231 feet below the overflow elevation of the reservoir.

Pressures less than 25 psi at the service connection are not adequate for many water-using appliances or equipment. To maintain 25 psi (equivalent to a column of water 58 feet high), the upper elevation boundary of a service area should be located at least 100 feet below the reservoir's overflow elevation. Assuming the 42-foot difference between the 100-foot miminum recommended elevation is necessary to satisfy the system's hydraulic head losses, a maximum static pressure of 43 psi is required at the upper service level boundary.

During high demand periods the water level in a reservoir may fluctuate 10 to 15 feet. This leaves 27 to 32 feet of the 42-foot head available to force water through the pipelines to consumers near the upper elevation boundary. For example, a 10-inch diameter pipe two miles long would require about 30 feet of head differential to convey water at 600 to 700 gpm. The exact available capacity depends on the age and condition of the pipe. The same pipe would convey water at 425 to 475 gpm if the head differential were only 15 feet. Larger pipelines must be used in the distribution system if the services are located less than 100 feet in elevation below the reservoir.

Recommended Service Levels

Based upon these considerations, five service levels are recommended as shown on Figure II-1 and Map 1. These levels will serve the following elevation limits:

| Zone | Elevation Range | Reservoir Overflow Elevation |
|---|---|---|
| Zone 1 | 900-1020 | 1108.5 |
| Zone 2 | 1020-1140 | 1240.0 |
| Zone 2A | 960-1035 | 1240.0 |
| Zone 3 | 1140-1280 | 1370.0 |
| Zone 4 | 1280-1420 | 1520.0 |
| Zone 5 | 1420-1560 | 1660.0 |

Figure II-1 shows the boundaries of the proposed service areas together with the general mapping and topography information of the study area. Figure III-3 is a schematic hydraulic profile of the proposed water system showing the relationship of the existing and proposed major facilities within the five zones.

The water pressure at some services in the first and second levels under static conditions with a full reservoir, will exceed the recommended 100-psi maximum. The excessive pressure, while not desirable, affects only a small number of properties. This pressure does not appear sufficiently high to cause serious problems. If necessary, at the most economical cost, individual pressure-reducing valves may be installed at these services.

As urban development increases within the third and fourth levels, the conversion of the existing hydropnuematic systems to gravity reservoir systems is desirable. Inter-connecting these systems and placing storage facilities in the higher levels will increase the water distribution system's reliability.

Development near the 6th-Street interchange with Interstate 5 is creating a northern third-level service area. This area is zoned primarily for commercial and industrial uses with residential areas north of Interstate 5 and west along Highland Avenue and Woodbrook Drive. Map 1 shows the proposed

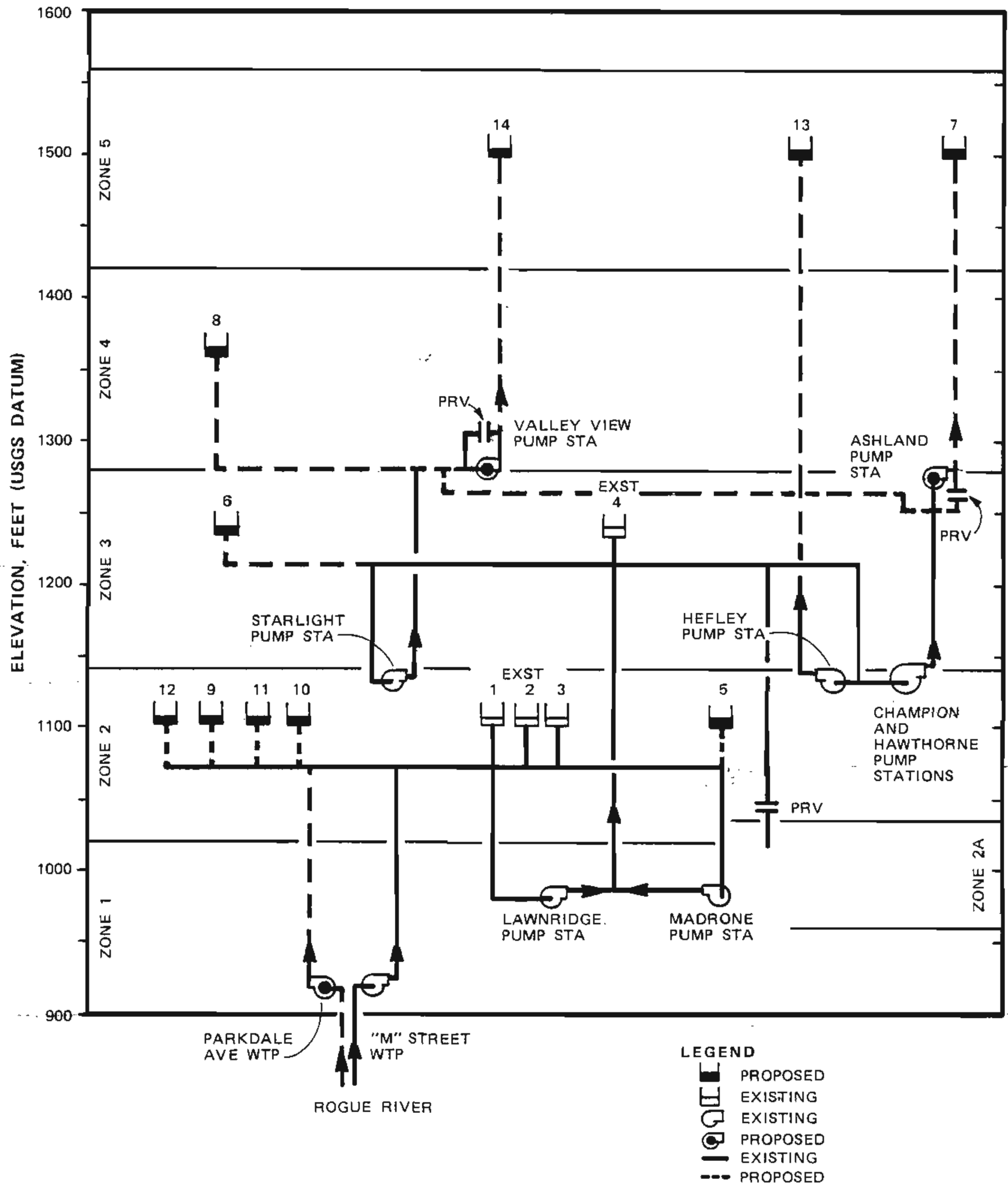


FIGURE III - 3

HYDRAULIC PROFILE
EXISTING AND PROPOSED WATER SYSTEM

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limits of this Zone 3. Presently, areas along Morgan Lane, Vine Street, and Hawthorne Avenue are served by Zone 2. It is recommended that Reservoir Number 8 be constructed to the northwest at elevation 1370 and that valving be provided and piping modifications made along Morgan Lane to convert this area into Zone 3. Because of economic considerations, these piping modifications may not be justified until development progresses and Reservoir Number 8 is constructed.

Zone 3 area along Valley View Road, Pleasant View Road, and Starlight Place should be inter-connected with north central Zone 3. In addition to the 12-inch transmission pipeline shown on Map 1, 8-inch pipelines should be used to strengthen this intertie. This will provide redundance of supply and fire protection, and it will eliminate the hydropneumatic system at Starlight Place. The pump station at Starlight Place will need to be expanded to provide for this increased water demand. The valving and piping modifications along Morgan Lane will also provide third-level service to the area north of Interstate 5.

Three fourth-level areas exist within the "draft urban growth boundary" service area as shown on Map 1. Terrace Subdivision located east of Beacon Drive is presently served by Hefley Pump Station. The developer would like to continue construction in this subdivision and has proposed the addition of Reservoir Number 13 and enlargement of the Hefley Pump Station. Further discussion and recommendations concerning Reservoir Number 13 and the Hefley Pump Station are presented later in this chapter. With continued development, the Terrace Subdivision may eventually become connected with the northern third- and fourth-level areas where Reservoir Number 7 is proposed. The reservoirs serving these areas should have their overflow set at elevation 1520.

A fourth-level area located west of Valley View Road is currently under design for residential development. A new pump station and reservoir (Number 14) will be required to serve this area. Since the development of the area within the "draft urban growth boundary" will access additional fourth-level land, these new facilities should consider further expansion.

There are several fifth-level parcels within the "draft urban growth boundary" area. These parcels are isolated such that an interconnected system is not feasible. It is proposed that water service to these areas be considered only as the need develops, and a plan for expansion into the Zone 5 areas is not shown.

South of the Rogue River, the majority of the land in the "draft urban growth boundary" area is within the first level.

The southern edge of the area is in the second level which continues south between Allen and Fruitdale Creeks. A portion of this second-level area is already served by the Fruitdale Heights and Pine Creek Water Utilities. The extension of the Grants Pass water system to these second level areas will depend upon growth and extensions within the first-level area. To serve the area, three pump station sites are shown on Map 1. Transmission pipelines are not shown in this area as development is not expected to occur within the study period.

## PUMP STATIONS

The capacity of pump stations which serve levels with distribution storage reservoirs must be adequate to provide the maximum day demand to all areas served by the pump station, while smaller pump stations which serve areas without a distribution storage reservoir must have sufficient capacity to furnish the maximum hour demand. Pump stations in a given service level must pump sufficient water to serve all levels above their particular level, in addition to serving their own level. Except for small service areas, the pump stations should have at least two pumps, and the total capacity of the pumps should be generally 20 to 50 percent in excess of the calculated capacity of the pump station. This allows a pump to be out of service for maintenance or repairs without causing a deficiency in supply. To have emergency reliability, one pump should be equipped with an auxiliary driver such as a diesel, gasoline, or natural gas engine to supplement the standard electric motor. This will provide some supply capability to the high service levels in the event of an electrical power outage. The auxiliary driver is not necessary if adequate reserve storage is available.

When possible, pump stations should be located at storage reservoirs serving the next lower service level to keep land costs at a minimum and to maintain stable suction pressures. Pump stations and reservoirs serving the same level should not be located in close proximity since larger pipelines will then be required in the remainder of the service level.

Each pump station should be controlled by the water level in the reservoir to which it pumps, with the pumped water metered at the pump station. It is desirable to transmit the reservoir water level and the metered flow data to a central location for supervisory control.

### Modifications to Existing Pump Stations

The existing pump station locations are shown on Map 1. Table III-3 lists the existing and proposed pump stations which are briefly discussed below.

## TABLE III-3

| PUMP STATION REQUIREMENTS | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | DEMAND YEAR 2000 (GPM) | | | | DEMAND FULL SITE DEVELOPMENT (GPM) | | | REQUIRED PUMP CAPACITY | | TOTAL PUMPING CAPACITY (GPM) |
| | LOCATION | NODE NO. | MAX. DAY | FIRE | MAX. DAY + FIRE | MAX. DAY | FIRE | MAX. DAY + FIRE | EXISTING | FUTURE | |
| ZONE 2 & 2A | LAWNRIDGE | 660 | 3,190 | -0- | 3,190 | 4,154 | -0- | 4,154 | 400<br>1,000<br>1,000 | | |
| | MADRONE | 344 | | | | | | | 300<br>600 | 900 | |
| | | | | | | | | | 3,300 | 900 | 4,200 |
| ZONE 3 | STARLIGHT | 421 | | | | | | | 200 | 600 | |
| | HAWTHORNE | 920 | 1,350 | 2,300 | 3,650 | 1,740 | 2,300 | 3,040 | 1,500 | 500 | |
| | CHAMPION | 814 | | | | | | | 1,500 | 800 | |
| | | | | | | | | | 3,200 | 1,900 | 5,100 |
| ZONE 4 | HEFLEY | 483 | 140 | -0- | 140 | 175 | -0- | 175 | 300 | 175 | |
| | AUSLAND | 713 | 382 | -0- | 382 | 531 | -0- | 531 | | 200<br>350<br>350 | |
| | VALLEY VIEW | 844 | 68 | -0- | 68 | 177 | -0- | 177 | | 175<br>175 | |
| | | | | | | | | | 300 | 1,425 | 1,725 |

Lawnridge. No capacity modifications are recommended for the Lawnridge Pump Station. Control modifications should include pump control from Reservoir Number 6 water level with existing manual override.

Madrone. A third pump rated at 900 gpm capacity should be added, and the existing controls maintained.

Starlight. As development within the third level and Valley View fourth level progresses, a new pump station is recommended. This pump station would initially provide two pumps rated at 200 gpm and 600 gpm with provisions in piping and floor space for expansion of another 600 gpm pump. A third pump may not be required until after year 2000. This pump station may be manually controlled, remotely from the water treatment plant or automatically from the water level in Reservoir Number 8.

Hawthorne. In conjunction with the construction of Reservoir Number 8 and the shift of the service level in the northern sector from Zone 2 to Zone 3, the Hawthorne Pump Station should be designed to provide service to this Zone 3 area. It is recommended that two new pumps be installed at rated capacities of 500 gpm and 1200 gpm. The pumps may be controlled manually at the pump station, remotely from the water treatment plant, or automatically by the water level in Reservoir Number 8.

Champion. This pump station should be renovated with the Hawthorne Pump Station and at the same time as the construction of Reservoir 8. A new pump is recommended at a capacity of 800 gpm. The existing 1500 gpm fire pump should be equipped with an auxiliary engine, either gasoline, diesel, or natural gas. The installation of auxiliary drivers would reduce the required capacity of Reservoir 8 by utilizing the fire storage from second-level storage.

Hefley. In conjunction with the construction of Reservoir 13, renovation of the Hefley Pump Station is recommended. The existing 300 gpm pump is to be used with a new 175 gpm pump. Fire flow will be provided from reservoir storage in this service area.

Woodson. This pump station will be phased out of service by connecting Woodson Park and Omar Drive areas with a new 6-inch pipeline. This pump station will operate to maintain system pressure during periods when otherwise low pressures would occur. The 150 gpm pump will boost the low pressures on Sun View Place and Crescent Drive. This pump station will also supply Zone 2. Pressure sensing at Sun View Place will control the pump operation.

### Proposed Pump Stations

Two new pump stations are proposed as part of the water system expansion. These pump stations will supply the isolated Zone 4 areas, as discussed below:

Ausland. The Ausland Pump Station will serve Zone 4 north of Interstate 5 and Reservoir 7. This pump station, consisting of three pumps rated 200 gpm, 350 gpm, and 350 gpm will be controlled by Reservoir Number 7 water level.

Valley View. The Valley View Pump Station will serve Zone 4 west of Valley View Drive and Reservoir 14. This pump station, consisting of two 175 gpm pumps with additional capacity for a future third pump will be controlled by the Reservoir Number 14 water level.

## STORAGE RESERVOIRS

### General

Distribution storage consists of three elements, as outlined below, which are considered in sizing storage reservoirs.

Equalizing Storage. This provides for hourly variations in demand. The peak hour demand for Grants Pass is normally about 150 percent of the maximum day demand. The typical hourly variation during a summer day is shown on Figure II-3. For this typical condition, equalizing storage equals approximately 25 percent of the maximum day demand for that portion of the service level served by the reservoir.

Fire Storage. Fire storage is based upon the Insurance Services Office criteria for grading municipal water systems for fire protection. This criteria evaluates the ability of the system to maintain the maximum daily consumption rate plus the basic fire flow at the required duration. To have no deficiency in the grading, the remaining system capacity in conjunction with storage must be able to provide the basic fire flow for the specified duration at any time during an emergency period with consumption at the maximum daily rate. The actual duration of this maximum flow is specified relative to parts of the system that might be out of service as a result of maintenance and repair work or an emergency. Fire storage to meet fire flow demands should be equal to the volume of water required to provide the maximum recommended fire flows in the service area for the required duration as recommended by the Insurance Services Office, and shown in Table III-1.

Reserve Storage. Reserve storage supplies the system's needs during the disruption of the supply capabilities. The reserve storage can vary widely among individual water systems.

Supply for the Grants Pass water system could be interrupted by a power failure, a malfunction at the water treatment plant, or a transmission pipeline break. Upon completion of the proposed new water treatment plant and related supply facilities, the reliability of the water supply will be increased. Reserve storage partially offsets the need for duplicating various parts of the supply works and the value of storage depends upon its amount, location, and availability. The reserve storage recommended for the Grants Pass water system is 50 percent of maximum day demand. This is equivalent to a 12-hour supply during a day of maximum use and about 1-1/3 days of average annual day demand.

The location for future reservoirs should be based on the following criteria: 1) the site should be as close to the service area as possible to minimize cost; 2) the site should be on reasonably flat ground to keep excavation costs down; 3) the reservoir and the pump station serving the reservoir should be located at opposite ends of the service level in order to minimize system pressure losses during peak demands; and 4) the site should allow the reservoir to blend into the environment with an aesthetically pleasing appearance.

At each reservoir, water-level sensing equipment should be used to control the operation of the pumps and transmit the reservoir level to a central supervisory control center for observation and recording.

Proposed Storage Reservoirs

Table III-4 summarizes the proposed reservoirs and their sizing criteria and Map 1 shows the reservoir locations. Service areas have been assigned to each of these reservoirs. Reservoirs 9, 10, 11, and 12 are required as system extension occurs south of the Rogue River. Reservoirs 13 and 14 are needed as growth occurs in their respective service levels. Reservoir 9 will provide storage to the areas served by Pine Creek Water Utility and Fruitdale Heights Water Company. To serve the western Redwood area reservoirs 11, and 12 will be required. Figure III-4 shows the projected reservoir requirements based on the uniform rate of system expansion discussed in Chapter II.

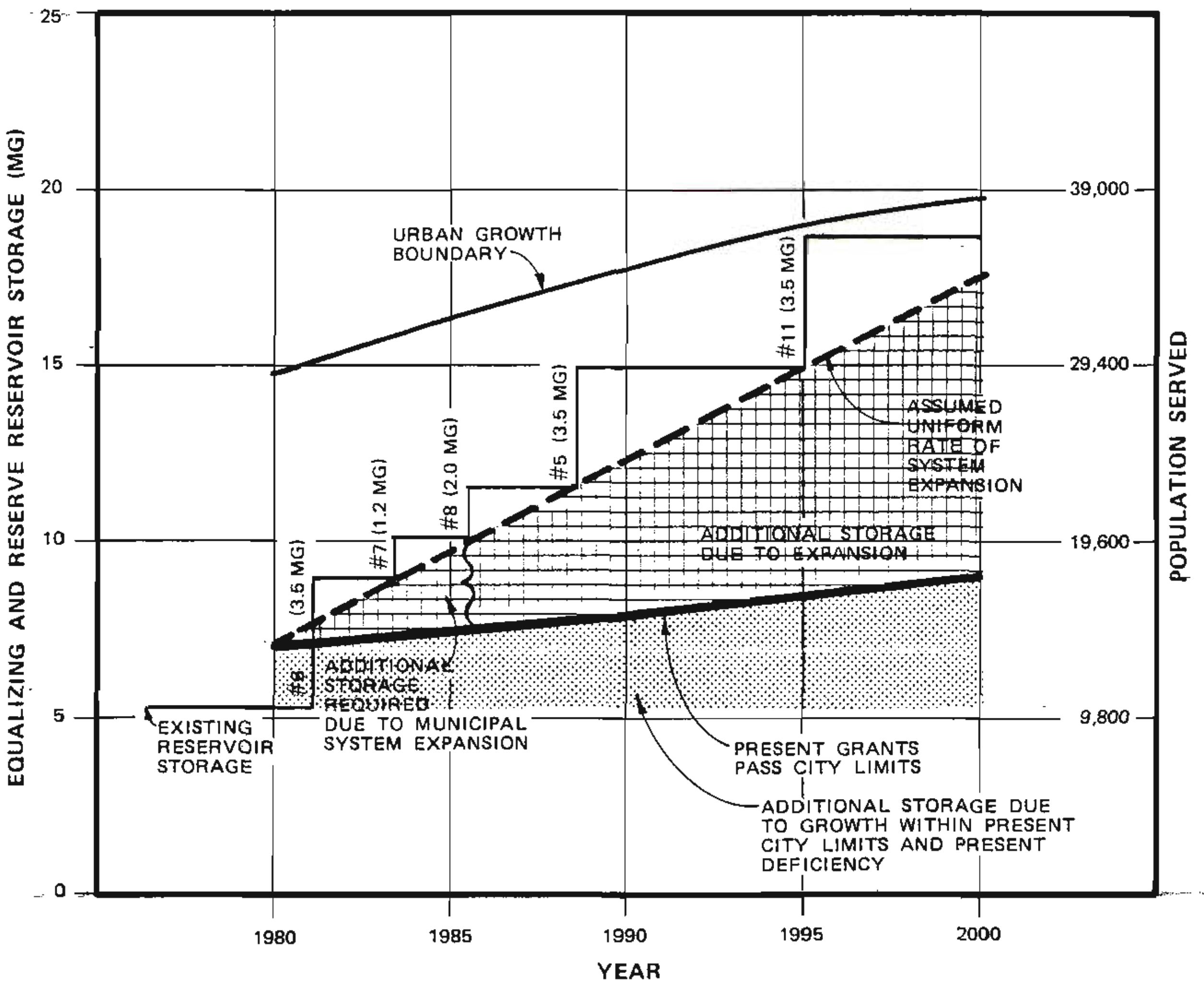
Proposed Reservoir 6 is to be located on the west side of Zone 2. This reservoir will improve low pressure conditions along Highland Avenue, Morgan Lane, and Valley View Road by raising the hydraulic grade line. This reservoir will also provide a nearby source of supply for the Hawthorne, Starlight and Champion pump stations. This reservoir will also contain some Zone 3 fire protection storage.

## TABLE III-4

**PROPOSED RESERVOIRS**

| NO. | LOCATION | NODE NO. | ZONE/ WATER SURFACE ELEVATION (USGS) | EQUALIZING STORAGE (GAL.) | RESERVE STORAGE (GAL.) | FIRE PROTECTION STORAGE (GAL.) | TOTAL STORAGE (GAL.) | RECOMMENDED STORAGE CAPACITY (GAL.) | CONSTRUCTION PHASE |
|---|---|---|---|---|---|---|---|---|---|
| 5 | GRANDVIEW DR. | 716 | 1/1108.5 | 1,167,000 | 2,333,000 | -0- | 3,500,000 | 3,500,000 | II |
| 6 | N. HIGHLAND | 810 | 2/1240.0 | 1,139,000 | 2,313,000 | -0- | 3,452,000 | 3,500,000 | I |
| 7 | AUSLAND DR. | 715 | 4/1510.0 | 360,000 | 720,000 | 120,000 | 1,200,000 | 1,200,000 | II |
| 8 | NORTHWEST AREA | 770 | 3/1390.0 | 410,000 | 820,000 | 750,000 | 1,980,000 | 2,000,000 | II |
| 9 | FRUITDALE | 733 | 1/1108.5 | 335,000 | 671,000 | 500,000 | 1,506,000 | 1,500,000 | A |
| 10 | HIGHWAY 99 EAST | 790 | 1/1108.5 | 547,000 | 1,093,000 | 270,000 | 1,910,000 | 2,000,000 | A |
| 11 | REDWOOD AREA | 775 | 1/1108.5 | 1,216,000 | 2,433,000 | 750,000 | 4,399,000 | 4,500,000 | A |
| 12 | REDWOOD AREA | 787 | 1/1108.5 | 586,000 | 1,171,000 | 240,000 | 1,997,000 | 2,000,000 | A |
| 13 | TERRACE SUBDIVISION | 857 | 4/1510.0 | 46,000 | 93,000 | 120,000 | 259,000 | 250,000 | A |
| 14 | VALLEY VIEW WEST | 845 | 4/1510.0 | 64,000 | 127,000 | 120,000 | 311,000 | 350,000[A] | A |
| | | | | 5,870,000 | 11,774,000 | 2,870,000 | 20,514,000 | 20,800,000 | |

A TO BE CONSTRUCTED AS GROWTH IN SERVICE LEVEL REQUIRES.




* ACTUAL POPULATION SERVED WILL DETERMINE THE RATE OF EXPANSION AND RELATED RESERVOIR REQUIREMENTS.

FIGURE III - 4

DISTRIBUTION STORAGE REQUIREMENTS

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Reservoir 8 is centrally located within Zone 3 and will provide reserve, equalizing, and fire flow storage for this zone. The construction of this reservoir will improve pressure and reliability of supply within this zone.

Reservoir 7 serves the fourth level in the northern portion of the city. This area is developing rapidly and will soon need this reservoir for fire storage. Upon extreme pressure drop in Zone 3, a pressure-reducing valve will also serve Zone 3.

Reservoir 5 is located on the east side of the first level. This reservoir will provide flows for fire protection to the high school and industrially zoned land to the east of the city. This reservoir will also provide equalizing and reserve storage.

Reservoir 11 is located in the Redwood area west of Allen Creek Road. This reservoir will provide fire protection to the junior high school along Harbeck Road and to commercial and industrial zoned land along Highway 199, and improve the pressures in the Redwood area.

Reservoirs 9 and 10 will serve Zone 1 south of the Rogue River. The need for this reservoir will depend upon system expansion in the area.

Reservoir 12, located south of the Redwood Highway and west of Hubbard Lane, will serve the western Redwood Area. This reservoir will provide equalizing, reserve, and fire flow storage and pressure stabilization.

Reservoirs 13 and 14 will serve isolated Zone 4 areas. The 60,000 gallons of storage proposed by the Terrace Subdivision developer is inadequate because 250,000 gallons is needed to provide 2 hours fire flow at 1,000 gpm, equalizing and reserve storage. Both Reservoirs 13 and 14 will be needed as growth in their respective service zone progresses.

A plan for the construction sequencing of reservoirs is shown in Table IV-1, and discussed further in Chapter IV. This plan is flexible and dependent upon water demand. It is feasible to construct Reservoir 7 prior to Reservoir 6 although it may not be the most economical long-term solution. The initial capital expenditure will be less by delaying the construction of Reservoir 6. The pressure-reducing valves enable Reservoir 7 to provide fire storage to the north central service area. The disadvantages of this option is that Reservoir 7 is sized to supply equalizing, reserve, and fire storage for the north central Zone 4 service area. Frequent low pressures in Zone 2 may require use of this fourth-level storage capacity, thereby affecting service to Zone 4 customers.

Reservoir 6 is needed to provide a base for growth to upper service levels in this area, and its construction will correct low pressure conditions and provide a source of supply for Hawthorne and Champion pump stations.

# IV

# CONSTRUCTION SEQUENCE AND COST ESTIMATES

# Chapter IV
# CONSTRUCTION SEQUENCE AND COST ESTIMATES

## GENERAL

This study has investigated the existing water distribution system and proposes system improvements which will enable the City to provide adequate and reliable water service within the "draft urban growth boundary" area through the year 2000. The proposed improvements are shown in Table IV-1; Water System Improvements - Construction Sequence and Estimated Costs.

## CONSTRUCTION SEQUENCE

Three phases for the construction of system improvements have been developed for the through year 2000. Phase I includes those improvements that are urgently needed to meet existing system requirements and should be constructed within the next two to three years. Phase II improvements are less important but are required to upgrade the system to meet fire flow requirements and growth within the immediate service area. Urban service extension improvements consist of expanding the distribution system to provide service into the urbanizing area outside the present city limits. The construction of these facilities will depend upon the growth within the "draft urban growth boundary" area. The improvements within these phases are presented in their order of priority in Table IV-1.

## COST ESTIMATES

The cost estimates, shown in Table IV-1, and are based upon current material and construction costs within the State of Oregon. The *Engineering News Record* construction cost indexes may be used as the basis for escalating these costs at the time of construction. The estimated cost assume that the work will be undertaken by a private contracting company. The project cost of each of the improvements includes an allowance for contingencies, administrative and engineering costs.

Recent price trends indicate that changes in construction costs are still unpredictable and the cost estimates are preliminary ones based upon the availability of materials from normal supply sources, and were prepared without the detail drawings. The inflationary trends and the uncertainty in the nation's economic condition makes the *Engineering News Record* construction cost index a less reliable indicator of future costs than it has been in the past. These cost estimates must be reviewed prior to undertaking any part of the work to ensure that financing is adequate at the prices prevailing at the time.

Reservoir costs, except for reservoirs 13 and 14, are based on current construction costs for ground-level prestressed concrete reservoirs. Prices include the cost of site work, piping, controls, exterior treatment, and miscellaneous appurtenances. The cost for purchase of land and construction of an access road to the site are not included. A 20 percent allowance for engineering and contingencies is included.

Reservoirs 13 and 14 were based on current construction costs for steel reservoirs. Reservoir protective coatings are assumed to be a high-quality vinyl paint with a useful life of 15 years or more. Prices include the cost of site work, foundations, piping, controls, and miscellaneous appurtenances. An allowance for contingencies, administrative and engineering costs is included.

The pump station cost includes the building, piping, pumps as indicated, control valves and telemetering. The costs for the purchase of land, and extension of an access road and electrical service to the site are not included. A 25 percent allowance for engineering and contingencies is included.

Installed pipe costs were based on the assumption that all water lines would be constructed of ductile iron pipe. The estimated unit costs for assumed trench conditions, surface restoration, and fitting requirements used for pipeline construction are shown in Table IV-2.

Phase I improvements include the construction of Reservoir 6 which will strengthen the northern Zone 2 service area and bring the total storage to 8.95 million gallons. Additional Phase I improvements include the installation of a new pump at Madrone Pump Station, pump controls at Woodson Pump Station, and transmission and distribution pipelines. The new pipelines will: (1) improve existing first-level reservoir refill conditions and provide improved distribution capacity to the central business district and Lawnridge Pump Station; (2) connect Reservoir 6 to the second-level system; (3) strengthen the second-level distribution system. These improvements are urgently needed to upgrade the existing system.

Phase II improvements include constructing several reservoirs, modifying pump stations, and constructing various pipelines. Reservoirs 7 and 8 should be constructed first to serve the rapidly developing northern area of the city. In conjunction with the construction of these reservoirs, modifications to the Hawthorne and Champion Pump Stations and the construction of the Ausland Pump Station should be completed. The construction of pipelines will accommodate service to these reservoirs and pump station facilities. The remaining Phase II improvements consist of constructing Reservoir 5, modifying Starlight Pump Station, and constructing additional pipelines. The estimated costs of this remaining portion of Phase II improvements are $2,541,000.

The urban service extension facilities consist of expanding the distribution system to serve the urbanizing area outside the present city limits.

TABLE IV-1
WATER SYSTEM IMPROVEMENTS
CONSTRUCTION SEQUENCE AND ESTIMATED COSTS

PHASE I

| Description | Facility Capacity and Pipeline Size and Length | | | Estimated Cost |
|---|---|---|---|---|
| Reservoir 6 | 3.5 MG | | | $ 763,000 |
| Madrone Pump Station | New 900 GPM Pump | | | 4,000 |
| Woodson Pump Station | Control modification | | | 10,000 |
| | | | Subtotal | $ 777,000 |
| Pipelines to first-level reservoir. | | | | |
| Pipeline Number* | | | | |
| 808 | 20 | 1480 | | 102,000 |
| 142 | 24 | 480 | | 41,000 |
| 141 | 24 | 630 | | 54,000 |
| 138 | 24 | 710 | | 61,000 |
| 208 | 24 | 700 | | 60,000 |
| 804 | 24 | 380 | | 33,000 |
| 200 | 24 | 380 | | 33,000 |
| 664 | 20 | 560 | | 39,000 |
| 661 | 20 | 360 | | 25,000 |
| 760 | 20 | 300 | | 21,000 |
| 662 | 20 | 280 | | 19,000 |
| 663 | 20 | 280 | | 19,000 |
| 664 | 20 | 560 | | 25,000 |
| 665 | 20 | 280 | | 19,000 |
| 569 | 20 | 630 | | 44,000 |
| 154 | 20 | 510 | | 35,000 |
| 666 | 20 | 1160 | | 80,000 |
| | | | Subtotal | $710,000 |
| Pipeline to Reservoir 6. | | | | |
| Pipeline Number* | | | | |
| 651 | 16 | 1400 | | 79,000 |
| 650 | 16 | 1500 | | 85,000 |
| | | | Subtotal | $164,000 |
| Pipeline to strengthen Zone 2. | | | | |
| Pipeline Number* | | | | |
| 767 | 10 | 1920 | | 71,000 |
| 762 | 12 | 600 | | 26,000 |
| 763 | 12 | 400 | | 17,000 |
| 764 | 12 | 600 | | 26,000 |
| 765 | 12 | 700 | | 30,000 |
| 766 | 12 | 440 | | 19,000 |
| | | | Subtotal | $189,000 |
| | | | Total | $1,840,000 |

*Computer model pipe number.

PHASE II

| Description | Facility Capacity | | | Estimated Cost |
|---|---|---|---|---|
| Reservoir 7 | 1.2 MG | | | $432,000 |
| Reservoir 8 | 2.0 MG | | | 516,000 |
| Reservoir 5 | 3.5 MG | | | 763,000 |
| Hawthorne Pump Station | 1700 GPM | | | 90,000 |
| Champion Pump Station | 2300 GPM | | | 150,000 |
| Ausland Pump Station | 900 GPM | | | 48,000 |
| Starlight Pump Station | 800 GPM | | | 42,000 |
| | | | Subtotal | $2,041,000 |
| Pipeline to Reservoir 7 | | | | |
| Pipeline Number* | | | | |
| 768 | 12 | 225 | | 10,000 |
| 652 | 12 | 1110 | | 47,000 |
| 774 | 12 | 450 | | 19,000 |
| 653 | 12 | 760 | | 32,000 |
| 654 | 12 | 1240 | | 53,000 |
| 655 | 12 | 1070 | | 46,000 |
| | | | Subtotal | $207,000 |
| Pipeline to Reservoir 8 | | | | |
| Pipeline Number* | | | | |
| 785 | 12 | 580 | | 25,000 |
| 769 | 12 | 50 | | 2,000 |
| 781 | 12 | 1960 | | 83,000 |
| 782 | 12 | 840 | | 36,000 |
| 783 | 12 | 360 | | 15,000 |
| 788 | 16 | 1100 | | 62,000 |
| 770 | 16 | 2080 | | 118,000 |
| | | | Subtotal | $341,000 |

* Computer model number.

PHASE II (continued)

Pipeline to Madrone and Lawnridge Pump Stations and Reservoir 5.

Pipeline Number*

| | | | | |
|---|---|---|---|---|
| 143 | 24 | 1130 | | 97,000 |
| 257 | 24 | 620 | | 53,000 |
| 161 | 24 | 320 | | 27,000 |
| 162 | 24 | 290 | | 25,000 |
| 163 | 24 | 150 | | 13,000 |
| 403 | 24 | 500 | | 43,000 |
| 731 | 24 | 260 | | 22,000 |
| 732 | 24 | 400 | | 34,000 |
| 174 | 20 | 280 | | 19,000 |
| 175 | 20 | 180 | | 12,000 |
| 176 | 20 | 610 | | 42,000 |
| 733 | 20 | 140 | | 10,000 |
| 386 | 16 | 1000 | | 56,000 |
| 382 | 16 | 350 | | 20,000 |
| 758 | 16 | 1400 | | 79,000 |
| 757 | 16 | 500 | | 28,000 |
| 602 | 16 | 320 | | 18,000 |
| 601 | 16 | 350 | | 20,000 |
| 721 | 16 | 320 | | 18,000 |
| | | | Subtotal | $ 636,000 |

Transmission pipeline (Alternative A)

Pipeline Number*

| | | | | |
|---|---|---|---|---|
| 715 | 36 | 1440 | | 180,000 |
| 714 | 36 | 540 | | 67,000 |
| 730 | 36 | 300 | | 37,000 |
| 659 | 30 | 900 | | 96,000 |
| 190 | 30 | 1300 | | 139,000 |
| 103 | 30 | 760 | | 81,000 |
| 674 | 30 | 640 | | 69,000 |
| 809 | 30 | 280 | | 30,000 |
| 810 | 30 | 280 | | 30,000 |
| | | | Subtotal | $ 729,000 |

Pipeline to Reservoir 5 and Starlight Pump Station

| | | | | |
|---|---|---|---|---|
| 656 | 20 | 1600 | | 110,000 |
| 529 | 12" | 2590 | | 110,000 |
| | | | Subtotal | $ 220,000 |

* Computer model pipe number

PHASE II

Pipeline within Zone 2A

Pipeline Number*

| | | | | |
|---|---|---|---|---|
| 455 | 12 | 660 | | 28,000 |
| 612 | 12 | 400 | | 17,000 |
| 422 | 12 | 400 | | 17,000 |
| 421 | 12 | 400 | | 17,000 |
| 420 | 12 | 400 | | 17,000 |
| 419 | 12 | 500 | | 21,000 |
| 418 | 12 | 800 | | 34,000 |
| | | | Subtotal | $ 151,000 |
| | | | Total | $4,325,000 |

* Computer model pipe number

Urban Services Extension

| Description | Facility Capacity/ Pipeline Size and Length | | Estimated Cost |
|---|---|---|---|
| Reservoir 9 | 1.5 MG | | $ 456,000 |
| Reservoir 10 | 2.0 MG | | 516,000 |
| Reservoir 11 | 4.5 MG | | 918,000 |
| Reservoir 12 | 2.0 MG | | 516,000 |
| Reservoir 13 | 0.25 MG | | 70,000 |
| Reservoir 14 | 0.35 MG | | 85,000 |
| Hefley Pump Station | 350 gpm | | 12,000 |
| Valley View Pump Station | 350 gpm | | 21,000 |
| Pipeline Number* | | | |
| 775 | 12 | 930 | 40,000 |
| 776 | 12 | 780 | 33,000 |
| 777 | 12 | 1100 | 47,000 |
| 778 | 12 | 1000 | 43,000 |
| 779 | 12 | 2380 | 101,000 |
| 780 | 12 | 1000 | 43,000 |
| 784 | 10 | 2860 | 106,000 |
| 771 | 10 | 300 | 11,000 |
| 786 | 10 | 1480 | 55,000 |
| 787 | 12 | 1130 | 48,000 |
| 789 | 12 | 2650 | 113,000 |
| 790 | 12 | 3150 | 134,000 |
| 797 | 12 | 820 | 35,000 |
| 799 | 12 | 650 | 28,000 |
| 798 | 8 | 1330 | 43,000 |
| 802 | 8 | 1400 | 45,000 |
| 801 | 8 | 1060 | 34,000 |
| 803 | 12 | 230 | 10,000 |
| 791 | 12 | 1000 | 43,000 |
| 792 | 10 | 540 | 20,000 |
| 793 | 10 | 1510 | 56,000 |
| 794 | 8 | 1220 | 39,000 |
| 795 | 10 | 1510 | 56,000 |
| 796 | 10 | 700 | 26,000 |
| 107 | 12 | 790 | 34,000 |
| 125 | 12 | 860 | 37,000 |
| 124 | 12 | 340 | 14,000 |
| 123 | 12 | 360 | 15,000 |
| 122 | 12 | 430 | 18,000 |

*Computer model pipe number

| | | | |
|---|---|---|---|
| 746 | 12 | 1240 | $ 53,000 |
| 747 | 12 | 1300 | 55,000 |
| 748 | 12 | 1340 | 57,000 |
| 749 | 12 | 1440 | 61,000 |
| 750 | 12 | 1300 | 55,000 |
| 751 | 12 | 1460 | 62,000 |
| 752 | 12 | 1260 | 54,000 |
| 753 | 8 | 390 | 13,000 |
| 812 | 12 | 700 | 30,000 |
| 667 | 12 | 1300 | 55,000 |
| 668 | 12 | 1300 | 55,000 |
| 669 | 12 | 1160 | 49,000 |
| 670 | 12 | 1360 | 58,000 |
| 671 | 12 | 1840 | 78,000 |
| 672 | 12 | 880 | 37,000 |
| 673 | 12 | 1000 | 43,000 |
| 722 | 8 | 1400 | 45,000 |
| 723 | 8 | 840 | 27,000 |
| 724 | 12 | 2000 | 85,000 |
| 725 | 12 | 1500 | 64,000 |
| 726 | 12 | 1300 | 55,000 |
| 727 | 12 | 1140 | 49,000 |
| 728 | 12 | 1600 | 68,000 |
| 729 | 12 | 800 | 34,000 |
| 675 | 24 | 900 | 77,000 |
| 676 | 20 | 960 | 66,000 |
| 677 | 20 | 1140 | 79,000 |
| 678 | 20 | 1000 | 69,000 |
| 679 | 20 | 500 | 35,000 |
| 680 | 12 | 980 | 42,000 |
| 681 | 12 | 700 | 30,000 |
| 682 | 12 | 1000 | 43,000 |
| 683 | 12 | 500 | 21,000 |
| 684 | 12 | 720 | 31,000 |
| 685 | 12 | 1800 | 77,000 |
| 686 | 12 | 2000 | 85,000 |
| 687 | 12 | 1860 | 79,000 |
| 713 | 24 | 820 | 70,000 |
| 19 | 24 | 490 | 42,000 |
| 711 | 20 | 600 | 41,000 |
| 712 | 20 | 400 | 28,000 |
| 706 | 20 | 1200 | 83,000 |
| 705 | 20 | 840 | 58,000 |
| 704 | 16 | 1220 | 69,000 |
| 703 | 16 | 1160 | 65,000 |
| 702 | 16 | 1200 | 68,000 |
| 701 | 12 | 1400 | 60,000 |
| 739 | 20 | 700 | 48,000 |

Urban Services Extension (continued)

| | | | |
|---|---|---|---|
| 754 | 16 | 1100 | $ 62,000 |
| 688 | 12 | 550 | 23,000 |
| 689 | 12 | 840 | 36,000 |
| 690 | 12 | 1050 | 45,000 |
| 691 | 12 | 800 | 34,000 |
| 692 | 12 | 200 | 9,000 |
| 693 | 12 | 1360 | 58,000 |
| 694 | 12 | 1860 | 79,000 |
| 695 | 16 | 1300 | 73,000 |
| 696 | 16 | 1360 | 77,000 |
| 697 | 12 | 600 | 26,000 |
| 698 | 12 | 2600 | 111,000 |
| 699 | 12 | 1580 | 67,000 |
| 700 | 12 | 1320 | 56,000 |
| 707 | 12 | 700 | 30,000 |
| 708 | 12 | 1360 | 58,000 |
| 709 | 12 | 1860 | 79,000 |
| 710 | 12 | 1840 | 78,000 |
| 759 | 12 | 2000 | 85,000 |
| 740 | 12 | 1200 | 51,000 |
| 741 | 12 | 1400 | 60,000 |
| 742 | 12 | 1120 | 48,000 |
| 743 | 12 | 1440 | 61,000 |
| 744 | 12 | 1740 | 74,000 |
| 745 | 12 | 1440 | 61,000 |
| 815 | 12 | 1200 | 51,000 |
| 734 | 16 | 1800 | 102,000 |
| 735 | 16 | 1700 | 96,000 |
| 736 | 12 | 1200 | 51,000 |
| 737 | 12 | 1560 | 66,000 |
| 738 | 12 | 1160 | 49,000 |
| 820 | 16 | 640 | 36,000 |
| 821 | 16 | 2600 | 147,000 |
| 822 | 16 | 2600 | 147,000 |
| 823 | 12 | 1260 | 54,000 |
| 824 | 12 | 1300 | 55,000 |
| 825 | 12 | 1300 | 55,000 |
| 826 | 12 | 1300 | 55,000 |
| 827 | 12 | 2600 | 111,000 |
| 828 | 12 | 1740 | 74,000 |
| 829 | 12 | 2600 | 111,000 |
| 830 | 12 | 2260 | 96,000 |
| 831 | 12 | 2600 | 111,000 |
| 832 | 12 | 2600 | 111,000 |
| 833 | 12 | 3000 | 128,000 |
| | | Total | $9,676,000 |

## TABLE IV-2

| ESTIMATED COST OF INSTALLED DUCTILE IRON PIPE ($/LINEAL FEET INDEXED TO ENR 3190) | | | | | | | |
|---|---|---|---|---|---|---|---|
| PIPE SIZE | PIPE IN-PLACE | TRENCH EXCAVATION AND BACKFILL | FITTINGS VALVES AND HYDRANTS | SURFACE RESTORATION | SUBTOTAL | 20% ENGINEERING AND CONTINGENCIES | TOTAL |
| 6" | $ 6.35 | $5.25 | $ 2.90 | $9.20 | $23.70 | $ 4.75 | $28.45 |
| 8" | 8.60 | 5.50 | 3.45 | 9.30 | 26.85 | 5.40 | 32.25 |
| 10" | 11.65 | 5.80 | 4.05 | 9.40 | 30.90 | 6.20 | 37.10 |
| 12" | 15.30 | 6.00 | 4.60 | 9.55 | 35.45 | 7.10 | 42.55 |
| 14" | 18.95 | 6.30 | 6.10 | 9.65 | 41.00 | 8.20 | 49.20 |
| 16" | 23.05 | 6.60 | 7.15 | 10.15 | 6.95 | 9.40 | 56.35 |
| 18" | 27.55 | 6.80 | 8.20 | 10.15 | 52.70 | 10.55 | 63.25 |
| 20" | 31.40 | 6.90 | 9.10 | 10.15 | 57.55 | 11.50 | 69.05 |
| 24" | 39.60 | 8.65 | 11.45 | 11.65 | 71.35 | 14.30 | 85.65 |
| 30" | 49.55 | 10.80 | 14.30 | 14.55 | 89.20 | 17.85 | 107.05 |
| 36" | 57.50 | 12.55 | 16.60 | 17.30 | 103.95 | 20.80 | 124.75 |

ASSUMPTIONS:

1. FITTINGS BASED ON ONE TEE EVERY 500 FEET.
2. VALVES AND HYDRANTS SPACED AT APPROXIMATELY 500 FEET.

# V

# SUMMARY AND RECOMMENDATIONS

## Chapter V
## Summary and Recommendations

### SUMMARY

This study of the water distribution system has determined the water requirements and developed a plan to serve the present and future population to year 2000 residing within both the service areas contained by the existing city limits and the "draft urban growth boundary". The "draft urban growth boundary", land uses and occupancy, and the projected populations to be served by the Grants Pass water system conforms with the proposed policies and guidelines of the "draft" comprehensive plan. The capability of the water system to meet these requirements was evaluated, and from this evaluation the proposed improvements have been identified and each assigned a construction phase based upon its importance to the system. Each improvement is discussed in detail in Chapters III and IV.

Phase I improvements represent the immediate upgrading needs of the existing system to correct current deficiencies, while Phase II improvements are necessary to strengthen the existing system to serve the area within present city limits.

The urban services extensions are expansions of the system to serve the new areas within the "draft urban growth boundary" area. The system elements, and preliminary cost estimates, for each construction phase are summarized in the following table:

TABLE V-1
Summary of Water Distribution System Improvements

Present City Limit Service Area

| | Phase I | | Phase II |
|---|---|---|---|
| | | High Priority | |
| Reservoirs | $ 763,000 | $948,000 | $ 763,000 |
| Pump Stations | 14,000 | 288,000 | 42,000 |
| Pipelines | 1,063,000 | 548,000 | 1,736,000 |
| Total | $1,840,000 | $1,784,000 | $2,541,000 |

Urban Service Extensions

| | |
|---|---|
| Reservoirs | $2,561,000 |
| Pump Stations | 33,000 |
| Pipelines | 7,082,000 |
| Total | $9,676,000 |

## Recommendations

On the basis of this study, we recommend:

1. Adopt the plan for improving the distribution system as outlined in this report.

2. Review the plan periodically and make the necessary adjustments to accommodate the actual conditions and revised planning goals.

3. Initiate a preliminary design study for the future supply facility, including the new water treatment plant or the rehabilitation of existing plant alternatives; develop and design parameters, preliminary cost estimates, and construction phasing of the water supply needed to meet the planning goals.

4. Prepare and initiate a program to fund and construct the Phase I and II improvements of the water distribution system plan and the required water supply expansion. The implementation program should anticipate the need for new supply and distribution facilities before year 1982.

5. Borrowed funds, governmental grants, or contributions will finance the system improvements; therefore, to investigate the funding options, together with the repayment plans and user-charge schedules, a financial study, which establishes fair and equitable rates for all user classes, should be completed as part of the implementation program.

# Appendix A

TITLE

PASS1-OT2

CONDITIONS

Current year max day & fire flow (1500 gpm @ NODE 702) = 8284 gpm = 11.93 mgd, WTP producing 7474 gpm = 10.76 mgd Reservoir No. 4 supplying 725 gpm = 1 mgd Reservoirs 1, 2, & 3 supplying 85 gpm = 0.122 mgd, Lawnridge pump station is operating w/3 pumps on.

EXPLANATION

This run demonstrates the system's supply capability during current year for maximum day flow conditions with a second-level fire demand. The majority of head loss is obtained by flow crossing the downtown business district and directed towards the suction side of the Lawnridge Pump Station. Adequate pressures were maintained throughout the system with a suction pressure of 46 psi supplied to the Hawthorne Pump Station. The Cresent Drive area would experience pressure drops to 25 psi and those services near the plant may experience pressures in excess of 100 psi.

A larger transmission line through the main business district would relieve the head loss generated downtown and the completion of the second-level grid system on Hawthorne & Hillcrest Streets would improve water transmission to the north.

TITLE

PASS1-OT3B

CONDITIONS

1500-gpm fire flow at Hawthorne Pump Station. Nightime flow demand imposed on the system, water treatment plant shut down for the evening, Lawnridge Pump Station running with 3 pumps on. (2400 gpm) Reservoirs 1, 2, & 3 providing 2800 gpm to the system; Reservoir 4 maintaining level.

EXPLANATION

This run demonstrated the system's water supply ability utilizing first-level reservoirs and booster pumps to provide second-level fire demands, with the water treatment plant down. The pressures were very good with all piping of adequate carrying capacity.

The nighttime flow demands are low, and a better test of the system would be for maximum day flow conditions. This is demonstrated in run Pass1-OT2.

TITLE

PASS1.OT3A

CONDITIONS

Maximum day demand = 6800 gpm = 9.8 mgd w/Fire Flow 1500 gpm at Hawthorne Pump Station, plant has two pumps on, no second level pumps on, plant producing 7.716 mgd = 5358-gpm Reservoirs 1, 2, & 3 supplying 425 gpm, Reservoir 4 supplying 2501 gpm.

EXPLANATION

This runs tests the second-level reservoir capacity at maximum day condition with all booster pumps down. Transmission lines from Reservoir 4 into the distribution system are not large enough to serve a fire demand across the second-level distribution system. There are several areas where distribution grids are connected by only one line such as Steiger Street and Beacon Drive. A break in either of these lines would hinder service to several other areas within the second level. The head loss across the system has created negative pressures in the Morgan and Hawthorne Street area and low pressures along Highland, Hawthorne and Washington Streets.

These conditions can be improved by strengthening the distribution grid with the addition of lines on Ninth, Hillcrest and Highland Streets.

The addition of Reservoir 6 greatly improves the second-level water supply capabilities by providing additional storage supply capacity across the system. This is an excellent location in that it divides the system with Reservoir 4 to raise the west side hydraulic grade line and reduce the water demand rate from Reservoir 4.

TITLE

PASS1.OT4

CONDITIONS

Check on field tests and adjustments to pipe absolute roughness factors, water treatment plant shut down, all pump stations off, nighttime flow 1285 gpm = 1.85 mgd; flow from Reservoirs 1, 2, & 3 = 1026 gpm, 4 = 260 gpm.

EXPLANATION

Static pressures were compared with field pressures to adjust roughness factors.

TITLE

PASS1.OT6

CONDITIONS

Reservoir refill, nighttime demand, current year, "M" St. WTP producing 6800 gpm = 9.792 mgd, Lawnridge Pump Station pumping 1100 gpm, Reservoirs 1, 2, & 3 filling rate = 4700 gpm, Reservoir 4 filling rate = 815 gpm.

EXPLANATION

The pressure at the plant is 107.73 psi, which corresponds to a hydraulic grade line elevation of 1174.62. This pressure controls the plants maximum discharge. The water transmission to Reservoirs 1, 2, & 3 is by way of the downtown business district. This area grid system is comprised of 6-inch diameter pipes with considerable head loss resulting. The majority of the second-level demand also passes through this general area to the suction side of the Lawnridge Pump Station. Water demand has reached a level which requires nighttime reservoir refill within a 5- to 6-hour period. The existing transmission lines are overloaded and limit plant production. Future water demands and the addition of a new plant for increased production capability will necessitate new transmission mains.

TITLE

PASS1.OT8

CONDITIONS

Existing system; maximum day demand, current year, 6800 gpm = 9.792 mgd + fire flow @ node 920 = 1500 gpm = 2.16 mgd. All pumps off.

Comments: Total flow from pumps = 0
Total flow from reservoirs = 8239.17

RESERVOIRS 902,903 = 1, 2, & 3 = 5906.66 gpm = Low level demand
901 = 4 = 2333.52 gpm = 2 & 2NR demand + fire flow

EXPLANATION

The fire flow plus maximum day demand for level 2 and level 2NR is from Reservoir 4 only. The velocities in the distribution piping along Savage Street from Ninth Street to Beacon Street and on Steiger and Seventh Streets were in excess of 8 fps. The resultant head losses allowed pressures north of Steiger Street to drop below 30 psi with negative pressures developed north of Hillcrest Drive.

The addition of Reservoir 6 or the operation of Lawnridge or Madrone Pump Stations would provide adequate water supply and pressure should this condition occur.

The extension of a 10-inch line along Ninth Street from node 473 to node 468 and the addition of 12-inch lines along Hillcrest and Hawthorne Streets shall also help water transmission.

TITLE

PASS1.OT9

CONDITIONS

Peak hour demand, year 2000, total flow 27768.6 gpm = 40.0 mgd, "M" Street plant producing 6566 gpm = 9.46 mgd, Parkdale Drive plant producing 27,769 gpm, Reservoirs 1, 2, 3, 4, 5, 6, 7, & 9 on the system.

EXPLANATION

Peak hour demand is used to check line sizes, reservoir locations, and residual pressures. Several future line sizes, 16-inch and greater in diameter, were observed to be too large. Minimum transmission line size is held at 12-inch diameter to meet point fire flow demands. Reservoir 9 was filling at a rate of 5625 gpm which indicates this reservoir may be located too close to the water treatment plants. The hydraulic grade line drops from 1126.75 at the Parkdale Plant to 1108.5 at Reservoir 9. This drop is due to the overflow elevation restricting the grade line from rising. The use of Reservoirs 11 and 10 at their western & eastern locations, respectively, will supply peaking storage and fire flow more effectively.

TITLE

PASS1.T10

CONDITIONS

Peak hour demand, year 2000, total flow 27780 gpm = 40.0 mgd, "M" Street plant producing 6721 gpm = 9.68 mgd, Parkdale Drive plant producing 12084 gpm = 17.4 mgd, Reservoirs 1, 2, 3, 4, 5, 6, 7, 10 & 11.

EXPLANATION

Peak hour demand was used to check the reduced line sizes, new reservoirs, and resulting residual pressures. The new line sizes performed well under this condition and Reservoirs 10 & 11 were utilized in lieu of Reservoir 9 or 12. The residual pressures were in the range of 30 psi to 110 psi with pressures at the water treatment plants near 84 psi.

# Appendix B

| | | |
|---|---|---|
| Reservoir 8 | $ 516,000 | |
| Service line | 341,000 | |
| Hawthorne Pump Station | 90,000 | |
| Champion Pump Station | 150,000 | |
| | $1,097,000 | |
| Total area served | 828 | acres |
| City area | 327 | acres |
| County area | 501 | acres |
| Reservoir 7 | $ 432,000 | |
| Ausland Pump Station | 48,000 | |
| Service line | 160,000 | |
| | $ 640,000 | |
| Total area served | 141 | acres |
| City area | 13 | acres |
| County area | 128 | acres |

Note: Reservoir 8 is sized to serve a larger area than Reservoir 7; yet the volumes respectively are 2.0 MG and 1.2 MG. This is partly due to the service area related to Reservoir 8 relying on a fire flow of 120,000 gallons from Reservoir 7 and 630,000 gallons from lower level reservoirs. Reservoir 8 is sized to contain 750,000 gallons of fire protection storage. The combined total fire storage shall meet the required 5,000 gpm for 5 hours duration or 1.5 MG.

# Appendix C

## Appendix C

## DISTRIBUTION SYSTEM HYDRAULIC ANALYSES - GENERAL DISCUSSION

Computerized hydraulic analyses enable the engineer to determine the effects of system modifications without actually constructing the modifications in the field.

The five phases of computer analysis are briefly outlined below. They are presented in detail in the following sections.

1. The basic considerations involved when studying distribution hydraulics are presented. These apply regardless of whether or not the computations are computerized.

2. The computer program logic is explained. Certain acceptable approximations are used to comply with the computer program's limitations.

3. The information (input) that must be entered by the engineer and the format of the input are described.

4. Analyses of the results (output) of the computations are discussed.

5. The engineering judgment that must be applied to the computations is discussed.

The specific alternative system improvements and results of the analyzed computations are discussed in the body of the report.

The following terms are used in describing the computer analysis procedure and are therefore defined:

- Output is the computer answer.

- Specified input is used to indicate data that are entered by the engineer for each analysis. As an example, demand at a pipe junction (nodal demand) will probably vary for each analysis.

- The model represents the system facilities described mathematically and is part of the input. The term is used to differentiate the specified input from the system facilities, such as pipes and pumps.

## BASIC CONSIDERATIONS

The goal of a distribution system is to meet demands with an adequate residual pressure. This is done by conveying water from a few facilities to many demand areas. However, head loss (energy loss) occurs when water flows through pipes. A simple example is a garden hose conveying water from the faucet (supply) to the sprinkler (demand). The head loss from the faucet to the sprinkler will depend on:

1. The hose's length, interior smoothness, and diameter (long length, rough interior, and small diameter cause increased head loss).

2. The flow through the sprinkler.

3. Minor head loss, such as in the sprinkler, faucet valve constrictions, etc. This third category is normally insignificant in a distribution system and is therefore not considered further.

The hydraulic elevation less ground elevation is the residual pressure. As head loss increases, the hydraulic elevation and pressure are reduced from that existing under static or no-flow conditions. However, even if high head losses occur in a pipe where water is flowing from supply to demand at a lower elevation, the residual pressure at the demand may be acceptable. This is because ground level is reducing and, even though the hydraulic elevation is depressed, the ground elevation is low enough to result in an adequate residual pressure.

A distribution system is more complex than the simple example described above. Because the distribution system is a grid, the number of pipeline combinations through which water can flow from supply to demand is infinite. Water will seek the path--or a combination of paths--of least resistance when flowing from supply to the demand sites. The situation is further complicated by other conditions such as:

- Unlike the sprinkler hose, water in a distribution system can flow from several directions simultaneously.

- Booster pump stations boost water from lower zones, with various pressures at the pump suction, to higher zones. The higher zone pressure is dependent upon the suction pressure and the flow rate. Pump stations at reservoirs and water treatment plants pump from relatively constant suction pressures.

The computer program analyzes instantaneous situations only. Demand, reservoir water levels, and the operating status of pumps are usually not varied during each computer analysis. However, the system must be analyzed using many different instantaneous sets of conditions because reservoir levels and other conditions are continuously changing.

The most realistic, stringent demand criteria for analysis are the maximum hour demand, average demand of the maximum day plus a major fire, or reservoir replenishment. The successful use of computerized analyses depends on the engineer's ability to specify suitable conditions that meet these criteria.

## INPUT

Input is the information the engineer must have and enter into the computer. For the purposes of this discussion, it consists of two categories:

- Information that is not normally changed, such as pipeline size, interior roughness, pump characteristic curves, reservoir location, etc. This information describes the system's physical facilities and is known as the model.

- Information which is continually changing, such as nodal demands and reservoir water levels.

These categories and the constraints associated with them are discussed in the next two sections.

### Model Information

Model information is the mathematical information that describes the distribution system. It is called a model because the computer can compute the effects of varying conditions on a system almost as accurately as if a real system were constructed and pressures measured. Whenever possible, the existing system model is "checked" by specifying flows measured in the actual system and comparing computed pressures with measured pressures.

The model is verified by field survey data of measured flows, reservoir water surface elevations, and pressures. The computer output pressures are compared with the actual pressures. The model is revised as necessary until actual and computer pressures agree.

Specified Information

Specified information is input data, such as nodal demand and reservoir water level. These data are subject to the engineer's judgment. They are of a changing nature as opposed to physical facilities, which do not normally change from day to day.

The method of estimating the nodal demands is described in the report. Reservoir water level is specified at an elevation that reflects the probability that the reservoir is drawn down during the peak hour.

COMPUTER LOGIC

It is beneficial to understand the computer's method of "reasoning" because it affects the interpretation of the analyses. Unlike humans, the computer has no power of judgment. It can perform many computations in seconds, but its methods are plain, simple, and straightforward. The computer program is nothing more than a detailed set of instructions.

The computer analyses began with development of the system model. The system model, which was discussed in detail in the preceding section, is a description of the system facilities and characteristics--pipelines, pump stations, storage and supply locations, etc.

After the system model is mathematically developed (coded), the engineer then specifies certain conditions such as:

- Water surface elevations at reservoirs and number of pumps running at pump stations.
- Demand at each pipeline junction (node) in the sytem.

Adding up demands is the start of computations within the computer.

As discussed above, head loss will occur between the zone "supply" (treatment plant or reservoir) and junction demands. The computer successively "tries" combinations of flow rates and routes from supply to demand until the sum of head losses from any system point to another, after returning to the starting point regardless of route taken, equals zero.

A second constraint is that the sum of inflow and outflow at all junctions must be zero. Water flowing to a node must either be consumed (node demand), replenishing a reservoir, or flowing on to another node.

Water treatment plant high head pumps and reservoir outlet pumps are considered in the computer analyses by modeling the pumps' physical behavior mathematically. This is done by converting the pump curve to a mathematical equation and including it in individual zone analyses. Physically, this equation means that the pump outlet hydraulic grade level (hgl) is the pump head plus the available inlet hgl. Every pump operates according to its characteristic curve. Such a curve indicates the head (TDH) at which it can pump water at any given flow. This head is added to the available inlet hgl to compute outlet hgl.

When these criteria are met, nodal ground elevations are subtracted from the computed nodal hgl. The difference is the pressure head in feet which is converted to psi. As a measure of system performance, these pressures are compared with the City's service standard.

## OUTPUT

The output consists of a portion of the "answers" -- the computed results. Once the model has been verified, the primary output is the nodal pressure because the goal of system operators is to meet demands with at least the City's standard minimum pressure. Other results printed in the output are valuable to the engineer in deciding which system improvements will result in adequate performance.

## ENGINEERING JUDGMENT

The output data are not a complete indication of system performance. Even though the computer is fast and accurate, it lacks the judgment to correct mistakes; it also does not suggest improvements.

As noted above, the computer calculates an instantaneous flowrate from the supply facilities. There is no direct consideration whether this flow can be maintained or whether the storage has adequate capacity. Further analyses are required to consider this because each analysis is for a single instant in time.

Distribution systems also have a characteristic curve, called a system curve, that indicates the head loss caused by flow. When flow increases, head loss increases. The characteristic curves have an effect upon the analysis because the computer is successively attempting to calculate the flow that will result in the pumps' TDH intersecting with the system head loss. Often, there is no physically stable solution (no intersection) to certain combinations of pumps. Therefore, a significant amount of judgment must take place prior to analysis of the system with an alternative configuration. Water treatment plant high head pumps and reservoir outlet pumps have a suction hgl that is relatively constant, and the analysis assumes that clearwell and reservoir capacities are limitless. This point is important to remember when analyzing results.

In summary, the analyses of all pump and reservoir supplied zones in the system have been made using the following criteria.

- Head loss caused between two junctions is the same regardless of the route taken.
- The summation of inflow and outflow at every junction must equal zero.
- Reservoir outlet pumps and water treatment plant high head pumps must behave as the pump curves indicate.

Clearwells are small and do not usually have significant storage capacities when compared with system reservoirs. For this reason, if the computed flow required from a water treatment plant exceeds the plant design capacity, the engineer will probably propose that the plant be expanded or storage be constructed. Alternatively, if other supplies have unused capabilities, a new pipeline may reduce head loss from such a supply, increasing its contributions, and resulting in a redistribution of flows. The burdened water treatment plant's supply rate is decreased.

Sustained supply from storage is analyzed separately. Hourly demands during the afternoon and evening are normally greater than the daily average demands. Therefore, during high demand periods equalizing storage is depleted. During low demands it is replenished. The depleted volume is equivalent to equalizing storage volume.

The ability of each reservoir to complement flow in its service area with supply to that area from the water treatment plant is analyzed using the equalizing storage concept. The distribution analysis output includes direction of flow in each pipe and hydraulic elevation at each node. From these computations, the area served by the reservoir is determined by drawing a line along the lowest hgl's computed between reservoirs, water treatment plants, or any combination of these. The sum of nodal demands in this influence area is equal to the computed reservoir flowrate and is the average maximum hourly demand for the area of influence. Other hourly demands of the same day will vary according to the diurnal demand curve.

The flow from the reservoir, and other supplies into the area, must be sufficient to meet all hourly demand in the area. These flows are a function of the available equalizing storage volume combined with demand rates and available supply to the area. The storage analysis computer program compares the interaction of these three factors -- supply rate, demand rate, and reservoir volume -- as they vary throughout the day. The computation results in a needed equalizing volume and replenishment flow for the specific area's demand pattern.

The major arterial capability to convey the replenishment flow to the area is tested using the computerized hydraulic analysis by specifying an equlivalent reservoir replenishment "demand" and minimum hourly demands simultaneously. If the computed hydraulic elevation available to the reservoir during the replenishment simulation is not as high as the reservoir maximum water level, the reservoir cannot be refilled. Improved arterial capacity or increased equalizing storage volume is then considered.

## CONTINUING USE OF COMPUTER PROGRAM

The computer program can assist water department staff during future operations by determining:

- Effect on the system if individual facilities are taken out of service.
- Adequacy of flushing programs and suggested modifications to provide sufficient velocity.
- Verification of size and location of proposed water mains.
- Location of closed or partially closed valves.
- Capacity of the system to provide fire flows.

The system model used during this study has been retained for future use. Necessary data required for continued use of the program are:

- For each new pipeline constructed, the location, length, diameter, pipe material, and ground elevation at each end.
- Changes in supply location and characteristics.
- Addition or removal of reservoirs with corresponding maximum water surface elevations and capacities.
- Location and normal rate of demand of large customers.
- Recognition of developing areas.

The last item is needed to determine demand for water use in these areas. To determine developing areas, a map should be prepared showing new services, numbers and locations of building permits issued, or an estimate of the number of houses in a newly developed area.