City of Sherwood Oregon

City of Sherwood, Oregon

Resolution No. 99-816

A RESOLUTION ADOPTING THE 1999 WATER SYSTEM MASTER PLAN AND REPEALING PAST WATER MASTER PLANS

WHEREAS, the City of Sherwood originally adopted a Water Service Plan in 1979 and incorporated elements of that Service Plan into the 1981 Comprehensive Plan; and

WHEREAS, the Water Service Plan Update dated May 1988 was adopted through Resolution No. 88-407; and

WHEREAS, the Water Service Plan was again updated in 1991 to incorporate revised population projections and a revised list of capital projects; and

WHEREAS, the 1991 version of the Water Service Plan Update was adopted through Resolution No. 91-502; and

WHEREAS, growth in Sherwood since 1991 necessitates the need to again update the Water System Master Plan and revise the water system capital improvements program; and

WHEREAS, Resolution No. 95-611 establishing Water Supply System Development Charges requires a Long Range Capital Improvements Program listing projects that qualify for use of funds derived from System Development Charges; and

WHEREAS, the City commissioned Bookman-Edmonston Engineering, Inc. to prepare the Water System Master Plan Update dated April 1999; and

WHEREAS, the City commissioned Squire Associates, Inc. to prepare the Municipal Well Field Hydrogeological Evaluation dated August 1999; and

WHEREAS, the City commissioned Murray, Smith & Associates, Inc. to prepare the Southwest Sherwood Service Zone Alternatives study dated September 1999; and

WHEREAS, the City consolidated the recommendations of these reports and prepared a Water System Capital Improvements Program that is contained in the 1999 Water System Master Plan Summary report dated October 1999; and

WHEREAS, the 1999 Water System Master Plan is composed of the above listed reports prepared in 1999.

NOW, THEREFORE, THE CITY RESOLVES AS FOLLOWS:

Section 1: The 1999 Water System Master Plan and Water System Capital Improvements Program contained in the plan summary are hereby adopted.

Section 2: Resolution No. 88-407 and other resolutions adopting earlier Water System Master Plans are hereby repealed.

Section 3: The portions of Resolution No. 91-502 adopting revisions to the Water Service Master Plan and Water System Capital Improvements Program are hereby repealed.

Section 4: The portions of Resolution No. 91-502 dealing with transportation and sanitary sewer are not repealed.

Section 5: This Resolution shall be effective upon its approval and adoption.

Duly passed by the City Council this 12th day of October 1999.

Walt Hitchcock

Walt Hitchcock, Mayor

ATTEST:

C.L. Wiley

C.L. Wiley, Recorder

City of Sherwood, Oregon

1999 Water System Master Plan Summary

October 1999

Walt Hitchcock, Mayor

<u>Council Members</u>

Mark Cottle
Tom Krause
Scott Franklin
Bill Whiteman

Lee D. Weislogel, City Manager Pro Tem

Prepared by:
Terry Keyes, P.E., City Engineer
Robert E. Meyer, P.E., PLS, Consulting Engineer
Nicki Colliander, Engineering Coordinator

REGISTERED PROFESSIONAL ENGINEER
18989PE
OREGON
MARCH 18, 1987
TERRY KEYES
EXPIRES 6/30/01

City of Sherwood, Oregon
Water System Master Plan Summary
October 1999

Components of Plan:

In addition to this summary, the City of Sherwood 1999 Water Master Plan consists of the following documents:

1. City of Sherwood, Oregon Water System Master Plan Update, dated April 1999, authored by Bookman-Edmonston Engineering, Inc, authorized by Sherwood, October 1997.

2. City of Sherwood Municipal Well Field Hydrogeological Evaluation, dated June 11, 1999, authored by Squier Associates, authorized by Sherwood, January 1999.

3. Analysis of Southwest Sherwood Service Zone, dated September 13, 1999, authored by Murray Smith Associates, authorized by Sherwood, February 1999.

4. In addition to the above three documents, Sherwood authorized the preparation of a Water Management and Conservation Plan. This Plan, mandated by Oregon State Water Resources Department (Division) is underway and will be presented to Sherwood for review and adoption shortly. When adopted, it will be an important element of Sherwood's 1999 Water Master Plan.

The above listed documents and this summary titled "City of Sherwood 1999 Water System Master Plan Project Summary" constitute Sherwood's 1999 Water System Master Plan. The last Sherwood Water Master Plan was adopted in June 1991.

Plan Development Process:

The projects listed in the attached Project Summaries are identified as necessary to provide Sherwood's water customer with a reliable, safe and economical product. The guidelines and standards used in identifying the projects were Oregon Administrative Rules, Chapter 333, including ORS 448, Drinking Water Program of the Oregon State Health Division and the American Water Works Association.

Capital Improvements Projects:

Based on the needs analyzed in the plan, a priority list of needed capital improvements for the water system is identified in the attached list. Costs shown are estimated 1999 design and construction costs. These projects are intended to be primarily funded through System Development Charges. When the projects are constructed as part of land development projects, reimbursement to developers for the cost of these projects is governed by the city's ordinances dealing with system development charges.

Absent from the Capital Improvements Program is the cost of connecting to a new water supply from the Willamette River. When the city's participation in this program is solidified, the Water Master Plan will need to be revised.

Water System Upgrades:

Also based on the needs analyzed in the plan, a list of upgrades and major maintenance tasks for the existing water system are identified in the attached list. The cost of these items are intended to be funded primarily through user rates.

City of Sherwood, Oregon

1999 Water System Master Plan

Capital Improvement Program

Priority	Description	Costs Est.
1	**West Sherwood Service Zone** - *Phase 1:* Construct new water main from Wyndham Ridge Booster Station to water system on south side of Hwy 99W at Sunset Blvd. *Phase 2:* Construct a 3.0 Million Gallon ground level reservoir at about elevation 440 feet along Kruger Road with associated water mains from water main constructed in Phase 1.	$ 4,000,000
2	**Scholls-Sherwood Rd. Loop** - Install 12 inch main across Hwy 99W from 10 inch on north side of Tualatin-Sherwood Road northwesterly to connect to 10 inch line in Scholls-Sherwood Road.	$ 385,000
3	**Snyder Park to Sunset Main** - Install 1,600 feet 12 inch mainline from Snyder Park Service Zone Booster Station southerly, through Snyder Park, to intersection of Sunset Blvd. and Aldergrove Avenue.	$ 125,000
4	**Synder Park to Lincoln Main** - Install 300 feet of 24 inch pipe to replace the 16-inch Gravity Zone pipe between the Snyder Park reservoir site and the intersection of Lincoln and Division Streets.	$ 50,000
5	**Replace Undersized Mains** - Replace approximately 14,500 feet of 2, 4, and 6 inch water mains with 8" minimum size water mains.	$ 885,000
6	**Galbreath to Cipole Loop** - Install 2,400 feet of 12 inch pipe along Galbreath and Cipole Roads to connect to system at north edge of BMC West.	$ 270,000
7	**Murdock 24" Main** - Install 5,000 feet of 24 inch pipe along Murdock Road and Division Street from the Regional Supply line to the Snyder reservoir site	$ 790,000
8	**Snyder Park Reservoir #2** - Provide an additional 3.0 Million Gallon reservoir at Snyder Park. (Not required until 2005).	$ 3,000,000
	TOTAL CAPITAL IMPROVEMENTS	**$ 9,505,000**

City of Sherwood, Oregon
1999 Water System Master Plan
Water System Upgrades

#	Description	Costs Est.
	Projects listed here are generally water system upgrades and major maintenance projects. These projects are intended to be funded through on-going water rates rather than system development charges.	
1	**Deepen Well #5** - Increase yield. Well #5 depth was terminated 20 feet above a primary water bearing basalt pillow. If successful, deepening will allow the closing off of the water zone which cascades into the water causing milky (aerated) water.	To be determined
2	**Lower Well #3 Pump Bowls** - Bowls are at 130 feet depth and well depth is at 319 feet. Lowering bowls will help insure a reliable yield.	To be determined
3	**Spada Farm Well Analysis** - The Spada Farm Well is located outside of the Urban Growth Boundary and east of the proposed Home Depot site. The eight inch well drilled 1983 to depth of 500 feet was tested at 400 gallons per minute. The owner has approached Sherwood to investigate the well as a possible source for municipal use. This well is to be investigated and tested as a possible potable source for Sherwood. This well may be a consideration of a municipal irrigation source if it is not economically feasible to improve the Spada Well for use as a potable source.	To be determined
4	**Snyder Park Pressure Zone Booster Station** - This station, constructed in 1996, services the southeastern area of the City.	$ 160,000
5	**Water meter inspection and replacement program**	To be determined

MSA
Murray, Smith & Associates, Inc.
Engineers/Planners

121 S.W. Salmon, Suite 1020 • Portland, Oregon 97204 • PHONE 503-225-9010 • FAX 503-225-9022

SEP 1999
BY:

LETTER OF TRANSMITTAL

To: City of Sherwood
20 NW Washington Street
Sherwood, OR 97140

Attn.: Mr. Robert E. Meyer, P.E., P.L.S.
City Engineer

Date: September 13, 1999

Job No. 99-0404.101

Re: Southwest Sherwood Service Zone Alternatives Review

WE ARE SENDING YOU: ☐ Attached ☐ Under separate cover

☐ Shop Drawings ☐ Prints ☐ Plans ☐ Samples

☐ Specifications ☐ Copy of letter ☐ Change Order ☐ _______

Copies	Date	Description
1	9/13/99	Draft Southwest Sherwood Service Zone Alternatives Memorandum

THESE ARE TRANSMITTED as checked below:

☐ For approval ☐ Approved as submitted ☐ Resubmit__copies for approval

☐ For your use ☐ Approved as noted ☐ Submit__copies for distribution

☐ As requested ☐ Returned for corrections ☐ Return__corrected prints

☐ For review/comment ☐ _______________

REMARKS:

COPY TO:_______________

SIGNED: [signature]
Chris H. Uber

F:\PROJECTS\99\0404.101\Transmittals\Meyer 8-24-99.doc

MSA
Murray, Smith & Associates, Inc.
Engineers/Planners
121 S.W. Salmon, Suite 1020 • Portland, Oregon 97204 • PHONE 503-225-9010 • FAX 503-225-9022

DRAFT

DRAFT TECHNICAL MEMORANDUM

DATE:	September 13, 1999
PROJECT NO.:	99-0404.101
TO:	Mr. Robert E. Meyer, P.E., P.L.S. City of Sherwood
FROM:	Chris Uber, P.E., Murray, Smith & Associates Inc.
RE:	Southwest Sherwood Service Zone Alternatives Review – Engineering Services for West Sunset Booster Pump Station

Introduction and Purpose

On February 19, 1999, Murray, Smith & Associates, Inc. was authorized to prepare an engineering report documenting preliminary design efforts for the proposed West Sunset Booster Pump Station. The original scope of services was expanded to include further hydraulic model development and calibration and consideration of three alternatives for serving the Southwest Sherwood Service Zone. This memorandum summarizes these work efforts and presents findings and recommendations.

Background

The preliminary design efforts for the West Sunset Booster Pump Station were initiated based on recommendations presented in the City's draft Water System Master Plan Update. The intent of the preliminary design study was to confirm the booster pump station service area for existing and future populations, establish design criteria, determine necessary waterline improvements, and establish station capacity needs.

As part of the completed analysis efforts, a service area zone above a ground elevation of approximately 250 feet and east of Highway 99W was identified for the proposed booster pump station. Areas west of Highway 99W were assumed to be served by the Wyndham

Ridge Booster Pump Station. Existing and future water demands were developed for the area east of Highway 99W, in addition to required fire flows for the YMCA and an elementary school proposed for construction south of Colfelt Lane between Highway 99W and Old Highway 99W. The proposed school is on a fast track construction schedule and is planned for completion and occupancy in September 2000. The City of Sherwood must provide water service to the school and is under time constraints to complete the planning, design and construction of needed water system improvements.

As part of work efforts completed for this analysis, the City's existing water distribution system hydraulic model was updated and calibrated in order to evaluate the ability of the existing system to provide adequate flow to the proposed pump station suction piping. Results of the hydraulic modeling showed that significant existing water distribution system improvements are required between the proposed West Sunset Booster Pump Station and the City's existing water storage reservoir located near the intersection of Division and Pine Streets. These improvements are necessary to maintain adequate service pressures within the West Sunset Booster Pump Station service area and within the existing water distribution system.

Concurrent with the West Sunset Booster Pump Station preliminary engineering analysis, the construction of the Wyndham Ridge Booster Pump Station was nearing completion. The new pump station is located west of Highway 99W on SW Handley Street. A July 1998 memo suggested that the service area for the Wyndham Ridge Pump Station included the entire Southwest Sherwood Service Zone. Considering the existing water distribution system deficiencies found in supplying the proposed West Sunset Booster Pump Station, the City's hydraulic model was used to generally evaluate the effect of using the new Wyndham Ridge Booster Pump Station to provide service to the area identified in the July 1998 memo. Preliminary results from the hydraulic modeling showed that significant existing distribution system improvements are required between the Wyndham Ridge Booster Pump Station and the City's main water storage reservoir to maintain adequate service pressures within the existing water distribution system.

Service Alternatives

With the determination that significant distribution system improvements were required to supply the West Sunset Booster Pump Station and/or the Wyndham Ridge Booster Pump Station, four alternatives were identified for further consideration. These alternatives are presented and discussed below.

Alternative 1

Alternative 1 includes the construction of the West Sunset Booster Pump Station to provide service to the Southwest Sherwood Service Zone east of Highway 99W.

Alternative 2

Alternative 2 includes the modification of the new Wyndham Ridge Booster Pump Station to provide for the entire Southwest Sherwood Service Zone.

Alternative 3

Alternative 3 includes construction of a ground level storage reservoir to serve the entire Southwest Sherwood Service Zone by gravity. The new Wyndham Ridge Booster Pump Station would pump to the new reservoir.

Alternative 4

Alternative 4 includes construction of a ground level storage reservoir and a reduced capacity West Sunset Booster Pump Station. The proposed reservoir would provide service to the Southwest Sherwood Service Zone east of Hwy 99W and the West Sunset Booster Pump Station would pump to the new reservoir. The new Wyndham Ridge Booster Pump would provide constant pressure pumping for areas west of Hwy 99W within the service zone.

Planning and Analysis Criteria

Service Area and Land Use

The Southwest Sherwood Service Zone encompasses an area in Southwest Sherwood above a ground elevation of approximately 250 feet with ground elevations up to approximately 320 feet. Total developable acreage within the service zone is approximately 179 acres. Of this acreage, approximately 80 percent is zoned for residential housing with the remaining 20 percent zoned for commercial use. Approximately 11 acres of residentially zoned land is presently identified for construction of a proposed elementary school, which will have an estimated population of 600 students. Although the City's draft Water System Master Plan Update indicates that Urban Reserves lie to the south of the service zone, the majority of these areas are below a ground elevation of 250 feet and are considered outside of the Southwest Sherwood Service Zone.

Population and Water Demand Estimates

Information and data used in determining the population and water demand estimates for the Southwest Sherwood Service Zone and greater Sherwood were taken from the City's draft Water System Master Plan Update.

Population Estimates

City land use plans were reviewed to develop population estimates for the Southwest Sherwood Service Zone. At build-out, approximately 930 dwelling units are anticipated within the service zone. Based on 2.7 persons per dwelling unit, an ultimate population of approximately 2,500 is estimated at build-out.

Water Demand Estimates

The water requirements for the Southwest Sherwood Service Zone include domestic, commercial, public facility and fire protection needs. Average daily water use is estimated at approximately 140 gallons per capita day (gpcd). Based on the estimated maximum saturation population of 2,500 and a maximum daily demand factor of 2.5, the estimated maximum daily demand for the service zone is approximately 0.9 million gallons per day (mgd) or 625 gallons per minute (gpm). A peak instantaneous demand to maximum daily demand ratio is estimated at 2.0 based on service zone size and number of dwelling units. The estimated instantaneous domestic demand for the proposed school is approximately 0.7 mgd or 486 gpm, resulting in a total estimated peak instantaneous water demand in the service zone of approximately 2.5 mgd or 1,740 gpm.

The Southwest Service Zone is comprised of residential, commercial and public facility development each with varying fire flow needs. Residential fire flows for the City of Sherwood are 1,500 gpm as recommended in the City's draft Water System Master Plan Update. The recommended fire flow for the YMCA, located at the intersection of Highway 99W and West Sunset Boulevard, is 3,200 gpm based on discussions with City staff and Tualatin Valley Fire & Rescue (TVF&R) personnel. Although fire flows for the proposed elementary school have not yet been established, the recommended fire flow is expected to be less than approximately 3,000 gpm based on discussions with TVF&R personnel.

Analysis Criteria

The City's hydraulic water system model was used to analyze the Southwest Sherwood Service Zone piping system and the existing water distribution system between the existing water storage reservoir and the pump stations. The following additional criteria and conditions were used to evaluate and analyze the alternatives:

- The booster pump station pressure on the suction side of the pumps should be maintained at or above 85 percent of static pressure when the station is in operation.

- The controlling fire flow demand at the YMCA is approximately 3,200 gpm during a fire event. Pressure at the YMCA must be maintained at or above 40 pounds per square inch (psi) for operation of the building sprinkler system. Pressures elsewhere in the water system must be maintained at or above 20 psi.

- Peak hour demands are distributed throughout the City's water system.

- For Alternatives 1 and 2, estimated instantaneous demands are distributed equally throughout the Southwest Sherwood Service Zone. Using this approach reflects the pump stations' operation function as a constant running pump station providing service under all demand conditions.

- For Alternative 3, estimated maximum daily demands are distributed throughout the Southwest Sherwood Service Zone. This approach reflects the pump station's operation function to supply maximum daily demand to the proposed reservoir.

- For Alternative 4, estimated maximum daily demands are distributed throughout the area within the Southwest Sherwood Service Zone east of Hwy 99W. Instantaneous demands are distributed equally throughout the area within the service zone west of Hwy 99W. This approach reflects the West Sunset Booster Pump Station's operation function to supply maximum daily demand to the proposed reservoir and the Wyndham Ridge Booster Pump Station's operation function as a constant running pump station providing service under all demand conditions.

Findings

The City's hydraulic water system model was used to analyze and evaluate system performance and to determine required facility improvements within the service zone. Figure 1 illustrates system improvements for each alternative.

Table 1 summarizes planning level cost estimates for the alternatives under consideration. The planning level cost estimates include anticipated construction costs and a 40 percent contingency factor for administrative, legal and engineering costs. The estimate for Alternative 4 does not include transmission piping costs to provide fire flow to commercially zoned properties within the service zone for areas west side of Hwy 99W. These improvements would likely occur as development of the area requires.

Alternatives Analysis

Alternative 1 assumes the construction of the West Sunset Booster Pump Station to serve the eastern portion of the Southwest Sherwood Service Zone. The Wyndham Ridge Booster Pump Station would serve the western portion of the Southwest Sherwood Service Zone. The analysis found that extensive waterline improvements between the proposed/existing pump stations and the existing reservoir are necessary to minimize pressure fluctuations on the suction side of the pump stations. For the purposes of this analysis the cost for the main service zone distribution system improvements to serve the Wyndham Ridge Pump Station have not been included in the cost estimate. Additional waterline improvements are also necessary within the service zone to provide fire flow to the new elementary school.

Table 1
Planning Level Project Cost Estimate Summary

Alternative	Estimated Project Cost (millions)			
	Pump Station Construction and/or Modification	Transmission Piping	Reservoir	Total Project Cost
1	\$0.9	\$2.6[1]	-	\$3.5
2	\$0.4	\$3.5	-	\$3.9
3	\$0.2	\$1.5	\$1.6	\$3.3
4	\$0.6	\$1.2[1]	\$1.6	\$3.4

Note: 1. Does not include main service zone distribution system transmission main improvements to serve theWyndham Ridge Booster Pump Station.

Alternative 2 assumes that the Wyndham Ridge Booster Pump Station will serve the entire Southwest Sherwood Service Zone. As with Alternative 1, the existing waterlines between the existing pump station and the existing reservoir require improvements to minimize pressure fluctuations on the suction side of the pump station. Additionally, transmission piping is required to connect the east and west portions of the Southwest Sherwood Service Zone.

Alternative 3 considers the construction of a 1.4 million-gallon reservoir located west of Sherwood on Kruger Road. This reservoir would provide storage for the entire Southwest Sherwood Service Zone. A three component storage volume analysis including operational, fire and emergency storage requirements was performed to estimate the reservoir size for service to the Southwest Sherwood Service Zone. Transmission piping improvements are required across Highway 99W within the service area to link the east and west portions of the service zone and the proposed reservoir. The Wyndham Ridge Booster Pump Station pump units would also require evaluation and potential modification to meet new hydraulic conditions imposed by the proposed reservoir. A pump performance evaluation should be completed as part of further design efforts on this alternative. Improvements to the existing water system between the pump station and the existing reservoir do not appear to be necessary under this alternative. It is anticipated that additional system improvements will be completed as system expansion and upgrades occur within the distribution system. The extent of pump station and other system modifications should be determined through the completion of further Alternative 3 design efforts.

Alternative 4 assumes the construction of a 1.4 million gallon reservoir located west of Sherwood on Kruger Road to serve the eastern portion of the Southwest Sherwood Service Zone. For the purposes of this analysis, the reservoir is sized to serve the entire Southwest Sherwood Service Zone using the same three component storage analysis used under

Alternative 3. As commercial development occurs within the service zone in areas west of Hwy 99W, transmission piping may be extended to provide for recommended commercial fire flow. The West Sunset Booster Pump Station would be constructed to pump to the proposed reservoir at a firm capacity of approximately 330 gpm. Transmission piping improvements between the proposed reservoir and West Sunset Booster Pump Station are illustrated on figure 1. The new Wyndham Ridge Booster Pump Station would serve the western portion of the Southwest Sherwood Service Zone. Additional piping improvements between the Wyndham Ridge Booster Pump Station and the existing reservoir are necessary to minimize pressure fluctuations on the suction side of the pump station. For the purposes of this analysis the cost for the main service zone distribution system improvements to serve the Wyndham Ridge Pump Station have not been included in the cost estimate.

Planning level project cost estimates show that all four alternatives are relatively close in total first cost with Alternative 3 being the least expensive option. The selection of Alternatives 1or 2 requires significantly more pipeline improvements than Alternative 3. Alternatives 1 and 2 will result in major construction disruptions through central Sherwood. Alternative 3 will require substantial pipeline construction but the work will be in areas where disruption will be minimal. The operation and maintenance costs associated with Alternatives 1, 2, and 4 would be higher since each alternative involves a pump station(s) with constant running pump units and must meet instantaneous demands and fire flow needs by direct pumping.

Selection of Alternative 3 provides the City with an opportunity to provide additional system storage. Consideration may also be given to increasing the proposed reservoir's storage to potentially provide emergency and fire storage to Urban Reserves and to portions of the City's main service zone.

Schedule

Figure 2 presents a preliminary project schedule for planning, design, and construction of Alternative 3, a 1.4 million gallon reservoir on Kruger Road and associated waterline improvements. The schedule is based upon a typical design-bid-build procurement process for the facilities. Under this procurement method, the pipeline work could be completed prior to the school opening in September 2000. The reservoir construction is shown to be completed by the spring of 2001, almost a year after occupancy of the proposed elementary school. This schedule assumes that the reservoir is constructed of fabricated steel or prestressed concrete. The steel reservoir, while quicker to construct, requires a period of good weather for painting. The prestressed concrete reservoir requires no painting but requires a longer time to construct. A bolted steel reservoir could be considered as an alternative. It can be constructed in any weather condition and does not require painting.

The City could consider an accelerated procurement process under State law that would allow earlier procurement of the reservoir and potential completion prior to the school occupancy date of September 2000. Provisions under OAR 125-310-003-0 are available for the City to declare an emergency and enter into a construction contract without competitive bidding. An accelerated competitive selection process could be utilized to assure reasonable pricing of the reservoir construction even under accelerated conditions. The City should seriously consider an approach such as this to provide a complete water supply system by the time of occupancy of the school.

The option of providing interim water supply to the school but without gravity storage can be considered. The Wyndham Pump Station, however, could not deliver peak design flows due to the limitations on the suction supply to the station. This would not be a desirable situation for a public school facility to be without full water supply and fire flow capacity.

The reservoir and portions of the pipeline will be located outside the Urban Growth Boundary and in an exclusive farm use area. In addition to a land use permit from Washington County, a special land use process will be required to obtain approval for installation of these facilities in an EFU zone. House Bill 2865 (1999 legislative session) provides for a new process to obtain such approval. It is not anticipated that this process will cause undue delay to the project.

Recommendations

Based on the analysis and evaluation presented above, it is recommended that the City pursue the development of Alternative 3. It is also recommended that preliminary design efforts be undertaken immediately and include the following elements:

- Completion of a comprehensive storage analysis to confirm the need for additional storage within the City's water distribution system. This effort may determine that the size of the proposed reservoir should be increased to provide additional system storage.

- Evaluation of the extent and nature of required improvements to the Wyndham Ridge Booster Pump Station.

- Confirmation of the need for and extent of isolation piping and additional connections between the Southwest Sherwood Service Zone and the main zone.

- Acquisition of reservoir site and commencement of land use permitting.

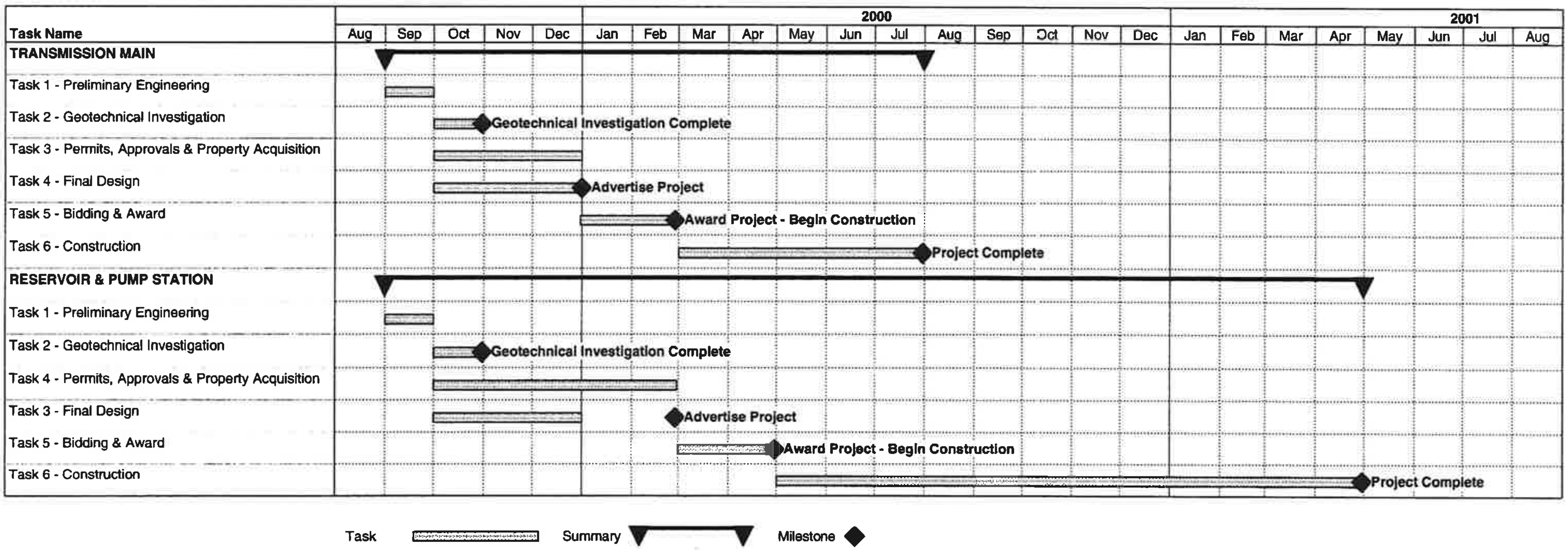
- Evaluation of the project schedule and the potential need for an accelerated project procurement process to complete the reservoir and pump station improvements prior to school occupancy.

- Evaluation of interim water supply and fire flow protection measures and options for the proposed elementary school if the accelerated schedule cannot be achieved.

Conclusion

The initial preliminary design efforts for West Sunset Booster Pump Station were expanded and modified to include a comprehensive pressure zone service evaluation of four alternatives. The analysis found that Alternative 3, construction of a 1.4 million gallon reservoir, will provide operational, fire and emergency storage for the entire Southwest Sherwood Service Zone at an estimated planning level project cost of $3,300,000. Preliminary design should be initiated immediately for Alternative 3 to confirm the reservoir size, location and associated facility and waterline improvements. In addition, alternative project procurement methods should be explored immediately to determine the potential ability of the City to complete the project prior to occupancy of the new school.

FIGURE 2
PRELIMINARY PROJECT SCHEDULE ALTERNATIVE 3 - SOUTHWEST SERVICE ZONE
CITY OF SHERWOOD, OREGON



City of Sherwood Oregon

WATER SYSTEM MASTER PLAN UPDATE

APRIL 1999

PREPARED BY

BOOKMAN - EDMONSTON
ENGINEERING, INC.
4915 SW Griffith Drive, Suite 300
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(503)643-7531, (503)626-7422 FAX

CITY OF SHERWOOD, OREGON

WATER SYSTEM MASTER PLAN UPDATE

APRIL 1999

REGISTERED PROFESSIONAL ENGINEER
18,330
Kenneth A. Condit
OREGON
MARCH 19, 1986
KENNETH ARTHUR CONDIT

PRESENTED BY

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CITY OF SHERWOOD
WATER SYSTEM MASTER PLAN UPDATE

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CITY OF SHERWOOD
WATER SYSTEM MASTER PLAN UPDATE

EXECUTIVE SUMMARY

PURPOSE AND SCOPE

This Water System Master Plan Update has been authorized by the City of Sherwood to address the needs of the water system through the Year 2017. This includes a preliminary evaluation of the impacts associated with developing the planned Urban Growth Reserve. As a planning document, it is designed to help in the establishment of a capital improvements plan for the current water system. The conclusions of the report provide descriptions of the recommended improvements and an opinion of probable project cost for each item.

The scope of this Master Plan Update is consistent with the work proposed in the following documents.

- Scope of Work prepared by Bookman-Edmonston Engineering, Inc. (B-E) and transmitted to the City with the proposal letter dated October 29, 1997.
- Scope revisions described in correspondence sent from B-E to the City dated May 21, 1998, June 22, 1998, and October 26, 1998.

This phase of the Master Plan Update concentrates on the water storage and distribution system. Projections of water supply capacity requirements are included in this report. However, the current supply facilities and alternative sources of supply will be evaluated as part of a separate planning phase.

DESCRIPTION OF EXISTING SYSTEM

General. A map showing the major components of the existing water system is presented in Figure 2-1. A schematic of the system is shown in Figure 2-2.

Source of Supply. Historically, groundwater has been the only source of drinking water for the City of Sherwood and it will continue to be the primary source over the next several years. Limited quantities of surface water from the City of Portland's Bull Run supply will be available when the Bull Run Connection described below is placed into service. Eventually, this connection is expected to become the primary means for delivering drinking water to the City's system. Surface waters directed through the Bull Run Connection may be drawn from either the Bull Run system or alternative sources such as the Willamette or Clackamas Rivers.

Supply System. The City of Sherwood's water supply currently consists of four municipal production wells located within the City limits. These wells feed into the main service zone of the City's water distribution system. Descriptive data for these wells are presented in Table 2-2 on Page 2-4 of this report. The total permitted production capacity of the four wells is 2500 gallons per minute; however, the actual total production capacity measured during the summer of 1997 was approximately 1850 gallons per minute.

The Bull Run Connection has recently been installed and connects Sherwood's water system to that of the City of Tualatin. The facilities primarily consist of a four-mile long, 24-inch pipeline with control valves at each end. The project is substantially complete, but some segments of the pipeline have yet to pass a final pressure test at the time of this writing. Like the wells, this Tualatin intertie will deliver water into the system's main service zone. The connection was designed for a future maximum capacity of 12 MGD.

A second intertie with the Tualatin water system is also available along Cipole Road in the northeast corner of Sherwood as an emergency backup.

Treatment. The only treatment currently provided to the water supply by the City is the addition of sodium hypochlorite at each well for disinfection.

Storage. Water is stored in a 2.0-million gallon (MG) circular concrete reservoir. The elevation of the water surface in the tank typically varies from 375 to 379 feet above sea level. This operating level is used to maintain water pressure in the system's main service zone.

Booster Pumping Station. Higher ground elevations in the southeastern part of the City make the use of a booster pumping station necessary to serve that area. A new booster pumping station was constructed at the existing reservoir site in 1997 to replace the old station located at the same site. The pumping station draws water from the reservoir and delivers it into the distribution system. The total booster station design capacity is intended to satisfy the projected peak demands plus fire flow requirements for the tributary service zone.

Distribution System. The City's water distribution system consists primarily of two service zones. The main service zone operates off the free water surface in the reservoir and is, therefore, typically referred to as the "Gravity Zone." The portion of the system fed by the booster station is referred to as the "Pressure Zone."

There are also four small, isolated intermediate zones in the distribution system that are served from the Pressure Zone. Pressure reducing valves are used to separate the intermediate zones from the Pressure Zone to prevent excessive operating pressures from occurring in these areas.

In general, the distribution system is fairly well looped to maintain reliable service; however, fragmented development on the northwest side of State Route 99W has produced unlooped sections in that part of the Gravity Zone.

Due to recent development, much of the City's water system is relatively new and unaccounted for water is estimated to be only about 5 to 6 percent of the total water produced.

System Control. The water system is controlled by computer at the Public Works Building. The operations of the wells, the booster station and the reservoir are monitored automatically and reported to the computer by telemetry. Operators can control these system components remotely at the computer terminals. The Bull Run Connection will also be monitored and controlled in this manner when it is placed into service.

EXISTING SYSTEM DEFICIENCIES

Production Wells. The data presented in Tables 2-2 and 2-3 indicate the well system suffers from following two general deficiencies:

- The groundwater level in Well No. 3 during drawdown is very close to the bowl setting; and
- The production capacities of Wells No. 3 and No. 4 during the high-demand summer months are substantially less than the reported design capacities.

As a separate phase in the Master Planning effort, the City will evaluate well production capacity and the feasibility of upgrading the wells to restore their capacity.

The only other well deficiencies that were identified have been corrected during the preparation of this report.

Treatment. Elevated levels of iron and manganese in the discharge from Well No. 6 have resulted in customer complaints. An evaluation of alternative treatment methods to alleviate this problem is being conducted by the City separately from this report.

Booster Station. The equipment in the booster station is new and in good condition. The one deficiency in the system's operation is currently being addressed by the Public Works Department. Demands typically are only a small fraction of the design capacity of one 50-hp pump. To improve system efficiency, the installation of a smaller pump is planned. This smaller pump will be used to satisfy demands under most conditions with a lower power draw.

Reservoir. The only reservoir operating deficiency mentioned by Public Works is the lack of any level gauge on the tank to allow operating staff to verify the water level visually while at the tank site. This capability can be added to the new reservoir that will need to be constructed as discussed in Section Five.

Undersized Distribution System Piping. The City has developed standards for minimum pipe sizes in the distribution system. To bring the distribution system up to the current standards, most pipe under 8 inches in diameter would need to be replaced with new pipe having a minimum diameter of 8 inches. Table 3-1 (Page 3-3) identifies the pipe sections 6 inches and smaller that need to be replaced to meet the City's minimum pipe size standards. Based on the breakdown in Table 3-1, the total length of undersized pipe is approximately 17,000 lineal feet (l. f.).

The City plans to replace the 2,400 l. f. of 6-inch line in Oregon and Lincoln Streets with a 12-inch pipe. This size has been proposed to improve capacity between the Bull Run Connection and the reservoir.

One upgrade in distribution system capacity currently under construction is the installation of approximately 900 l. f. of 12-inch pipe under Pine, Columbia and Washington Streets.

Distribution System Operating Pressures. Problems with inadequate service pressures in some areas of the Gravity Zone have resulted in customer complaints. The City has instituted improvements in those areas that are adjacent to the Pressure Zone. One other area experiencing low operating pressure that must still be addressed is in the southwest corner of town. Service connections near Highway 99W and Sunset Boulevard are at an elevation that is too high for recommended system pressures to be maintained. Other connections just east of Middleton Road also are reported to experience inadequate pressures. The City plans to address this problem by having a booster station constructed to serve this area.

POPULATION AND WATER DEMANDS

Population. Table 4-1 (Page 4-1) presents population figures for the recent past. The average household occupancy is currently estimated to be about 2.9 people per dwelling unit (DU).

The December 1997 *Urban Growth Report* prepared by Metro projected a population of 18,566 and a total of 7,002 dwelling units for Sherwood in the Year 2017. These projections were prepared based on the current City boundaries. The population and housing figures for 2017 equate to an average per household occupancy of 2.65.

The establishment of a 460-acre Urban Growth Reserve has been planned to the south of the current City limits to accommodate additional development. Metro will decide in 1999 whether to approve a shift in the Urban Growth Boundary to include this land. Development there could begin before the end of 2000. Metro has estimated that the Urban Reserve could accommodate a total of 2,067 housing units. Since an Urban Reserve Study has not yet been prepared to provide initial planning data, this report will assume that all 2,067 housing units will be constructed by 2017. At a future per household occupancy of 2.65, this translates to an additional population of 5,480. The combined population for the expanded City limits is projected to be about 24,050 in 2017.

Since no projections are available from Metro for intervening years, the City Planning Department has proposed that a range of intermediate population projections be developed. This range has been identified from minimum and maximum near term housing construction rates of 200 and 500 dwelling units per year through 2002. These figures can be correlated to a most favorable and a least favorable development climate. Since the 20-year projection is set, the growth rates for the period from 2002 to 2017 have been calculated to offset the assumed near-term trend. Therefore, the high near-term growth rate would be followed by a lower annual growth rate and the low near-term growth rate would be followed by a higher annual growth rate. Table 4-2 lists the two population projections developed from the method described above. Figure 4-1 presents these projections graphically.

Customer Water Demands. Monthly water production records for 1994, 1995, 1996 and 1997 were used to update estimates of use. The water consumption data suggest a significant decline in per capita water use over the past four years. This trend is also supported by the fact that previous Water System Master Plans used an average per capita water demand of 160 gpd per capita.

The following factors suggest the sharp drop-off in per capita water use may not be representative of an actual long-term pattern.

- Unusually heavy rainfall between late 1995 through October 1997 probably reduced water demands.
- The rapid rate of development may have reduced the accuracy of population estimates.
- Commercial and industrial development in the City has been lagging behind residential development, but may catch up in the future.

Some of the reduction in average per capita water use may, however, be part of a permanent shift due to voluntary limitations the City has instituted for irrigation, particularly at the local schools. Additionally, revisions to the Plumbing Code that require low-flow plumbing fixtures for new homes may be reducing indoor demands. There also may be a trend toward smaller lots which can reduce per capita demands for irrigation.

A total average per capita demand of 125 gpd has been used for planning purposes in this report. This figure is slightly above the overall average per capita water production from 1994 through 1997. Because it is based on total production, this estimate includes an allowance for system losses through leakage.

Total peak daily demand projections have been estimated using a peaking factor of 2.75. This translates to 350 gpd per capita, including system losses. For the network analyses performed on the distribution system a short-term peak 6-hour demand that is 20 percent above the peak daily rate was assumed. The resulting peak per capita demand is 420 gpcd.

Projected water demands were calculated over the 20-year planning period using the per capita water requirements identified above and the population projections in Table 4-2 (Page 4-3). These water demand projections are presented in Table 4-4 (Page 4-6).

WATER SUPPLY AND STORAGE CAPACITIES

Water Supply Capacity. Water supply capacity is becoming a critical issue in Sherwood as demands continue to increase rapidly. The potential for developing additional wells within the City is limited by Chapter 690, Division 502 of the Oregon Administrative Rules. These rules place virtually all of Sherwood inside Groundwater Limited Areas. Additionally, the Bull Run Connection will have a limited impact on supplies until an alternative primary water supply is developed that can be delivered through this pipeline. Limitations on well supply capacity heighten the importance of promoting water conservation in the near term to reduce problems of water shortages.

Existing water supply capacities have been calculated based on the premise that the production capacity of all the wells will remain at the level reported for August, 1997. Operating times of 8 and 20 hours per day were assumed to identify average and peak daily well supply rates, respectively. Limiting the hours of operation reduces wear on equipment, provides opportunities for preventive maintenance, and limits the demand on groundwater resources. Calculating the system capacity at these reduced operating times also builds some backup capability into the system. Using these criteria, the total average and peak daily capacities of the well system are 0.89 and 2.23 MGD, respectively.

Supply capacity deficits are presented in Table 5-2 (Page 5-3). These supply deficits are based on the projected peak daily demands and peak-day well capacities. Figure 5-1 also presents a graph that illustrates these deficits.

It is recommended that the City pursue the following courses of action to meet system demands.

- **Well Capacity** - As a separate phase in the Master Planning effort, the City needs to evaluate well production capacity and the impact the wells are having on groundwater levels. The feasibility of upgrading the wells so they can operate at their permitted capacities should be addressed. The City is currently having an investigation of Well No. 3 completed as part of that separate Planning effort.

- **Conservation** - It is recommended the City use the recommendations of the Water Management and Conservation Plan once it has been completed to implement a water conservation program. Development of this alternative resource could both reduce near-term shortages and save money on long-term improvements.
- **Bull Run Connection** - Discussions should be initiated with the Cities of Tualatin and Portland regarding an increase in the agreed upon supply rate from this source.

Water Storage Capacity. Typically, water storage tanks are sized to provide enough volume to meet peak daily demands plus fire flow requirements. A surplus allowance of 10 percent is also commonly included in the design capacity as a cushion against emptying the tank in emergencies.

Table 5-3 (Page 5-4) presents the required net storage expansion capacities determined from the criteria mentioned in the previous paragraph. The volumes have been calculated for both population growth alternatives using the peak daily demands listed in Tables 4-4 (Page 4-6). The effective storage capacity of the existing 2.0 MG reservoir has been subtracted from the total required volumes to identify the net expansion capacities listed.

Table 5-3 indicates that about 9.0 MG of additional storage capacity will be required by 2017 if the Urban Growth Reserve is developed. Since most of that expansion would be required by 2007, it may be cost effective to construct a single tank to provide the volume required through 2017. However, it may be more appropriate to expand reservoir capacity in phases due to the uncertainty regarding development in the Urban Growth Reserve. Updated planning data on the Urban Reserve could be combined with information on the early results of water conservation efforts to revise future storage capacity requirements. The condition of the existing 2.0 MG tank could also be evaluated prior to future reservoir expansions.

A 6.0 MG reservoir, in conjunction with the existing tank, is projected to be the City's needs through 2005 at the maximum near-term growth rate and through 2009 at the minimum near-term growth rate. A 3.0 MG tank would then be needed to satisfy the criteria 20-year storage requirements, if the existing tank is kept in service.

Section Seven presents opinions of probable project costs for constructing a 6.0-MG tank initially and a 3.0-MG tank in 2005. The probable project cost for one 9.0 MG tank is also included. It is recommended that the City plan to construct either a 6.0- or 9.0-MG reservoir as soon as is practical.

NETWORK ANALYSIS

General. Computer network modeling of water distribution systems is performed to identify areas that may suffer from inadequate or excessive pressures. Under most conditions, pressures in municipal water distribution systems should fall within a range of about 50 to 80 psi during normal operations. However, for practical purposes, pressures between 40 and 90 psi are often considered acceptable. The Uniform Fire Code requires a minimum pressure of 20 psi for the supply of fire flows.

Network Models. The City of Sherwood's water distribution system was modeled with the CYBERNET 3.0 program from Haestad Methods, Incorporated. The water distribution system was analyzed under four general conditions by developing alternative base models. The alternative models are summarized below.

1. **Model One** - system model using 1997 operating conditions and demands.
2. **Model Two (Immediate Future)** - model based on projected 1999 conditions.
3. **Model Three (Near-term Future)** - model based on projected 2002 conditions.
4. **Model Four (20-Year Projection)** - model based on Metro's 20-year population projection and full development of the Urban Reserve.

Separate network models were developed and analyzed for the Gravity and Pressure Zones. The intermediate zones were included in the Pressure Zone network. A more detailed network model of the Woodhaven area was also developed to check the results of the Gravity Zone model for that part of the system.

Analyses have been run with the wells both turned on and off. The Bull Run Connection was assumed to supply a steady rate of 125 gpm for Models Two and Three. Model Four was run with supply rates of 4.0 MGD and 6.0 MGD being fed through the Bull Run Connection.

Modeling of Demands and Fire Flows. Projected peak 6-hour demands were used for the base conditions in the network analyses. The total demands distributed through the system in Models Two, Three and Four roughly correspond to the total peak demands listed in Table 4-4 (Page 4-6) for 1999, 2002 and 2017, respectively. The breakdowns of water use by customer type presented at the end of Section Four were used to distribute demands throughout the system.

Separate fire-flow alternatives were modeled to analyze the system's ability to provide the recommended flow rates at a pressure of at least 20 psi. The flow rates used generally ranged from 1,500 gpm for residential areas to 3,500 gpm for the schools. Actual fire-flow requirements for specific structures are outlined in the Uniform Fire Code based on building construction type and square footage. Without this specific information for each structure in the City, the values used represent conservative estimates.

Results and Recommendations. In general, the results of the network analysis verified the problem areas previously identified by the City. Several other issues were also identified that will need to be addressed. Please refer to Section Six for a summary of the analysis results. Recommended improvements to the distribution system are summarized below.

RECOMMENDATIONS AND PROBABLE PROJECT COSTS

Summary of Capital Improvement Projects.

A. **Treated Water Storage**

1. Current Phase: a new 6.0 MG concrete reservoir should be constructed on City-owned land adjacent to the existing reservoir to increase storage capacity. This tank should meet the City's water storage requirements through at least 2005. The design and construction of the reservoir should proceed as soon as possible. **Probable Project Cost: $ 3,800,000.**
2. Future Phase: it is projected that an additional storage capacity of 3.0-MG would be needed to provide adequate storage volume through 2017. This capacity requirement should be reevaluated by 2005. **Present worth of probable project cost for a single 3.0-MG concrete tank: $ 1,925,000.**
3. Alternative: One 9.0-MG concrete tank. **Probable Project Cost: $ 5,100,000.**

B. **Southwest Booster Station**

A booster station that serves those portions of the Woodhaven Subdivision and adjacent areas that lie above elevation 245 should be installed. **Probable Project Cost: $ 700,000.** The addition of an 8-inch pipeline intertie across Highway 99W would add another **$ 150,000** in probable project costs.

C. **Distribution System**

The following piping improvements are recommended to upgrade the water system. Figure 7-1 should be referred to for project locations. In some cases parallel pipes could be installed to increase capacity instead of replacement lines. This can reduce material costs, but it also would leave older pipes in service. To be conservative, this report assumes replacement pipes will be installed.

1. Increase capacity of key water mains:
   - Install 1,600 l. f. 12-inch pipe from the Pressure Zone booster station through the park site to the intersection of Sunset Boulevard and Alder Grove Avenue. The upgrade is needed to deliver fire flows to the southerly portion of Alder Grove Avenue and the area of Highpoint Drive and Cascara Terrace. **Probable Project Cost: $ 125,000.**

- Install 900 l. f. of 12-inch piping under Pine, Columbia and Washington Streets to increase capacity and replace leaking 8-inch water mains. **Probable Project Cost: $ 140,000.**
- Install 2,950 l. f. of 8-inch piping and 300 l. f. of 12-inch piping to replace the 6-inch water lines under Gleneagle Drive and Twelfth Street. **Probable Project Cost: $ 195,000.**
- Install 2,400 l. f. of 12-inch piping to replace the 6-inch water lines under Lincoln and Oregon Streets. **Probable Project Cost: $ 220,000.**

2. Installations of water lines to complete system loops:
    - Install 3,000 l. f. of 12-inch pipe across Highway 99W under Tualatin-Sherwood and Tualatin-Scholls Roads to complete system loop. **Probable Project Cost: $ 385,000.**
    - Install 1,500 l. f. of 12-inch pipe from the north end of Roellich Avenue to Edy Road to complete system loop. **Probable Project Cost: $ 130,000.**
    - Install 500 lineal feet of 12-inch pipe northwest from Highway 99W near Cedar Creek. **Probable Project Cost: $ 40,000.**
    - Install 2,400 lineal feet of 12-inch pipe along Galbreath and Cipole Roads to connect to existing and proposed water lines. **Probable Project Cost: $ 270,000.**
3. Increase transmission main capacity:
    - Install approximately 5,000 lineal feet of 24-inch pipe along Murdock Road and Division Street from the Bull Run Connection to the existing reservoir site. **Probable Project Cost: $ 790,000.**
    - Install approximately 300 lineal feet of 24-inch pipe to replace the existing 16-inch Gravity Zone pipe between the existing reservoir site and the intersection of Lincoln and Division Streets. **Probable Project Cost: $ 50,000.**
4. Increase diameter of undersized water lines:
    - Replace approximately 11,300 l. f. of 2-, 4- and 6-inch pipe lines with 8-inch pipe. **Probable Project Cost: $ 690,000.**

D. Total Capital Improvements for Storage and Distribution Systems

A summary of the probable project costs itemized above is tabulated in Appendix D. The total for the reservoir and distribution system capital improvements is $9,610,000. This assumes two reservoirs would be constructed in phases.

Other Recommended Projects. The following projects are not part of the recommended capital improvements, but should be initiated to ensure that the water system can meet the City's needs.

1. As a separate phase in the Master Planning effort, the City needs to evaluate well production capacity and the impact the wells are having on groundwater levels. The feasibility of upgrading the wells so they can operate at their permitted capacities should be addressed. The City is currently having an investigation of Well No. 3 completed as part of this separate Master Planning effort.
2. Institute the recommended shifts in service zone boundaries as described in Section Six (See Figure 6-7).
3. An alternative water supply must be obtained to augment and potentially replace the municipal wells. The City should continue to participate in regional planning efforts to develop the alternative supply within 4 years or as soon after that as is practical. An additional supply capacity of about 6.2 MGD is needed by the Year 2017 based on current projections with the Urban Reserve included. This assumes the well production will be at August 1997 levels.
4. A Water Conservation and Management Plan should be completed and an ongoing program of water conservation measures should be implemented. Water conservation can reduce reliance on the wells and alleviate possible near-term water shortages before the additional storage capacity is available.
5. Implement a systematic water meter inspection and replacement program to remove meters that no longer function properly.
6. Develop a schedule for periodically flushing fire hydrants throughout the system.
7. Have a structural analysis of the existing reservoir completed in five to ten years.

CITY OF SHERWOOD
WATER SYSTEM MASTER PLAN UPDATE

SECTION ONE

INTRODUCTION

BACKGROUND AND OBJECTIVE

The last Water Service Plan Update for the City of Sherwood was prepared in 1991. Since that time, the City has undertaken a series of improvements to the existing water system to upgrade service and meet increasing water demands. Additional improvements are, however, still needed to keep up with the City's rapidly growing population.

The preparation of this Master Plan Update has been authorized by the City to evaluate the existing water system and address system needs over the next 20 years. The authorization was provided by City Council Resolution No. 97-717 passed on December 9, 1997. A copy of the resolution is included as Appendix A to this document. As a planning document, this report is designed to help in the establishment of a capital improvements plan for the current water system. The conclusions provide descriptions of the recommended improvements and an opinion of probable project cost for each item.

PLANNING AREA

Sherwood is located in southeastern Washington County at the southwest corner of the Portland metropolitan area. The Urban Growth Boundary (UGB) generally coincides with the City limits except at the northeast corner of the City where a common boundary is shared with the City of Tualatin. Since the surrounding unincorporated areas are outside the UGB, development there is severely restricted. Sherwood extends water service only to the City's inhabitants; thus there are currently no service connections outside the City limits.

An Urban Growth Reserve has been planned to the south of the City for future development. The City anticipates that Metro will soon approve a shift in the UGB to include this land. Preliminary assumptions regarding development within this Urban Reserve have been incorporated into this report. Figure 1-1 shows the current Urban Growth Boundary and planned Urban Reserve Boundaries for the City of Sherwood.

SCOPE OF WORK

The scope of this Master Plan Update is consistent with the work proposed in the following documents.

- Scope of Work prepared by Bookman-Edmonston Engineering, Inc. (B-E) and transmitted to the City with the proposal letter dated October 29, 1997.

- Scope revisions described in correspondence sent from B-E to the City dated May 21, 1998, June 22, 1998, and October 26, 1998.

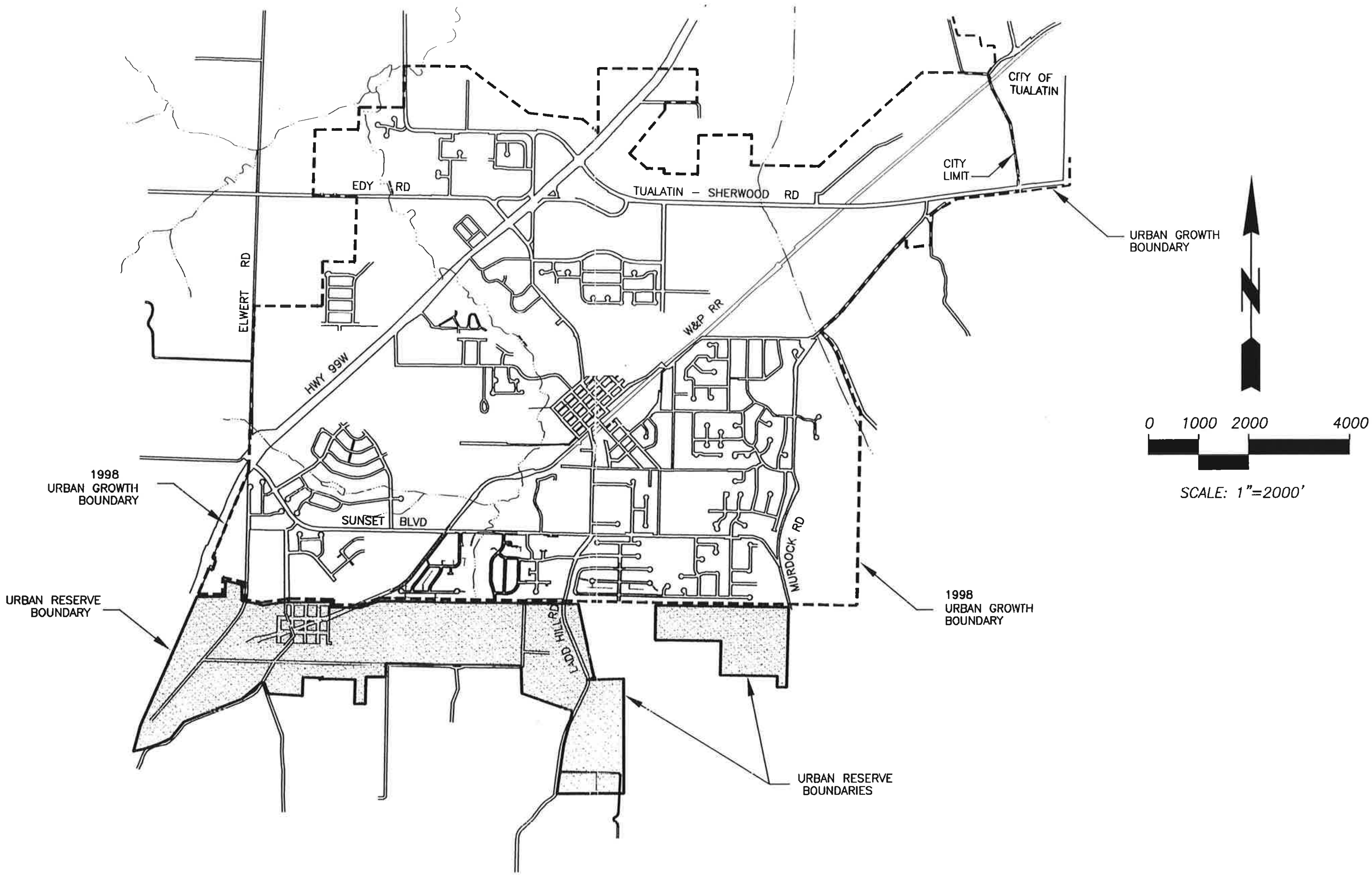
This Master Plan Update focuses primarily on the City's water distribution and storage system, but also identifies the water supply capacity required to meet projected demands. The scope of the report basically covers the following items:

- a description of the existing system and system improvements currently in progress;
- an update of population and water use projections;
- an updated distribution system network analysis to characterize existing and projected conditions;
- evaluations of improvement options to remedy distribution system deficiencies;
- discussions of long-term system upgrades required to meet future needs; and
- a summary of system improvement costs.

In accordance with the City's current planning needs, the Master Plan Update does not serve as a comprehensive plan in that it does not evaluate the following:

- alternative water supply and treatment options;
- user rates and other aspects of system funding;
- the conditions of existing pieces of equipment; and
- source water quality and protection.

The City is required to prepare a water conservation and management plan as a condition of the latest production well permit obtained from the State Water Resources Department. That report will be prepared under a separate scope of work.



CITY OF SHERWOOD	WATER SYSTEM MASTER PLAN UPDATE	FIGURE 1-1
BOOKMAN-EDMONSTON ENGINEERING, INC. BEAVERTON, OREGON	URBAN GROWTH BOUNDARIES	DATE 3/26/99

CITY OF SHERWOOD
WATER SYSTEM MASTER PLAN UPDATE

SECTION TWO

DESCRIPTION OF EXISTING WATER SYSTEM

GENERAL DESCRIPTION

The City of Sherwood's water supply currently consists of four municipal production wells. The wells are all located within the City limits and feed into the main service zone of the City's water distribution system. This zone serves all but the southeastern portion of the City.

The Bull Run Connection, a pipeline that connects Sherwood's water system to the City of Tualatin's system, has recently been installed. This connection, when it is placed into service, will be used to supply Sherwood with water from Portland's Bull Run supply system. Like the wells, this Tualatin intertie will deliver water into the system's main service zone.

A second intertie with the Tualatin water system is also available along Cipole Road in the northeast corner of Sherwood as an emergency backup.

Water is stored in a 2.0-million gallon (MG) reservoir and the operating level in this tank is used to maintain water pressure in the main zone. Because the main zone operates off the free water surface in the reservoir it is typically referred to as the "Gravity Zone" and will be labeled accordingly in this report.

Higher ground elevations in the southeastern part of the City necessitate the use of a booster pumping station to serve that area. The pumping station draws water from the reservoir and delivers it into the distribution system. The portion of the system fed by the booster station is referred to as the "Pressure Zone."

There are four small, isolated intermediate zones in the distribution system that are served from the Pressure Zone. Four pressure reducing valves (PRVs) are used to separate the intermediate zones from the Pressure Zone, thus preventing excessive operating pressures from occurring in these areas. The latest of these intermediate zones has been created during the preparation of this report. Therefore, the initial system network analysis, as described in Section Six, included the fourth intermediate zone as part of the Gravity Zone.

Figure 2-1 presents a map showing the major components of the existing water system and the boundaries for the different service zones in the distribution system. A schematic of the system is shown in Figure 2-2.

SOURCE OF SUPPLY

Groundwater Use. Historically, groundwater has been the only source of drinking water for the City of Sherwood and it will continue to be the primary source over the next several years. The production wells draw water from several aquifers that exist in the underlying Columbia River Basalt Group. This geologic formation was created by a series of lava flows of Miocene age. Basalt tends to be of low permeability and does not yield much groundwater; therefore, the aquifers exploited by Sherwood generally coincide with the interflow zones that occur between the successive lava flows. Fractures, flow breccia and weathering that can exist along these zones produce the permeability needed for favorable groundwater conditions. More information is provided on aquifers and wells in the Sherwood area in Groundwater Report No. 40, prepared by the State of Oregon Water Resources Department (1994). This study specifically covers the Parrett Mountain area, immediately to the south of Sherwood.

Records provided by the City's Public Works Department of groundwater levels at the three older wells indicate that the groundwater surface has dropped in recent years. Table 2-1 compares the 1994 summertime static groundwater levels in each well to the levels in 1997. The levels represent the approximate depth of the groundwater surface below each well pump discharge pipe when the well pump has been turned off for several hours.

Table 2 - 1

Trends in Groundwater Levels

Well Designation	Depth to Groundwater Surface (feet) 1994	Depth to Groundwater Surface (feet) 1997
Well No. 3	65-70	90
Well No. 4	80	95
Well No. 5	75	90

It is not known whether the changes over this three year period are part of any long-term trend. Past records for area wells indicate groundwater levels tend to fluctuate up and down considerably. The 1979 Sherwood Water Service Plan reported groundwater depths at Well No. 3 fluctuating between 20 and 65 feet below grade from 1963 to 1979 with no clear pattern of decline. It is apparent, however, that groundwater levels have dropped over the last 20 to 30 years. As a part of a separate phase in the Master Planning effort the City is having a geotechnical investigation of Well No. 3 completed.

The City's newest well, Well No. 6, was placed into service in August of 1997; however, it took several years to obtain the well permit from the State Water Resources Department. One of the permit conditions requires that the City end its sole reliance on groundwater and prepare a Water Management and Conservation Plan to address inefficiencies in water use. During the review process for the Well No. 6 application, the implementation of OAR 690-502 placed virtually all of Sherwood within either of two designated Groundwater Limited Areas: Chehalem Mountain and Sherwood-Dammasch-Wilsonville. These rules place strong restrictions on the development of new wells within the affected areas.

Surface Water. The City has recently constructed a permanent pipeline connection to the City of Tualatin's water system. This intertie, when placed into service, will provide an alternative water source for the City that will end its sole reliance on groundwater resources. Initially, the source of the water fed through this connection will be the City of Portland's Bull Run supply. But the connection also provides Sherwood with the potential to tap other surface water sources, including the Willamette and Clackamas Rivers. The City is currently entering into the planning process for developing a primary surface water supply with other water districts and municipalities in the area.

The agreement currently in place between Sherwood and Tualatin allows Sherwood to draw 175,000 gallons per day (gpd) through the Bull Run Connection on a continuous basis and up to 1.2 MGD for short-term emergencies. The connection was designed for a future maximum capacity of 12 MGD.

WATER SUPPLY AND TREATMENT

Well System. The four municipal wells currently in service are designated Wells No. 3 through No. 6, since Wells No. 1 and No. 2 have been previously abandoned. The four operating wells will continue to serve as the main water supply for the City even after the Bull Run Connection is placed into operation. The wells are turned on and off automatically as a group based on the water level in the reservoir. Well startup is staggered to reduce pressure surges by using different programmed delays for each well. Current operation allows the reservoir depth to drop by 4 feet between the time the wells are turned off and the time they are turned on. Standby emergency generators are installed at Wells No. 3 and No. 6 to provide power in case of an interruption in electrical service. These generators are turned on automatically upon power failure.

Design pumping capacities and other descriptive data for the wells are presented in Table 2-2 (following page).

Table 2 - 2

Descriptive Data For Production Wells

Well Parameter	Well No. 3	Well No. 4	Well No. 5	Well No. 6
Total Depth (ft)	319	400	800	880
Pump Bowl Setting (ft)	110	400	430	300
Well Casing:				
Depth (ft)	77	99	50	300
Diameter (in)	12	14	16	16
Capacity:				
Design production (gpm)	900	280	500	550
Permitted capacity (gpm)	900	375	675	550
Summer groundwater levels: (ft. below discharge)				
Static	90	95	90	137
Drawdown	103	250	385	179
Discharge pipe size (in)	8	8	8	8

The combined design capacity of the four wells is 2230 gallons per minute (gpm), or 3.2 million gallons per day (MGD). However, the actual production rate during the summer for the three older wells is less than the design rate. Table 2-3 compares the actual flow rates delivered by Wells No. 3, No. 4 and No. 5 during August of 1997 to the January 1998 flow rates. These data indicate that seasonal drawdown of the aquifers is having an adverse impact on the production capacities of these wells.

Table 2 - 3

Actual Capacities of Wells 3, 4 & 5

Well Identification	Well Production Rate (gpm)	
	August 1997	January 1998
Well No. 3	650	725
Well No. 4	180	280
Well No. 5	475	550

Well No. 6 has only recently been placed into service and no drop in capacity has been reported in that well. It has consistently produced water at the design flow rate of 550 gpm. Using the August 1997 production rates for Wells No. 3 through No. 5 listed above and the design rate for Well No. 6, a daily total of 2.67 MG can be produced if all wells are operated a full 24 hours. It is, however, preferable to operate the wells less than 24 hours per day to increase operating flexibility and reduce wear on equipment. A discussion of recommended well run times and capacities is included in Section Five.

Bull Run Connection. The Bull Run Connection basically consists of a four-mile long, 24-inch pipeline with a valve vault at each end. A plan of the pipeline is shown in Figure 2-3. Pressure reducing valves installed in parallel are located in the Sherwood vault near the downstream end. These valves are set so that water pressure is lowered from 150 to 170 pounds per square inch (psi) down to 80 to 90 psi as it enters the distribution system. The amount of flow through the Bull Run Connection is regulated by control valves located in the Tualatin vault at the pipeline's upstream end. The flow rate through the Connection is controlled by adjusting the degree to which these valves are opened.

An additional 12-inch intertie connecting the Sherwood and Tualatin distribution systems is available under Cipole Road. This intertie has been used as a backup in the past to serve Sherwood's customers and could be used again when the Bull Run Connection is out of service. A temporary 550 gpm pump and hose connections can be installed between fire hydrants to link the Tualatin and Sherwood systems. Water is drawn from a Tualatin water main that operates at about 50 psi and pumped into the Sherwood system at about 88 psi. This intertie can also be used in reverse to direct water from the Sherwood system to Tualatin's system. The difference in the operating pressures of the two systems allows gravity flow in this direction.

Treatment. The only treatment currently provided to the water supply by the City is the addition of sodium hypochlorite at each well for disinfection. Sodium hypochlorite solution is purchased in drums and stored at Well No. 3. The solution is taken to each wellhead in plastic containers where it is diluted with water before being metered into each well discharge line. The dilution ratio of water to hypochlorite solution varies between 7:1 and 11:1 depending on the season.

STORAGE

The 2 MG reservoir is a circular concrete tank that was constructed in 1972. The tank has a diameter of approximately 105 feet and a depth of 30 feet below the overflow. The tank floor sits at elevation 350, putting the overflow at elevation 380. Operating depths typically range from 25 to 29 feet. The tank water depth cannot be drawn down below about 3 feet due to the location of the outlet pipe. In the past, the tank level has been lowered to a depth of 10 feet; however, this occurred when the distribution system was smaller and demands were considerably lower than today.

The site of the reservoir, including new park properties, incorporates a total area of about 23 acres.

BOOSTER PUMPING STATION

A new booster pumping station was constructed at the existing reservoir site in 1997 to replace the old station located at the same site. The new structure houses three 50-horsepower (hp) pumps, each designed to handle 900 gpm at a total dynamic head of 140 feet. Space is also provided for a fourth 900-gpm pump. The total design capacity of 3600 gpm was intended to satisfy the projected peak demands plus fire flow requirements with one pump out of service. A standby generator is installed at the new station to supply electricity to the pumps in the event a power outage.

Under existing conditions, one pump operates to meet the base demand for the Pressure Zone with an additional pump being activated if the demand exceeds the capacity of the lead pump. Typically, one 50-hp pump operates at a small fraction of its design capacity to meet current Pressure Zone demands.

DISTRIBUTION SYSTEM

General. The City's water distribution system consists primarily of two service zones. These include the Gravity Zone, which encompasses most of the City, and the Pressure Zone, which is fed by the booster station and covers the City's southeast corner. The Gravity Zone operates off the reservoir level at a hydraulic grade line (HGL) of 375 to 379 feet above sea level. The new booster station maintains an HGL of about 525 to 535 feet in the Pressure Zone with a 50-hp pump in operation.

During development of the distribution system network analysis, 440 housing units were identified as being tributary to the booster station in 1997. This includes the intermediate zones. The total number of housing units served by the water system in 1997 was approximately 3000.

In general, the distribution system is fairly well looped to maintain reliable service; however, fragmented development on the northwest side of State Route 99W has produced unlooped sections in that part of the Gravity Zone.

System Leakage. Most of the system is relatively new and the leakage rate is low. A reconciliation of water produced and water consumed in 1995 was performed based on the well production records summarized in Table 4-3 (Page 4-4) and City billing records. The reconciliation indicated that only 6.5 percent of the water produced was not accounted for through billing. This 6.5 percent included water taken by construction contractors from hydrants, in addition to water lost due to leakage. The leakage rate should be dropping as more new lines are added to the system through ongoing development and older lines are repaired or replaced.

Intermediate Zones. Given the amount of area served by the Gravity Zone and variations in ground level, it is difficult to maintain operating pressures within a desirable range of 50 to 80 psi throughout the zone. Pockets of low pressure have been a problem in the past and will become a greater problem as the City grows. Three sections have been isolated from the Gravity Zone in the past due to inadequate service pressures and are now served from the Pressure Zone through pressure reducing valves. These intermediate zones are located along Oriole Court, along Bowmen Court, and in the area of Norton and Forest Avenues (see Figure 2-1). Two other areas that were experiencing low pressures have also been modified in 1998 to allow service to be provided from the Pressure Zone rather than the Gravity Zone. These areas include:

- Orchard Heights Court, and
- Mansfield and Smock Streets and William Avenue.

A connection has been installed from the intersection of Division and Pine Streets to serve homes along Orchard Heights Court. Another connection, this one with a pressure reducing valve, has been added between the new line in Murdock Road and the in William Avenue to serve the second area.

Discussions on other areas experiencing inadequate system pressures are included in Sections Three and Six.

System Inventory. Table 2-4 presents a system inventory of the approximate lengths of each pipe size in service during 1997. Given the rapid growth of Sherwood, these numbers have changed during the preparation of this report.

Table 2 - 4

Existing Distribution System
Pipe Inventory

Gravity Service Zone		Pressure Service Zone	
Pipe Size (in.)	Length (ft.)	Pipe Size (in.)	Length (ft.)
2"	3,950	8"	19,800
4"	1,550	10"	3,000
6"	14,400	16"	1,350
8"	109,500		
10"	31,700		
12"	43,800		
14"	4,000		
16"	550		

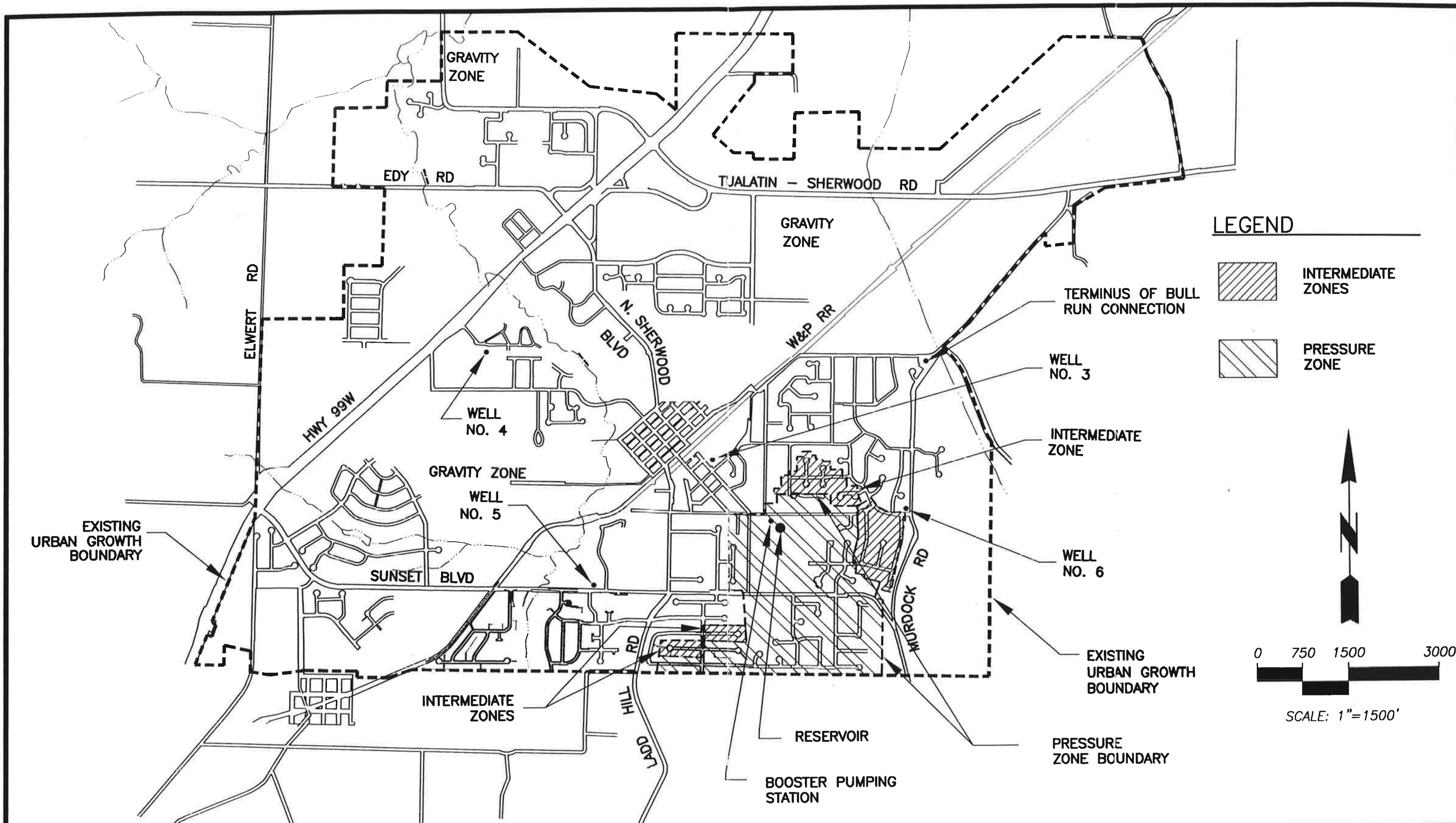
Current City standards require new water piping to be installed in size increments of 4 inches with a minimum allowable pipe size of 8 inches. Pipes with 6-inch diameter are acceptable for short extensions to service connections or fire hydrants.

SYSTEM CONTROL

The water system is controlled by computer at the Public Works Building on 540 N.W. Washington Street. The operations of the wells, the booster station and the reservoir are monitored automatically and reported to the computer by telemetry. Operators can control on/off status and adjust setpoints at the computer terminals. The Bull Run Connection will also be monitored and controlled in this manner when it is placed into service. An automatic dialer is provided to allow the telemetry system to contact operating personnel when an alarm is activated. Similarly, operators can use the dialer system to contact the computer from remote locations and check the operating status of the facilities.

The following is a list of the system operating parameters for which automatic monitoring and remote reporting capabilities are provided or planned.

1. Production Wells:
    - well pump status (on/off)
    - emergency generator status at Wells 3 & 6 (on/off)
    - pump run time
    - water production (pumping) rate and cumulative water pumped
    - groundwater level
2. Booster Station:
    - status of each pump(on/off)
    - emergency generator status (on/off)
    - pump run time
    - discharge rate and pressure
3. Reservoir:
    - operating level
4. Bull Run Connection:
    - Sherwood Vault - pressure-reducing valve upstream pressure
      pressure-reducing valve downstream pressure
      valve pressure setting
      current and total flow rate
    - Tualatin Vault - flow control valve setting (percent open)
      control valve upstream pressure
      control valve downstream pressure
      current flow rate each valve and total flow



CITY OF SHERWOOD

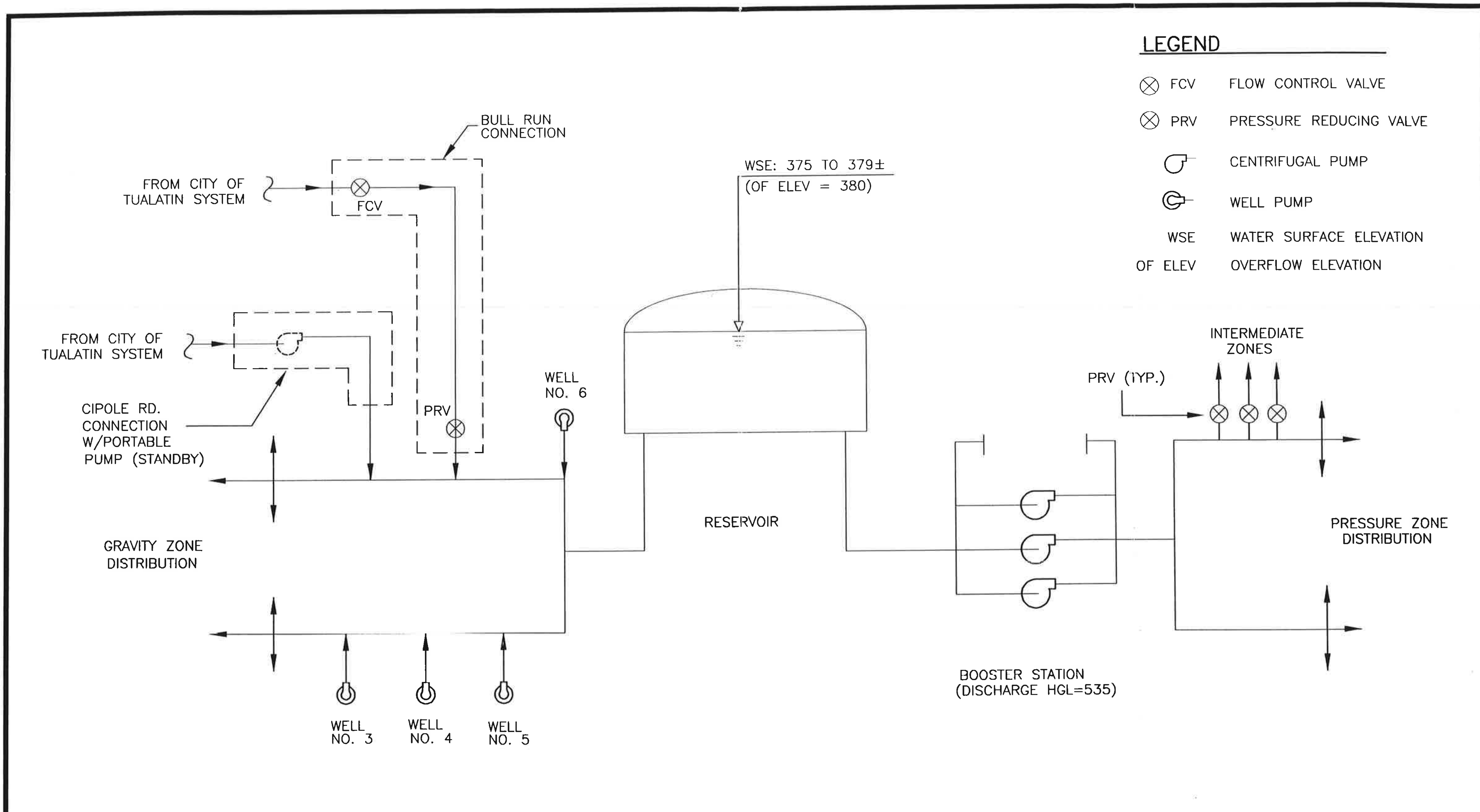
BOOKMAN-EDMONSTON ENGINEERING, INC.
BEAVERTON, OREGON

WATER SYSTEM MASTER PLAN UPDATE

WATER SYSTEM COMPONENTS & SERVICE ZONE BOUNDARIES

FIGURE 2-1

DATE 3/26/99



CITY OF SHERWOOD	WATER MANAGEMENT AND CONSERVATION PLAN	FIGURE 2-2
BOOKMAN-EDMONSTON ENGINEERING, INC. BEAVERTON, OREGON	SCHEMATIC OF EXISTING SYSTEM	DATE 9/17/98

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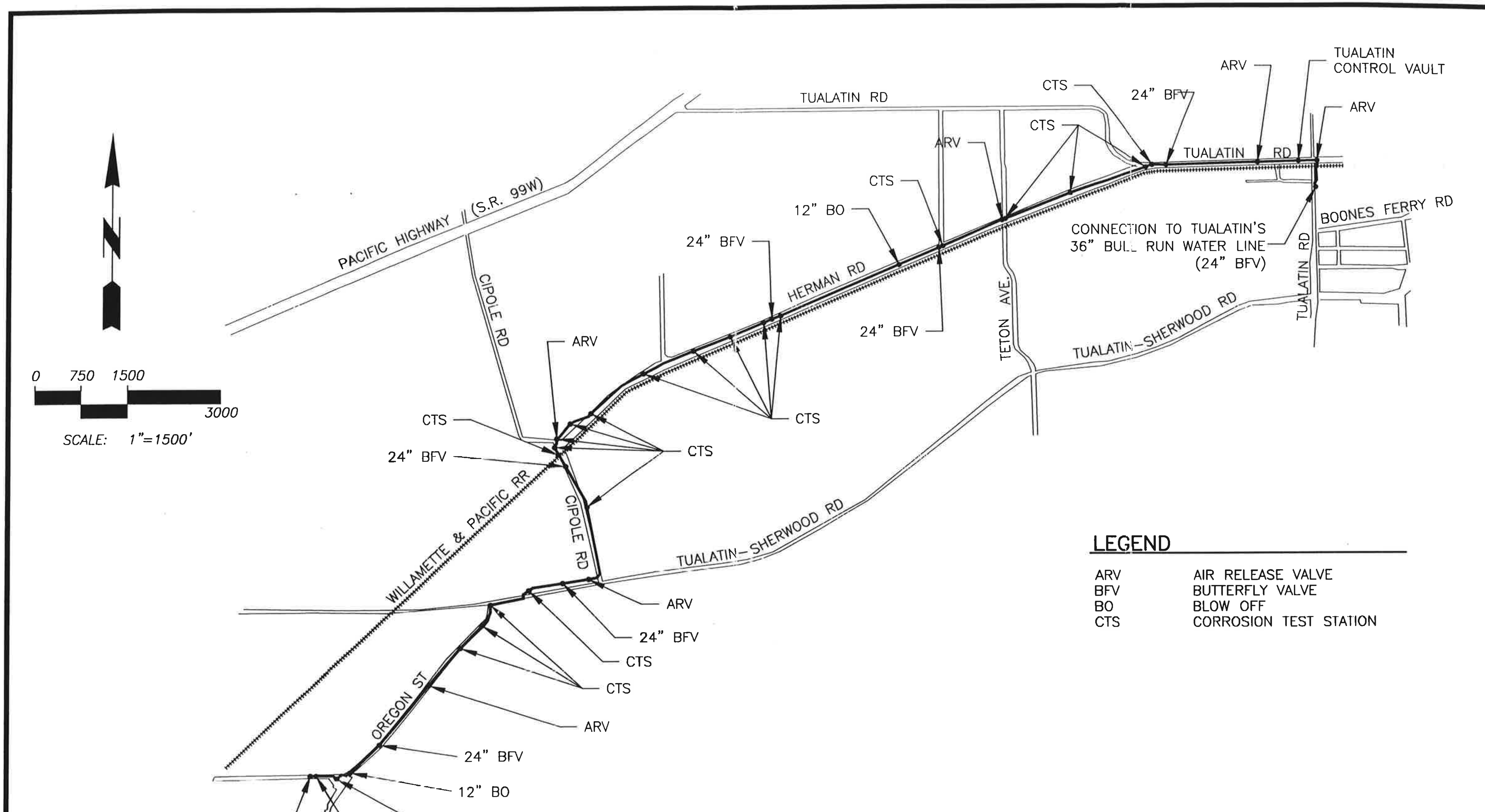


CITY OF SHERWOOD	WATER SYSTEM MASTER PLAN UPDATE	FIGURE 2-3
BOOKMAN-EDMONSTON ENGINEERING, INC. BEAVERTON, OREGON	BULL RUN CONNECTION	DATE 1/11/99

R:\..0590020\Figure 2-3.DWG

CITY OF SHERWOOD
WATER SYSTEM MASTER PLAN UPDATE

SECTION THREE

EXISTING WATER SYSTEM DEFICIENCIES

GENERAL

Deficiencies in the existing water system have been identified from discussions with City personnel. Field surveys of existing equipment by Bookman-Edmonston staff have not been included in the scope of this study.

PRODUCTION WELLS

The Public Works Department did not report any specific problems with the operating condition of the existing well facilities. The data presented in Tables 2-2 and 2-3 (Page 2-4) do, however, indicate the following two general deficiencies:

- The groundwater level in Well No. 3 during drawdown is very close to the bowl setting; and
- The production capacities of Wells No. 3 and No. 4 during the high-demand summer months are substantially less than the reported design capacities.

As a separate phase in the Master Planning effort, the City will evaluate well production capacity and the feasibility of upgrading the wells to restore their capacity. The City is currently having a geotechnical investigation of Well No. 3 completed as part of that separate planning phase. Consequently, these issues will not be further addressed in this report.

The only other well deficiencies that were identified are related to the system for recording the operating data that is reported to the computer. However, during the preparation of this report, the computer software has been upgraded to correct these problems. Formerly, the measured production rates for the wells were totalized automatically based on pump run time; but the monthly total had to be recorded manually. Also, the production totals for each well had to be summed manually to obtain the combined production for all the wells. The upgrade allows the computer to automatically calculate, totalize, and record the daily and monthly well production totals.

TREATMENT

Elevated levels of iron and manganese in the discharge from Well No. 6 have resulted in customer complaints. To alleviate this problem the installation of treatment equipment is being considered. An evaluation of alternative treatment methods is being conducted by the City separately from the scope of this report.

BOOSTER STATION

The equipment in the booster station is new and in good condition. However, one drawback to the system operation has been that demands typically are only a small fraction of the design capacity of one 50-hp pump. This results from the pumping station being designed to deliver fire demands in addition to projected peak domestic demands. The design capacity of each new pump is 900 gpm, whereas the average demands in the pressure zone are currently less than 80 gpm. Since the pumps are constant speed units, they operate inefficiently while delivering the relatively small flows most of the time.

The Public Works Department is currently working to install a smaller pump in the new station as the fourth pump. This smaller pump could then be used to satisfy demands under most conditions with a lower power draw. The City may also wish to evaluate the installation of variable speed drive controls in the future for at least one 50-hp pump.

Another shortcoming had been the lack of automatic switchover capabililites so that a back-up pump would start if the operating pump failed. This feature has, however, been added to the system controls this year to increase reliability.

RESERVOIR

The only reservoir operating deficiency mentioned by Public Works is the lack of any level gauge on the tank to allow operating staff to verify the water level visually while at the tank site. This capability can be added to the new reservoir that will need to be constructed as discussed in Section Five.

No structural analysis of the reservoir has been completed as part of this study. Given the critical nature of this facility and the fact that the tank is over 25 years old, it is recommended such an analysis be performed within the next 5 to 10 years.

DISTRIBUTION SYSTEM

Undersized Piping. The City has developed standards for minimum pipe sizes in the distribution system. To bring the distribution system up to the current standards, most pipe under 8 inches in diameter would need to be replaced with new pipe having a minimum diameter of 8 inches. Table 2-4 (Page 2-7) lists approximately 20,000 lineal feet (l. f.) of pipe 6-inch and smaller in the system; however, not all of the lines are considered undersized. Six-inch pipe sections located in cul-de-sacs or that are extensions serving a single connection or fire hydrant may be determined to be acceptable by the City on a case-by-case basis..

Table 3-1 identifies the pipe sections 6 inches and smaller that need to be replaced to meet the City's minimum pipe size standards. The lengths listed for the pipe sections are based on measurements taken from the water system map provided by the City. The 3-inch supply line from Well No. 3 is not listed, but also needs to be increased in size.

Table 3 - 1

Undersized Pipe Sections

Size	Location	Length
2"	Old Town (alley between First St. and Second St.)	300 l. f.
	Meinecke Rd. and Pacific Hwy.	2,450 l. f.
	Tualatin St.	400 l. f.
	Clifford Ct.	250 l. f.
	April Ct.	180 l. f.
	June Ct. (Meadow)	250 l. f.
4"	10th St., 11th Ct., Glencoe Ct. and N. Sherwood Blvd.	1,550 l. f.
6"	Old Town (Main St., Second St., and alley between First St. and Second St.)	1,050 l. f.
	Oregon St. and Lincoln St.	2,400 l. f.
	Gleneagle Dr. and 12th St.	3,250 l. f.
	Roy St.	1,450 l. f.
	Cochran Dr. and May Ct.	1,300 l. f.
	Norton Ave.	840 l. f.
	Sunset Ct.	550 l. f.
	Lee Dr.	580 l. f.
	Restwood	230 l. f.

Figure 3-1 identifies the locations of the pipe sections listed in Table 3-1, except for Sunset Court which is south of the area shown in the figure. Based on the breakdown in Table 3-1, the total length of undersized pipe is approximately 17,000 l. f.

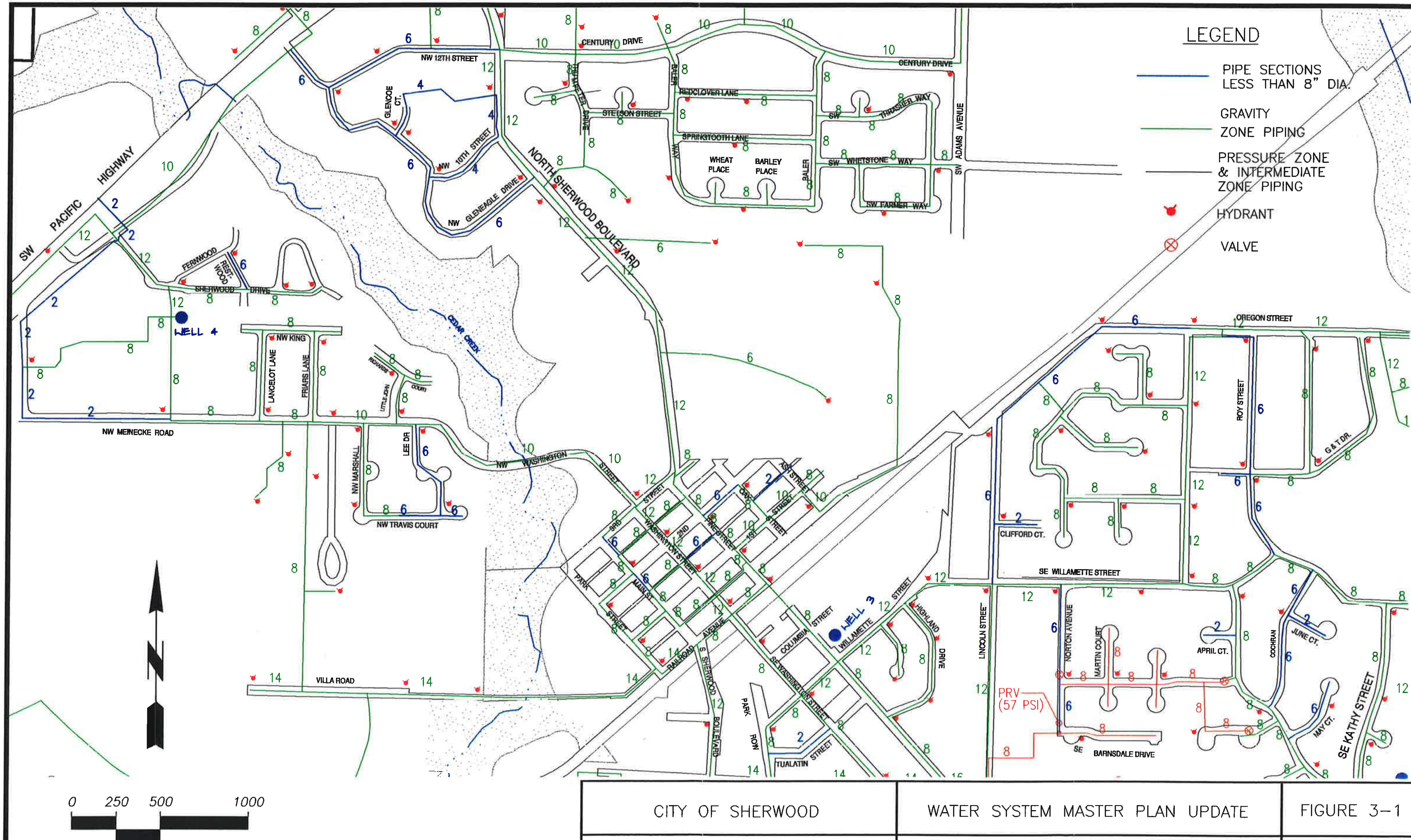
The City plans to replace the 2,400 l. f. of 6-inch line in Oregon and Lincoln Streets with a 12-inch pipe rather than an 8-inch pipe. That will further upgrade system capacity between the reservoir and the Bull Run Connection.

Other upgrades in distribution system capacity previously considered by the City include the following pipe replacements:

1. Install 12-inch piping to replace the 8-inch water mains under Pine Street from Willamette Street to Columbia Street and under Washington Street from Columbia Street to Railroad Avenue. Also install a 12-inch pipe under Columbia to connect the two segments described above. The work consists of approximately 900 lineal feet of pipe, including a bore under the railroad tracks.
2. Install approximately 2,000 lineal feet of 16-inch water main in the pressure zone under Pine Street and Sunset Boulevard from Division Street to Alder Grove Avenue to replace with an 8-inch pipe.

System Operating Pressures. Problems with inadequate service pressures in some areas of the Gravity Zone have resulted in numerous customer complaints. The City has instituted improvements in five areas that are adjacent to the Pressure Zone as discussed in Section Two. One other area experiencing low operating pressure that must still be addressed is in the southwest corner of town. Houses south of Sunset Boulevard and west of Middleton Road are at an elevation such that system pressures below 40 psi can occur there. Thus operating pressures are significantly below the preferred range of 50 to 80 psi. Other connections just east of Middleton Road also are reported to experience low pressures. The City plans to address this problem by constructing a booster station to serve this area. The approximate service area of the booster station is discussed under the heading "Network Model Results" in Section Six. Further discussions on system operating pressures are also presented in Section Six.

Miscellaneous. In addition to the above issues relating to the distribution system, the City is planning to establish a program for the ongoing inspection and replacement of older water meters. This will help maintain the accuracy of water consumption readings and reduce the potential for leakage. A program is also being planned for the periodic flushing of fire hydrants.



CITY OF SHERWOOD	WATER SYSTEM MASTER PLAN UPDATE	FIGURE 3–1
BOOKMAN–EDMONSTON ENGINEERING, INC. BEAVERTON, OREGON	UNDERSIZED PIPELINES	DATE 10/02/98

R:\..0590020\Figure 3–1.DWG

CITY OF SHERWOOD
WATER SYSTEM MASTER PLAN UPDATE

SECTION FOUR

POPULATION AND WATER DEMANDS

POPULATION

Past Population and Housing Trends. The City of Sherwood has been experiencing rapid growth in recent years and this trend is expected to continue in the near future. Table 4-1 presents population figures obtained from the City Planning Department for the recent past.

Table 4 - 1

Historical Population Figures

Year	Population	Annual Pct. Growth
1990	3,125	----
1994	4,615	10.2% *
1995	5,320	15.3%
1996	6,900	29.7%
1997	8,625	25.0%
1998	9,600	11.3%

* Average annual growth over four-year period.

A review of housing data provided by the City Planning Department indicates there were about 2,990 dwelling units in the City that had been built or for which a building permit had been issued through November 1996. Given the brief lag time between the issuance of building permits and the construction of the permitted housing units in Sherwood, this number is probably close to the number of units occupied during the summer of 1997. Using the estimated July 1997 population of 8,625, this translates into an average household occupancy of 2.88 people per dwelling unit (DU). By comparison, Metro's *Urban Growth Report* dated December 18, 1997 indicates that in 1994 Sherwood had 4,615 people residing in 1,580 DUs. This is equal to an average household occupancy of 2.92 people per DU. Based on this data, the current average household occupancy should be close to 2.9.

Population Projections for Current City Limits. The December 1997 *Urban Growth Report* prepared by Metro projected a population of 18,566 for Sherwood in the Year 2017. The same report also projects that Sherwood will contain 7,002 dwelling units by the Year 2017. These projections were prepared based on the current City boundaries; therefore, they do not include an allowance for the planned Urban Reserve.

The population and housing projections equate to an average per household occupancy of 2.65. The decline in per household occupancy from 2.9 to 2.65 is consistent with Metro's overall regional projections.

Population Projections for Urban Growth Reserve. The establishment of a 460-acre Urban Growth Reserve has been planned to the south of the current City limits to accommodate additional development. Metro will decide in 1999 whether to approve a shift in the Urban Growth Boundary to include this land. If this expansion is approved and the City annexes the area, the construction of new housing units could begin in the Reserve by the Year 2000.

Metro has estimated that the Urban Reserve could be developed to include a total of 2,067 housing units. The December 1997 *Urban Growth Report* includes a preliminary projection of 685 housing units in the Reserve by 2017. Since an Urban Reserve Study has not yet been prepared to provide initial planning data, this report will assume that all 2,067 housing units will be constructed by 2017. This conservative estimate is based on the premise that the Reserve would be incorporated into the Urban Growth Area to meet projected 20-year land requirements. At a future per household occupancy of 2.65, this translates to an additional population of 5,480. These housing and population figures are preliminary in nature and should be reevaluated once the Urban Reserve Study is completed.

Combined population projections. Using the figures presented in the previous paragraphs, the combined population for the expanded City limits is projected to be about 24,050 in 2017. In reality, Metro will reevaluate their population projections for the Sherwood area once the Urban Growth Area has been expanded. However, this combined total represents a conservative estimate based on information currently available.

The increase in population from 9,600 in 1998 to 24,050 in 2017 works out to an average annual growth rate of 4.95% over 19 years. This is significantly lower than the annual growth rates experienced during the 1990s. Since no projections are available from Metro for intervening years, the City Planning Department has proposed that a range of intermediate population projections be developed using the following assumptions.

- A minimum of 200 and a maximum of 500 housing units will be added per year through 2002.
- The average household occupancy will remain at 2.9 through 2002.
- A constant growth rate will occur for the following 15-year period to reach Metro's projection for 2017.
- The average per household occupancy will decline at a constant rate from 2.9 to 2.65 from 2002 to 2017.

The two annual housing construction figures for the next four years represent maximum and minimum near-term growth rates that can be correlated to a most favorable and a least favorable development climate. A tendency toward one rate or the other could be effected by economic conditions or the City's growth management practices. Since the 20-year projection is set, the growth rates for the period from 2002 to 2017 have been calculated to offset the assumed near-term trend. Therefore, the high near-term growth rate would be followed by a lower annual growth rate and the low near-term growth rate would be followed by a higher annual growth rate.

Table 4-2 lists the two population projections developed from the method described above. Figure 4-1 presents these projections graphically.

Table 4 - 2

Population Projections for City

Year	High Near-Term Growth Trend	Low Near-Term Growth Trend	Percent Difference *
1999	11,050	10,180	8.6%
2000	12,500	10,760	16.2%
2001	13,950	11,340	23.0%
2002	15,400	11,920	29.2%
2007	17,870	15,060	18.6%
2012	20,730	19,030	8.9%
2017	24,050	24,050	0.0%

* Low near-term growth trend used as base population for percent difference.

WATER DEMANDS

Historical Water Demands. To update estimates of per capita water use, monthly water production records were obtained from the Public Works Department for the years of 1994, 1995, 1996 and 1997. Total annual production amounts for these years are listed in Table 4-3 (following page) along with the equivalent figures for total and per capita average daily production. The amount listed for 1996 is approximate since estimated production values were used for three months of that year.

The figures listed in Table 4-3 suggest a significant decline in per capita water use over the past four years. This trend is also supported by the fact that previous Water System Master Plans used an average per capita water demand of 160 gpd per capita.

Table 4 - 3

Recent Water Production Records

Year	Total Water Production (gallons)	Avg. Daily Production (gallons/day)	Avg. Per Capita Production (gallons/day)
1994	242,306,500	663,850	144
1995	254,668,400	697,720	131
1996	270,000,000	737,700	107
1997	326,491,330	894,500	104
4-Year Avg.	273,366,558	748,443	122

There are several reasons why the sharp drop-off in per capita water use illustrated in Table 4-3 may not be representative of an actual long-term pattern. These include the following:

- the City instituted mandatory water use restrictions for major landscape irrigation users during the summer of 1996 when Well No. 5 was lost from service;
- the unusually heavy rainfall that occurred from late 1995 through October 1997 probably reduced water demands, particularly for irrigation;
- the rapid rate of development may have reduced the accuracy of population estimates and also may have resulted in estimates that do not reflect average population figures for a given year; and
- commercial/industrial development in the City has been lagging behind residential development, but is projected to catch up in the future.

Some of the reduction in average per capita water use may be part of a permanent shift due to voluntary limitations the City has instituted for irrigation. Additionally, revisions to the Plumbing Code that require low-flow plumbing fixtures for new homes may be reducing indoor demands. There also may be a trend toward smaller lots which can reduce per capita demands for irrigation.

Since the trend towards lower per capita consumption could be temporary to some extent, it is recommended that a total average per capita demand of 125 gpd be used for planning purposes. This figure is slightly above the overall average per capita water production from 1994 through 1997.

Since this per capita figure is based on total water production, it includes an allowance for distribution system losses through leakage. As indicated in Section Two, the City last performed a reconciliation between water use and water production in 1995 and the system-wide losses were determined to be less than 6.5%. It can be expected that this leakage rate has declined over the past few years as significant lengths of new pipe have been installed during development while the City has continued ongoing repair and replacement efforts. Thus, losses of 5% or less can be assumed for current and future conditions.

This year the City has added the capability to record daily water production, but in the past the City has keep production and water use records only on a monthly basis. Therefore, peak daily demands must be estimated using an assumed peaking factor applied to annual average water production. In the past, when the City has requested curtailment of water use for major landscape irrigation, it was found that this type of usage made up about 33 percent of the demand during hot, dry weather. This heavy use for landscape irrigation would tend to result in high peak daily demands. However, the City has taken measures to reduce peak demands for irrigation by instituting a staggered system for major users. Additionally, the ratio of peak demands to average demands typically declines as the number of system users continues to rise. If conservation measures aimed at outdoor water uses are instituted, they can also reduce peak irrigation demands.

It is recommended that total peak daily demand projections be estimated using a peaking factor of 2.75. This value should be reasonably conservative given the size of Sherwood's system and the relatively high outdoor water use. Applying the peaking factor results in an estimated peak daily use of about 350 gpd per capita, including system losses.

For the network analyses performed on the distribution system a short-term peak 6-hour demand was assumed that is 20 percent above the peak daily rate. This additional factor was included to account for the variation in demand that typically occurs on a daily basis in municipal water systems. The resulting peak per capita demand is about 420 gpd.

Water Demand Projections. A range of projected water demands has been calculated through 2017 using the estimated per capita water requirements identified above and the two alternative population growth trends presented in Table 4-2 (Page 4-3). These water demand projections are presented below in Table 4-4 (following page) and shown graphically in Figure 4-2. The short-term peak 6-hour demand is listed under the high near-term growth alternative only since these more conservative figures were used in the network analysis.

An additional line has also been included in Table 4-4 identifying the projected base 2017 demands excluding the Urban Reserve. This information has been added because of uncertainty regarding development of this area. The base 2017 demand illustrates the incremental increase in demands associated with the development of the Reserve.

Table 4 - 4

Projected Water Production Requirements

	High Near-Term Growth Trend			Low Near-Term Growth Trend	
Year	Avg. Daily Demand (MGD)	Peak Daily Demand (MGD)	Peak 6-Hr. Demand (MGD)	Avg. Daily Demand (MGD)	Peak Daily Demand (MGD)
1999	1.38	3.87	4.64	1.27	3.56
2000	1.56	4.38	5.25	1.35	3.77
2001	1.74	4.88	5.86	1.42	3.97
2002	1.93	5.39	6.47	1.49	4.17
2007	2.23	6.25	7.50	1.88	5.27
2012	2.59	7.26	8.71	2.38	6.66
2017	3.01	8.42	10.10	**	**
2017 base *	2.32	6.50	7.80	**	**

* Excludes demands for Urban Reserve.
** Same as high near-term growth alternative.

Breakdown of Existing Water Use. The total per capita water production described above includes water supplied for nonresidential, as well as residential uses. The nonresidential component includes demands for commercial, industrial, institutional and major landscape irrigation uses.

The City currently does not have the capabilities to analyze water use and estimate the relative proportions based on type of development. Additionally, the specific nature of future nonresidential development has not been identified in planning data provided by the City. Therefore, only broad assumptions can be made at this time regarding the breakdown of water demands by type of use. The following current breakdown of water uses has been assumed to distribute demands throughout the system for the network analysis:

- residential (indoor & outdoor) - 75 percent
- major landscape irrigation - 17 percent
- commercial/industrial/institutional - 8 percent

Major landscape irrigation includes water used for outdoor purposes at the schools, large commercial and industrial developments, and major homeowners associations. This category does not include irrigation water used at individual residences, individual commercial and industrial properties and most apartment complexes, since these demands are included in the other categories listed above.

In the past, when the City instituted emergency water use curtailment measures, close to a one-third reduction in peak demand was achieved by targeting large-scale irrigation users. However, it is probable that the proportion of demands that can be attributed to major irrigation has declined in the past few years as the residential population has risen sharply. Major outdoor water users such as the schools and existing developments have not been increasing their consumption for irrigation, while residential water use has increased. Additionally, efforts have been made to develop a staggered schedule for irrigation practices by major users. Because of these factors, a value of 17 percent was used for the proportion of total demand resulting from major outdoor irrigation. Additionally, preliminary computer modeling has indicated that applying a higher percentage of the demands to individual residences results in a more conservative network analysis.

The low percentage for commercial, industrial and institutional water use was chosen based on the following criteria:

- Planning data from Metro indicate that non-farm employment has not been growing in proportion to population growth in Sherwood.
- Many of the existing commercial developments in Sherwood, such as retail shopping outlets, result in low to moderate indoor water use.
- The City has not identified any industrial customers that have significant demands for plant process water.
- The fall sessions at the local schools would typically begin after periods of peak summer demands.

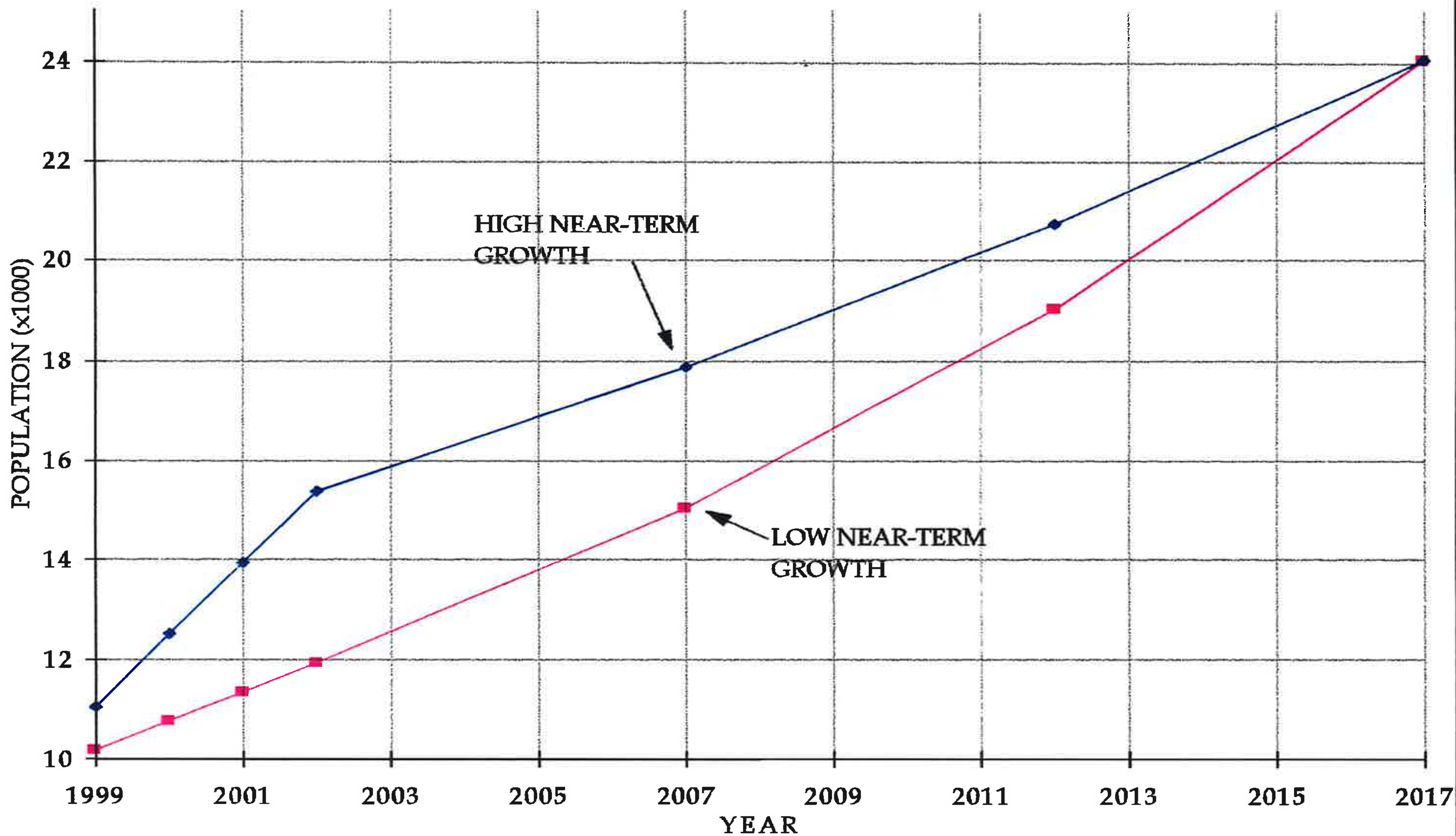
Also, as stated above, applying more of the demands to individual residences results in a more conservative network analysis.

Breakdown of Projected Water Use. The breakdown of water use listed above was assumed to be applicable through 2002. This is because population growth is projected to remain relatively high in the near term. However, the proportion of demands from commercial, industrial and institutional customers is expected to increase in the future. Data provided by the City indicates that the ratio of nonfarm employment to total population was about 0.27 in 1998 (2,600 non-farm employees relative to a population of 9,600). Projections from Metro indicate that this ratio is expected to increase to 0.49 by 2017 (11,850 non-farm employees relative to a population of 24,050). Indoor residential water use may continue to drop to some extent as low water use plumbing fixtures are installed in a higher percentage of residences. The above information suggests that the proportion of total water production associated with commercial, industrial and institutional uses could nearly double by 2017. However, actual growth in nonresidential demands will depend on the specific nature of future development.

For the purposes of this report the following projected breakdown of demands by type of use has been used to complete a network analysis of the system in 2017:

- residential (indoor & outdoor) - 72 percent
- commercial/industrial/institutional - 16 percent
- major landscape irrigation - 12 percent

It has been assumed industrial development will not include any users that require large amounts of process water. The City should review the infrastructure needs for such heavy industrial developments on a case specific basis.



CITY OF SHERWOOD	WATER SYSTEM MASTER PLAN UPDATE	FIG. 4–1
BOOKMAN–EDMONSTON ENGINEERING, INC. BEAVERTON, OREGON	POPULATION PROJECTIONS	DATE 12/31/98

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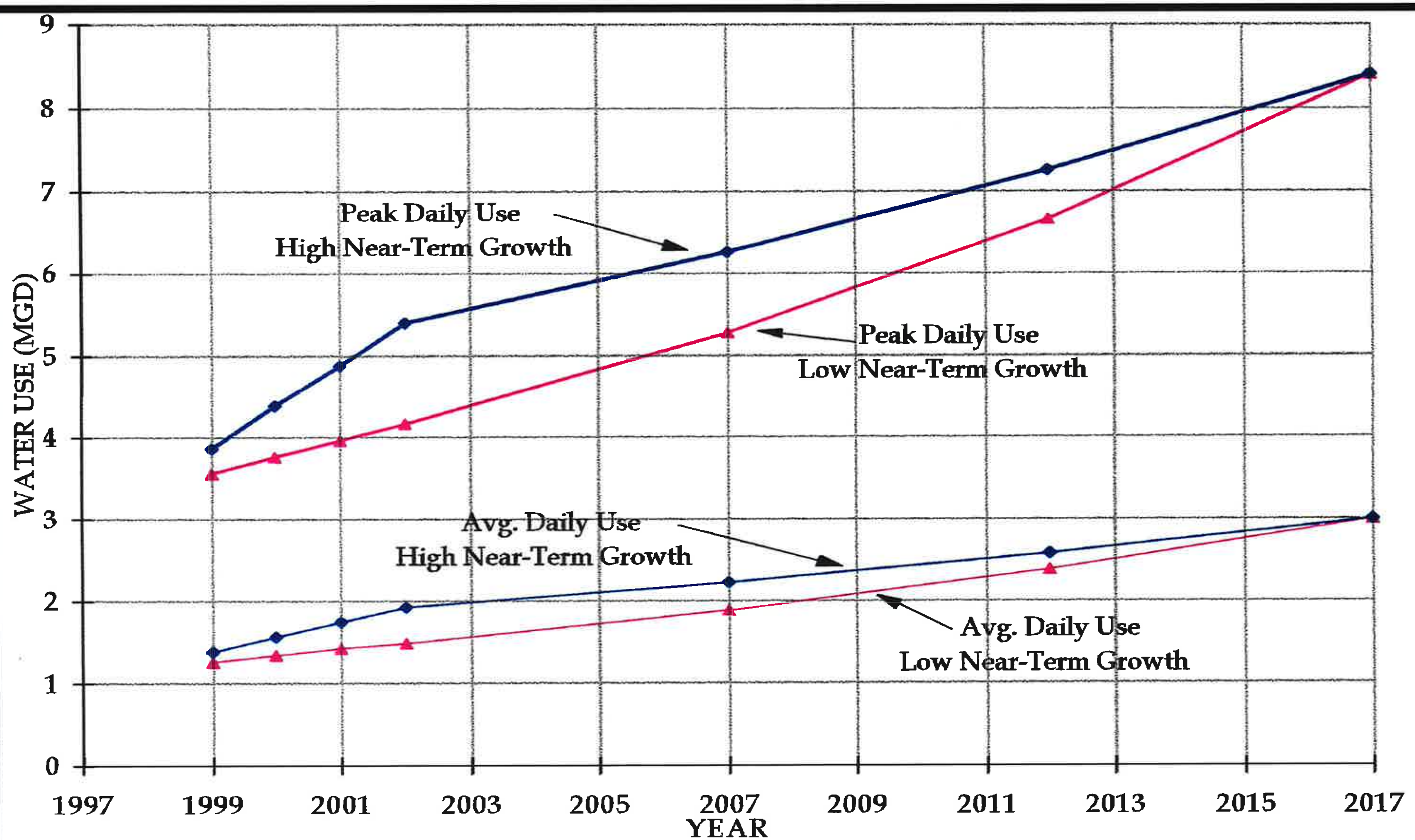
p. 4-9



CITY OF SHERWOOD	WATER SYSTEM MASTER PLAN UPDATE	FIG. 4–2
BOOKMAN–EDMONSTON ENGINEERING, INC. BEAVERTON, OREGON	WATER USE PROJECTIONS	DATE 12/31/98

p. 4-10

R:\..0590020\Figure 4–2.DWG

CITY OF SHERWOOD
WATER SYSTEM MASTER PLAN UPDATE

SECTION FIVE

WATER SUPPLY AND STORAGE CAPACITIES

WATER SUPPLY CAPACITY

General. The City is having the existing well system and alternative sources of supply evaluated in a separate phase of the master planning process. Therefore, this report provides only a projection of supply capacity deficits based on the total production rate of the existing well system during the summer of 1997.

Typically, supply capacity is provided to meet the design peak daily demand. Part of the reservoir volume is then used to meet short-term spikes in demand that last for a matter of hours during the peak day. The reservoir level is drawn down during high-demand periods and refills during low-demand periods. The following estimate of supply capacity deficits is based on projections for peak daily demands.

Existing Well Capacity. It is recommended that the average and peak daily capacities of the production wells be based on operating times of 8 and 20 hours per day, respectively. Limiting the hours of operation to these levels can increase system reliability and operating flexibility. The down-time reduces wear on the equipment, provides opportunities for preventative maintenance, and limits the demand on groundwater resources.

Calculating the system capacity at these operating times also builds some backup capability into the system. Higher operating frequencies for the wells would reduce the margin of safety for these critical facilities. The wells should only be operated for more than 20 hours in one day to handle short-term emergencies.

Table 5-1 lists the total capacities of the well supply system using the above listed operating frequencies and the August, 1997 pumping rates. This assumes all four wells are in service.

Table 5 - 1
Total Well Supply Capacity

Total Pumping Rate (gpm)	Total Supply Capacity of Wells (MGD)		
	Average (8 hrs./day)	Peak Day (20 hrs./day)	Emergency (24 hrs./day)
1855	0.89	2.23	2.67

Water Supply Deficits. The supply capacity deficits have been calculated from the data presented in Table 4-4 (Page 4-6) and Table 5-1. Table 5-2 identifies the supply capacity deficits based on the alternative peak daily demands and the peak-day well capacities. Figure 5-1 also presents a graph that illustrates these deficits. The last line in Table 5-2 identifies the projected capacity deficits in 2017 that would result if the City limits are not expanded to include the Urban Growth Reserve.

Table 5 - 2

Projected Water Supply Deficits for Peak Day in MGD

Year	High Near-Term Growth Trend	Low Near-Term Growth Trend
1999	1.64	1.34
2000	2.15	1.54
2001	2.66	1.74
2002	3.16	1.95
2007	4.03	3.05
2012	5.03	4.44
2017	6.19	**
2017 base *	4.27	**

* Based on projected population of 18,570 without expansion of Urban Growth Boundary.
** Same as high near-term growth alternative.

Water Conservation. The City is required by the State to complete a Water Conservation and Management Plan; but even without this requirement, it is important that the City develop and begin implementation of a water conservation program. The shortfall in supply capacity summarized in Table 5-2 can be reduced by water conservation efforts. Therefore, conservation could serve as an important tool in the City's efforts to meet water supply needs. Water conservation represents a demand-side resource and the main objective of the plan is to aid the City in the cost-effective development of this resource.

Recommendations. The separate master planning phase that will evaluate sources of supply needs to address the following issues.

- **Well Capacity** - Investigations must be conducted to estimate long-term well production capacity and identify recommended modifications to the wells.

- **Conservation** - It is recommended the City use the recommendations of the Water Management and Conservation Plan once it has been finalized to implement a water conservation program. Development of this alternative resource could save money on long-term improvements.
- **Bull Run Connection** - Discussions should be held with the Cities of Tualatin and Portland regarding an increase in the agreed upon supply rate from this source.

WATER STORAGE CAPACITY

General. Communities can use a variety of criteria to identify the amount of storage capacity that should be provided for a water system. The basis for any given application can depend on the nature of the supply system and the preferences of the water providers. A common practice is to size water storage tanks to provide enough volume to meet peak daily demand plus fire flow requirements. This criterion would allow an interruption in water supply for a period of less than a day at any time of the year without completely drawing down the reservoir(s). Similarly, a partial interruption in water supply production or delivery could be accommodated for a period of days while still maintaining some storage volume for continued service. A surplus allowance of 10 percent is also commonly included in the design reservoir capacity to provide a cushion. This is based on the assumption that only 90 percent of the reservoir capacity can serve as effective storage volume in emergencies. The recommended reservoir volumes calculated for this report are based on peak daily demands plus an estimated volume for fire flow. The 10 percent surplus has also been included in the calculations to be conservative.

Recommended Storage Volumes. Table 5-3 (following page) presents the recommended net storage expansion capacities determined from the criteria described in the previous subsection. The volumes have been calculated for both population growth alternatives using the peak daily demands listed in Table 4-4 (Page 4-6). The effective storage capacity of the existing 2.0 MG reservoir has been subtracted from the total required volumes to identify the net expansion capacities listed. A preliminary analysis has estimated this existing effective capacity to be about 1.5 MG or 75 percent of the total volume.

In Sherwood, the estimated fire flow requirements for the schools represent the greatest demand. The basis for this condition is a fire flow of 3500 gpm over a 3.5 hour duration, which translates to a volume of 0.735 MG.

Table 5 - 3
Net Storage Volume Expansion Requirements

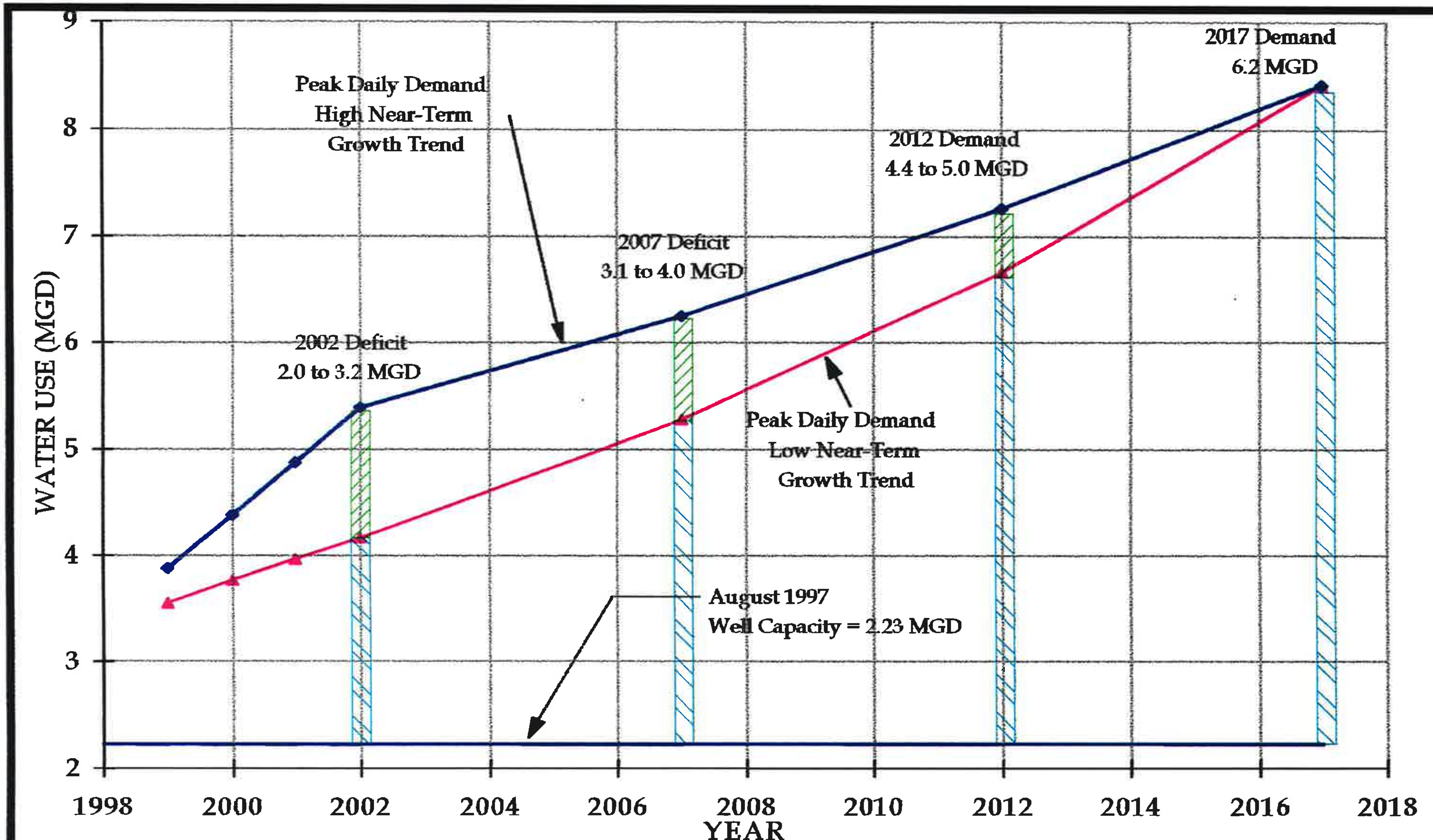
Year	Volume for High Near-Term Growth Trend (MG)	Volume for Low Near-Term Growth Trend (MG)
2000	4.18	3.50
2001	4.74	3.73
2002	5.31	3.95
2007	6.26	5.17
2012	7.38	6.72
2017	8.67	**
2017 base *	6.54	**

** Same as high near-term growth alternative.

Storage Expansion Alternatives. Table 5-3 indicates that close to a 9.0 MG expansion would be needed by 2017 to satisfy the recommended storage volume criteria. This is based on the assumption that the Urban Growth Reserve would be completely developed. Approximately 60 to 70 percent of that expansion would be required by 2007 depending on the near-term growth trend. These relatively high percentages suggest it would be cost effective to construct a single tank to provide the volume required through 2017. On the other hand, it may be more appropriate to expand reservoir capacity in phases due to the uncertainty regarding development in the Urban Growth Reserve. One tank could be constructed as soon as is practical to satisfy near-term requirements. Then, the additional capacity needed to provide the recommended volume through 2017 could be established once updated planning data on the Urban Growth Reserve are available. Data could also be gathered on the early results of a comprehensive water conservation program. Additionally, phased construction would provide the opportunity to evaluate alternative sites for future storage tanks to increase system reliability. Meeting storage requirements with multiple tanks would also increase operational flexibility in the future.

A 6.0 MG reservoir, in conjunction with the existing tank, is projected to be adequate through 2005 at the maximum near-term growth rate and through 2009 at the minimum near-term growth rate. Currently, it is projected that a 3.0 MG tank would then be needed to satisfy the 20-year storage requirements, if the existing tank is kept in service. Section Seven presents opinions of probable project costs for a 6.0- and 3.0-MG tank, added in phases. The probable project cost for one 9.0 MG tank is also included.

The City has purchased property adjacent to the existing tank site for an additional reservoir. Adequate space is available on the site for multiple tanks, if necessary.



CITY OF SHERWOOD	WATER SYSTEM MASTER PLAN UPDATE	FIG. 5–1
BOOKMAN–EDMONSTON ENGINEERING, INC. BEAVERTON, OREGON	SUPPLY CAPACITY REQUIREMENTS	DATE 3/26/99

p. 5-5

R:\..0590020\Figure 5–1.DWG

SECTION SIX

NETWORK ANALYSIS OF WATER SYSTEM

GENERAL

Computer network modeling of water distribution systems is performed to identify areas that may suffer from high or low pressures during some or all operating conditions. In the case of Sherwood, complaints regarding low water pressures in some areas have been received during periods of high residential demands. These low pressure conditions may worsen as tributary demands increase.

Under optimum conditions, operating pressures in municipal water distribution systems should fall within a range of about 50 to 80 psi during non-emergency operations. However, for practical purposes, pressures between 40 and 90 psi are often considered acceptable. A minimum of 40 psi is generally needed to provide adequate operating pressures after losses through individual plumbing systems. This is particularly the case for two-story dwellings and outdoor irrigation systems. Pressures above 90 psi are typically avoided because higher pressures tend to increase the amounts of water used by customers and lost through leaks. A target range of 40 to 90 psi has therefore been used as the basis for evaluating Sherwood's distribution system.

The Uniform Fire Code requires a minimum supply pressure of 20 psi for fire fighting. Therefore, that represents the benchmark value when evaluating the system's ability to deliver fire flows.

The City of Sherwood's water distribution system was modeled with the CYBERNET 3.0 program from Haestad Methods, Incorporated. Pipe friction losses in all the steady-state network analyses performed were calculated using the Hazen-Williams formula. A Hazen-Williams headloss coefficient of 130 has been used for each pipe segment except those in and around the old-town area. The headloss coefficient was assumed to be 100 for these older pipes.

Minor losses in system pressure caused by pipe fittings and valves have been ignored since they are generally not significant. The values used for friction headloss coefficients are believed to be conservative enough to adequately model system losses.

The system models prepared for this version of the CYBERNET program can accommodate a maximum of 100 pipes and junction nodes. The distribution system has therefore been characterized by selecting the water mains that produce a general description of the overall system. As a result, certain water lines, 8 inches and smaller, have been left off the network. Developing a model with these pipes included would provide a level of detail that is not necessary to evaluate the overall system operation.

NETWORK MODEL

Alternative Network Models. The water distribution system was analyzed under various development conditions by creating alternative base models. An initial model was developed using recent past conditions to help calibrate and check the network analysis. Then additional models were generated for the analysis of projected future conditions. The alternative models are summarized below.

1. **Model One**. This system model was developed using 1997 operating conditions and demands. The model was prepared to locate existing housing for distribution of demands and to compare results with actual 1997 operating conditions. Model One was not executed under alternative fire-flow requirements because it represents a past condition.
2. **Model Two (Immediate Future).** This system model has been based on anticipated conditions in 1999. The model includes all housing for which building permits had been issued by March 1998. The tributary population and demands are approximately equal to the 1999 projections listed in Table 4-2 (Page 4-3) and Table 4-4 (Page 4-6). The purpose of this model is to evaluate the distribution system before improvements planned by the City have been completed.
3. **Model Three (Near-term Future).** This model was based on the development of all approved housing projects as of March 1998, plus about 320 DUs in the Urban Reserve. The tributary population and demands are close to the projections for 2002 listed in Table 4-2 (Page 4-3) and Table 4-4 (Page 4-6). This model has been included to evaluate the effect of changes to the system over the next several years. It includes looped pipe extensions into the Urban Reserve from both the Gravity Zone and the Pressure Zone.
4. **Model Four (20-Year Projection with Urban Reserve).** This model was based on Metro's 20-year population projection for Sherwood plus full development of the Urban Reserve. Population growth beyond approved developments has been distributed based on current zoning of undeveloped areas. An alternative model was also executed that did not include the Urban Reserve.

Model One Distribution System. Figure 6-1 shows the pipe and node network that was used in Model One to analyze the Gravity Zone portion of the 1997 system. The areas served by gravity from the reservoir were included in this network model. The existing reservoir serves as a boundary node in the model with a set water surface elevation (WSE).

The Pressure Zone pipe and node network for Model One is shown in Figure 6-2. This network includes only the areas served by the booster station in 1997 and not those areas that have since been added. Pump curve data provided by the Public Works

Department has been used to model the new booster station. The reservoir serves as the source node for the Pressure Zone with the booster station node located immediately downstream.

Models of Future Distribution System. The distribution system models for each future condition were developed from the previous modeling condition by making the changes outlined below.

Model Two - Immediate Future (Figures 6-3 and 6-5)

- Two areas were shifted from the Gravity Zone to the Pressure Zone to reflect the changes implemented in 1998 to improve service as discussed in Section Two.
    - ⇒ The first area is represented by Pipes P-100 and P-101 (William Avenue), P-103 (Smock Street), and P-104 (Mansfield Street). Pipes P-31, P-33, P-34 and P-36 were also added along Sunset and Murdock to model the lines needed to connect this area to the Pressure Zone.
    - ⇒ The other area is along Orchard Court and is modeled by Pipe P-92.
- Several pipe segments were added in the Woodhaven Subdivision to reflect ongoing developments. These include Pipes P-381, P-383, P-384 and P-385.
- Pipe P-165 was added along Gleneagle Drive to better model the system.
- Pipe P-362 was also added to better model the southwest corner of the Woodhaven development between Sunset and Old Sunset Boulevards.

Model Three - Near-term Future (Figures 6-4 and 6-5)

- A 12-inch pipe (P-197) was added under Highway 99W at Tualatin-Sherwood Road to complete a loop to the north end of town.
- A 12-inch line (P-282) was added in the northwest part of town to complete a loop from Gillette Lane to Edy Road. Pipe P-280 was also relocated from Aldridge Terrace to Roellich Avenue to model this loop.
- The pipe size increases identified in Section Three to upgrade the system were instituted in the model. These included the changes listed below.
    - ⇒ The size of Gravity Zone Pipe P-120 (Pine Street and Railroad Avenue) was increased from 8 inches to 12 inches.
    - ⇒ The size of Gravity Zone Pipes P-165 and P-172 (Gleneagle Drive and Twelfth Street) was increased from 6 inches to 8 inches.
    - ⇒ The size of Gravity Zone Pipes P-410 and P-420 (Lincoln and Oregon Streets) was increased from 6 inches to 12 inches.
    - ⇒ The size of Pressure Zone Pipes P-70 and P-80 (Pine Street and Sunset Boulevard) was increased from 8 inches to 12 inches.

- Several more pipe segments were added in the Woodhaven Subdivision to reflect planned developments. These include Pipes P-233 through P-236, and P-238 in the northeast corner of the development, as well as, Pipes P-332 and P-334 through P-337 south of Sunset Boulevard. Other changes in pipe lengths and numbering were made in this area to accommodate the additional pipes.
- Based on the results of the Model Two analysis, two pipes in Roy Street (P-424 and P-425) were shifted from the Gravity Zone to the Pressure Zone. These were added to the Pressure Zone model as Pipes P-105, P-107 and P-108. Pipe P-426 was added in Kathy Street near Well No. 6 to better model the Gravity Zone after Pipes P-424 and P-425 were deleted.
- A 12-inch loop was added to the Gravity Zone from Ladd Hill Road to Middleton Road (Gravity Zone Pipes P-500 and P-502).
- A 12-inch loop was added to the Pressure Zone from Murdock Road to Cascara Terrace (Pressure Zone Pipes P-610, P-612, P-614 and P-616).

Model Four - 20-Year Projection with Urban Reserve (Figures 6-4 and 6-5)

- Pressure Zone Pipes P-70 and P-80 (Pine Street and Sunset Boulevard) were increased in size from 12 inches to 16 inches.
- Gravity Zone Pipes P-100 and P-110 (Division Street from the reservoir to Pine Street) were increased in size from 16 inches to 24 inches.
- Gravity Zone Pipes P-150 and P-160 (North Sherwood Boulevard) were combined into a single pipe to reduce the number of pipes. Node J-150 was deleted and the demand was shifted to Node J-160.

The Woodhaven development was modeled as part of the Gravity Zone under all conditions to evaluate the system's ability to serve this area without a booster station. A booster station is being constructed to serve the southwest part of the Wyndham Ridge development due to higher elevations in that area. However, the model simply includes a demand at Node J-270 (Handley Street) for this area. Modeling the Wyndham Ridge booster station would not serve a useful purpose as part of this study.

A more detailed network model of the Woodhaven area was developed to check the results of the Gravity Zone model for that part of the system. The north and northwest portions of the distribution system were left off to allow more pipes to be added at Woodhaven. Figure 6-6 illustrates this additional model. This model was executed under the same supply and demand conditions used for Model Three to check the results of that model. The two different network models predicted virtually the same results. This indicates the simplified model provides adequate detail to evaluate the system.

Modeling of Water Supply. Analyses have been run with the wells both turned on and off. Each one of the wells is modeled as a constant source of supply with delivery rates as listed below in Table 6-1.

Table 6 - 1

Well Supply Rates for Network Models

Well No. 3 (Node J-122)	650 gpm
Well No. 4 (Node J-250)	180 gpm
Well No. 5 (Node J-320)	475 gpm
Well No. 6 (Node J-432)	550 gpm

The reservoir WSE was set at 375 for the analyses that were run with the wells off to approximate the condition that exists just prior to the start-up of the wells. During normal operating conditions, system pressures are slightly lower with the wells off since the supply comes from a single source instead of four separate locations in the system. However, the difference is not significant without fire flows included in the analysis.

During the last couple of summers, the wells have not been able to supply enough water to offset peak demands. Therefore, the reservoir level has dropped below elevation 375 during these periods. To incorporate this into the model, the reservoir WSE was set at elevation 365 for the analysis of Models Two and Three with the wells running. A WSE of 375 was used in Model Four when the wells were running. This was done because increases in supply and storage capacity should prevent the WSE from dropping below this level unless fire flows are needed.

The Bull Run Connection (Node J-430) was assumed to supply a steady rate of 125 gpm for Models Two and Three. Model Four was run with supply rates of 4.0 and 6.0 MGD being fed through the Bull Run Connection.

Modeling of Demands. Projected peak 6-hour demands were used for the base conditions in the network analyses. As described in Section Four, these demands were based on a per capita demand of 420 gpd, including system losses due to leakage. The total demands distributed through the system in Models Two, Three and Four roughly correspond to the total peak demands listed in Table 4-4 (Page 4-6) for 1999, 2002 and 2017, respectively.

The water use breakdowns given at the end of Section Four between residential and nonresidential customers were used to distribute demands throughout the system. Models One through Three were developed using the current breakdown shown on Page 4-6 and Model Four was based on the projected breakdown given on Page 4-8.

The residential flows were distributed among the network nodes by identifying a tributary area for each node and estimating the number of dwelling units (DU) within that area. Housing data provided by the Planning Department, combined with a map showing lot lines and a 1996 aerial photograph, were used to develop these estimates.

Commercial and industrial flows were distributed by assigning to each node a tributary acreage of land zoned for these uses. This acreage was divided by the total acreage of commercial and industrial land to identify the flow proportion for each node. The areas zoned for commercial development along Highway 99W near the Woodhaven Subdivision were added into the analyses starting with Model Three.

Major landscape irrigation includes water used for outdoor purposes at the schools, large commercial and industrial developments and major homeowners associations. This category does not include irrigation water used at individual homes, small commercial properties and most apartment complexes, since these demands are included in the other categories listed above. The major irrigation demands primarily occur in the following areas:

- Sherwood High School (Node J-240) or Intermediate School (Node J-150);
- the vicinity of Tualatin-Sherwood Road and North Sherwood Boulevard near Pacific Highway and Langer Drive (Nodes J-160, J-170, J-180 and J-182);
- the Woodhaven development along Sunset Boulevard, Pinehurst Drive and Woodhaven Drive (Nodes J-350, J-360, J-380 and J-381); and
- the industrial developments in the northeast part of the City (J-440).

Based on information provided by the Sherwood school district, it is apparent that virtually all the peak summer water demands at the schools can be attributed to irrigation. No 1997 summer session was held in either the intermediate school or Hopkins elementary school and enrollment at Archer Glen elementary school and the high school were about 150 and 50 students, respectively. Therefore, it was assumed that indoor water use at the schools was minor. The outdoor demand was attributed to either the intermediate school or the high school in each model since this heavy irrigation demand occurs at only one school at a time. Thus, two separate analyses were run to model these demands for each school.

Summaries of the peak demand distributions for Network Models One through Four are provided in Appendix B.

Modeling of Fire Flows. Separate fire-flow alternatives were modeled to analyze the system's ability to provide the recommended flow rates at a pressure of at least 20 psi.

The flow rates used for each category of user were as follows:

- schools - 3,500 gpm
- residential - 1,500 gpm
- commercial/light industrial - 2,000 gpm
- major industrial - 4,000 gpm (future)

Actual fire-flow requirements for specific structures are outlined in the Uniform Fire Code based on building construction type and square footage. Without this specific information for each structure in the City, the above values represent conservative estimates.

The fire demands for the schools were applied at the nearest junction node to each facility (Nodes J-150, J-160, J-240 and J-310). Fire-flow demands for the other building types were applied at selected nodes based on current or projected land use. The analyses focused on peripheral areas of the distribution system where longer piping runs could be expected to produce substantial headlosses.

The fire-flow conditions were modeled under two alternative scenarios. One alternative assumed the reservoir WSE was 375 and the wells were off while the other assumed the reservoir was drawn down to elevation 360 and all wells were on.

NETWORK MODEL RESULTS

General. Copies of the computer analysis results for Models One through Four under base conditions (no fire flow) are included in Appendix C. The following paragraphs summarize the results for both base conditions and fire-flow conditions.

Model One - Gravity Zone (Figure 6-1). The analysis for the Gravity Zone verified the problems with low pressures in the two areas the City had planned to shift to the Pressure Zone (Nodes J-302 and J-426). Other areas that experience low pressures include:

- Woodhaven Drive and Sunset Boulevard (Node J-360),
- Colfelt Lane and Old Highway 99W (Node J-365),
- Cinnamon Hill Place (Nodes J-302 and J-304), and
- Roy Street at Cochrane (Node J-424).

The elevations in these areas are all above elevation 265. This elevation is too high to allow a pressure of 45 psi to be maintained without raising the system hydraulic grade line. Cinnamon Hill Place and Roy Street can be served from the pressure zone with minor modifications. However, a booster pumping station would need to be constructed near the west end of Sunset Boulevard to increase operating pressures in the southwest corner of the Woodhaven development. These issues are discussed further in subsequent paragraphs.

The City has also reported pressures below 40 psi along Galewood Drive at the south end of the Woodhaven development (Node J-354). The results of the network analysis do not correlate closely with these reports since the head loss through the system under peak flow conditions has been calculated to be less than 7 feet. Given an elevation in that area of about 255 and a minimum reservoir level of 365, the pressures are estimated to be about 45 psi. The actual pressure drop through the distribution system should not be much greater than this because velocities are quite low when fire demands are not applied to the system. Because the source of this discrepancy is unclear, it is recommended that an additional set of pressure readings be taken to verify operating conditions in the area. This effort should be coordinated with the preliminary design phase for the Woodhaven Booster Station. If significant headlosses are found in specific sections of pipe, this may indicate a problem.

Model One - Pressure Zone (Figure 6-2). The Pressure Zone was first modeled with the large 50-horsepower (hp) pumps in the new booster station. This analysis verified that the peak demand without fire flows is too low to allow one large pump to operate efficiently. Also, the HGL at the pump discharge is over elevation 530, which produces system pressures above 90 psi in several locations where the elevation is below 325.

To improve efficiency and reduce pressures, the City is installing one of the 25-hp pumps from the old booster station with a larger impeller. Therefore, the analyses of Models Two and Three without fire flows were run with curves for the 25-hp pump.

Model Two - Gravity Zone (Figure 6-3). The results of these analyses have indicated that the Gravity Zone is adequate to satisfy the base peak demands and the fire demands, except for the same locations identified in Model One. A fire flow of 1,500 gpm can be supplied to Node J-365 at 20 psi even though the pressure is low during normal operating conditions. A commercial fire flow of 2,000 gpm at Node J-360 (the YMCA) resulted in a pressure of over 25 psi at that location.

Inadequate pressures were, however, calculated at Node J-424 under fire-flow conditions, as well as base conditions. Because Roy Street is adjacent to the Pressure Zone, it was assumed this area would be served from that zone in all subsequent models. Figure 6-7 illustrates a suggested layout that would combine the Forest Avenue area and the Mansfield Street/Smock Street area into a single intermediate zone connected by the existing pipeline in Roy Street (Pressure Zone Nodes J-100 to J-107, Figure 6-5).

Cinnamon Hill Place is now adjacent to the Pressure Zone since Orchard Heights has been shifted from the Gravity Zone. Thus, Cinnamon Hill Place can also be incorporated into the Pressure Zone; however, to maintain the Gravity Zone loop down this street a separate pipe would need to be installed. Figure 6-7 shows a revised service zone layout for this location, as well.

Model Two - Pressure Zone (Figure 6-5). An analysis without fire flows has indicated that a 12-inch diameter impeller would allow the 25-hp pump to produce an HGL of about 520 feet at the pump discharge. This is adequate to produce system pressures above 40 psi everywhere in the pressure zone.

In general, an HGL in the Pressure Zone of over 520 leaves a small gap between the two main service zones. Locations between elevation 265 and elevation 310 will not be serviceable within the range pressure of 40 to 90 psi unless they are placed into an intermediate zone. This includes the east end of Division Street (Node J-20), the west end of Highpoint Drive (Node J-53), an area along Murdock Road (Nodes J-32 and J-36) and the intersection of Pine Street and Sunset Boulevard (Node J-80). Cinnamon Hill Place also falls into this category.

A PRV can be installed in Highpoint Drive, west of Cascara Terrace to eliminate excessive pressures there (over 98 psi). This can also be done for Cinnamon Hill Place by installing a PRV at the north end of Orchard Heights Place. Locating the valve there will allow pressures along Orchard Heights to be lowered, as well. The intersection of Pine and Sunset can be served by a water line installed from Cinnamon Hill to reduce the pressure in that area (see Figure 6-7).

The pressure calculated at Node J-20 was about 91 psi. If field investigations verify excessive pressures in this area, the east end of Division can be served by an extension of the 8-inch pipe that runs west from Roy Street (see Figure 6-7).

The high pressures along Murdock may not be an issue since service connections may not be provided in that location.

Under fire-flow conditions, the 50-hp pumps are needed to supply adequate flows. Since the smaller pump is not designed to operate with the larger pumps, it will need to be shut off when the 50-hp units are started. A pressure below 20 psi was identified at the intersections of Highpoint Drive and Cascara Terrace (Node J-52) and Alder Grove Avenue and Coyote Court (Node J-60) when fire flow demands were applied there. Increasing the size of Pipes P-70 and P-80 (Pine and Sunset from Division to Alder Grove) from 8 inches to 12 inches would provide adequate capacity to meet the 20 psi requirement in these areas. As an alternative, a 12-inch pipe could be installed through the park site, between Nodes J-10 and J-70, to augment the capacities of Pipes P-70 and P-80.

The low operating pressure calculated at Node J-60 without fire flow is not significantly affected by increasing the size of P-70 and P-80 to 12-inches.

Model Three - Gravity Zone (Figure 6-4). The planned system upgrades will allow the modified Gravity Zone to meet minimum pressure requirements for base peak demands and fire flows with the exception of the southwest corner of the Woodhaven subdivision. This area will be discussed in more detail later.

The planned pipe size increases under Twelfth Street and Gleneagle Drive (Pipes P-165 and P-172) from 6 to 8 inches and under Pine Street and Railroad Avenue (Pipe P-120) from 8 to 12 inches will help significantly in directing fire flows to the high school and surrounding areas.

The 12-inch crossing of Highway 99W at Tualatin-Sherwood Road will also be helpful in delivering fire flows to Iris Street and Violet Avenue near Borchers Drive.

It should be noted that the operating pressure at the downstream end of the Bull Run Connection (Node J-430) needs to be about 88 psi to feed water into the Gravity Zone at that location. This is based on a ground elevation of 160 feet at the intersection of Oregon Street and Murdock Road.

Model Three - Pressure Zone (Figure 6-5). The results of the Model Three analysis indicate that the 25-hp pump will produce a discharge HGL of about elevation 505 during peak demands. At that level, the calculated pressure at Node J-60 (Alder Grove) drops to about 36 psi. The HGL should be at or above elevation 515 to produce adequate pressures. Consequently, it may be necessary to switch from the smaller pump to one of the larger pumps during some high-demands periods. One 50-hp pump will produce a discharge HGL of about elevation 530.

Depending upon the condition of the existing 25-hp pump that is being installed, the City may wish to purchase a new pump within 3 years or so. This new pump can be designed to produce a discharge pressure that will maintain system pressures above 40 psi throughout the Pressure Zone.

Model Four - Gravity Zone (Figure 6-4). Based on the preliminary assumptions for development in the Urban Reserve, the distribution system would be adequate to serve the Gravity Zone except at the southwest corner (Nodes J-352, J-360 and J-365).

The upgraded system appears adequate for delivering 4.0 to 6.0 million gallons from the Bull Run Connection. However, the Bull Run PRV would need to be set at about 95 and 100 psi, respectively, to bring these amounts of water into the system at the intersection of Oregon Street and Murdock Road (Node J-430). To eliminate the need to discharge into the system at such high pressures, a 24-inch transmission main could be constructed directly from the downstream side of the PRV to the reservoir site. The pressure in the distribution system at Node J-430 would then be kept below 90 psi.

A key benefit to routing the flows directly from the Bull Run Connection into the reservoirs would be a reduction in retention time in the reservoirs. This would be caused by providing separate inlet and outlet piping to create a flow-through pattern in the reservoirs. Thus, more of the system demands would be met by drawing water from the tanks rather than delivering it directly into the distribution system. The drawback to this transmission system is the need to direct much higher flows out of the reservoir and into the distribution system along Division Street. To accommodate this, the water mains from the reservoir site to the intersection of Division and Pine Streets (P-100 and P-110) would need to be increased in size to 24 inches.

The transmission main from the PRV to the reservoir can be constructed along Murdock Road and Division Streets. This would allow the discharge from Well No. 6 to be fed into the transmission main and transported directly to the reservoir. Blending Well No. 6 water into other supplies upstream of the reservoir may eliminate the need to add chemicals at that well for iron and manganese control. However, if flows from Well No. 6 were fed into the transmission main while it carried 6 MGD, the discharge pressure at the well pump would be about 25 feet greater than the current level. Therefore, a larger pump impeller would probably be needed to maintain its production capacity under this alternative arrangement.

An updated network analysis that includes the proposed Urban Reserve should be performed after the area has been zoned for development.

Model Four - Pressure Zone (Figure 6-5). No additional problems were identified in the Pressure Zone for this future condition. One 50-hp pump would be required for peak demands without fire flows. Three pumps would be required to satisfy the demands plus fire flows. A fire flow of 1500 gpm could be delivered to the Urban Reserve south of Cascara at 20 psi as long as the reservoir level can be maintained at an elevation of 365 feet or higher.

A portion of the Urban Reserve along the east side of Ladd Hill Road is at an elevation that is too high to be served by the Gravity Zone. The model has assumed most of this area would be served by the Pressure Zone; however, another booster station would be required to serve the southernmost portion of this area, since elevations above 410 occur there.

An updated network analysis of the proposed Urban Reserve should be performed after the area has been zoned for development.

Woodhaven Subdivision (Figure 6-6). The results for Models Two, Three and Four all indicate that operating pressures below 40 psi can be expected in the southwest portions of the Woodhaven development. The pressures during Model Four base peak demands have been calculated to be 27 psi at Node J-365 (Colfelt Lane), 35 psi at Node J-360 (Sunset Boulevard and Woodhaven Drive) and 38 psi at Node J-352 (Middleton Road and Inkster Drive). Additionally, Models Three and Four indicate that the pressure at Node J-365 will drop below 20 psi when a fire flow of 1500 gpm is delivered to this node.

To maintain adequate pressures in this area through a 20-year planning period (Model Four), it is recommended that a booster pumping station be installed to serve those service connections located above elevation 255 as shown in Figure 6-8. The booster station should be designed to meet fire flow requirements for the service area, as well. A logical location for this booster station would be at the intersection of Sunset Boulevard and Timbril Lane (see Figure 7-1). Some additional piping may also be needed to maintain system loops in the Gravity Zone while also providing a looped system tributary to the booster station.

A preliminary design report should be prepared to identify a specific location, service area and design capacity for the booster station. Since the City has reported lower pressures along Galewood Drive than this analysis has identified, the preliminary design report should also investigate the need to serve that area with the booster station. Additionally, the report can investigate the specific nature of the structures in the service area to determine a design fire flow rate. Other issues to be addressed include the number and size of the pumps, pump design head, phasing of pump installation, and the potential for a pipeline intertie across Highway 99W. The portion of the Wyndham Ridge development just north of Highway 99W will also be served by a booster station (see Figure 6-8). Therefore, the intertie could link these sections of the Woodhaven and Wyndham Ridge developments to increase reliability.

RECOMMENDATIONS

It is recommended that the planned improvements to the system be implemented according to the priority ranking listed below.

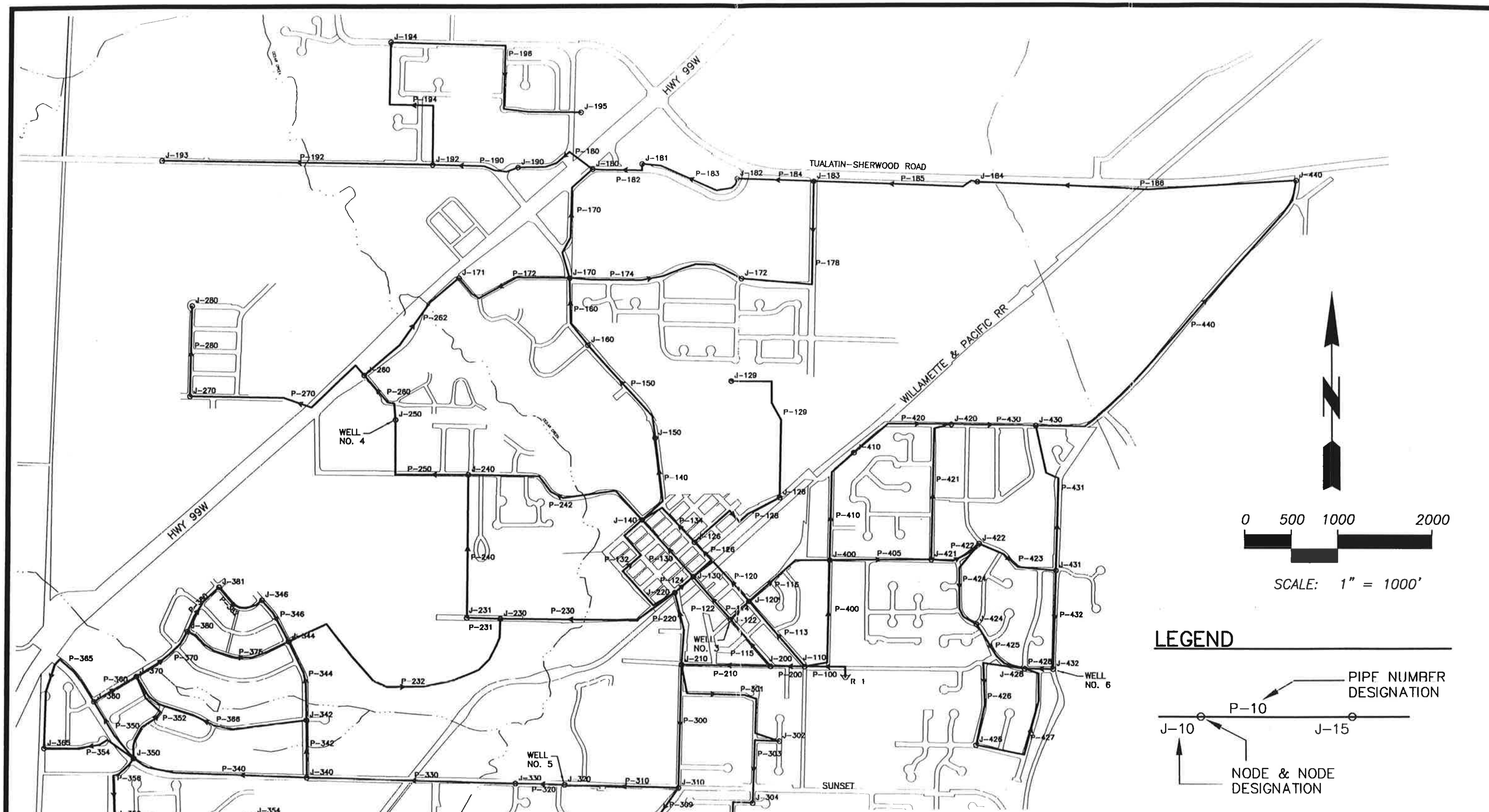
1. Improvements that should be completed within the next three years.
   - Have a booster station installed to serve roughly 80 acres in the southwest part of the Woodhaven development where elevations rise above 255 feet. The station should be designed to supply fire flows in addition to the peak hourly demand. A preliminary design report should be prepared to establish firm design criteria for the booster station.
   - Install 12-inch piping to replace the 8-inch water mains under Pine Street from Willamette Street to Columbia Street and under Washington Street from Columbia Street to Railroad Avenue. Also install a 12-inch pipe under Columbia to connect the two segments described above.
   - Install 8-inch pipe to replace the 6-inch water lines under Gleneagle Drive and Twelfth Street. A short section of 12-inch pipe should be installed in Gleneagle between Twelfth and Highway 99W.
   - Install 12-inch pipe across Highway 99W under Tualatin-Sherwood and Tualatin-Scholls Roads to complete a loop to the north end of town.
   - Install a 12-inch pipe from the Pressure Zone booster station through the park site to the intersection of Sunset Boulevard and Alder Grove Avenue. This transmission main will augment the existing Pressure Zone piping capacity to

deliver fire flows to the southerly portion of Alder Grove Avenue and the area of Highpoint Drive and Cascara Terrace.

- Modify the system to serve Cinnamon Hill Place and the southerly section of Roy Street from the Pressure Zone as shown in Figure 6-7. Install a PRV at the north end of Orchard Heights Place to reduce pressures along Cinnamon Hill and Orchard Heights.
- Install a PRV in Highpoint Drive west of Cascara Terrace to reduce pressures at the west end of Highpoint.
- Evaluate the need to purchase a "small" pump (30- or 40-hp) for the Pressure Zone Booster Station designed to produce a discharge HGL of about 515 feet.

2. Install 12-inch pipe from the north end of Roellich Avenue to Edy Road. This pipe should be added as development increases along Edy Road to improve system looping. This pipe is not critical for maintaining adequate pressures due to low elevations in the area.
3. Install 12-inch piping to replace the 6-inch water lines under Lincoln and Oregon Streets between Willamette and Hall Streets. This work should be completed within the next three to five years to increase system capacity.
4. Install a 24-inch transmission main from the Bull Run Connection to the reservoirs along Murdock Road and Division Street. The length would be approximately 5,000 lineal feet. This improvement will improve reservoir operation and keep pressures below 90 psi along Oregon Street. For budgeting purposes, it is recommended the City plan to construct the transmission main soon after an alternative source of supply is secured through the Bull Run Connection.

   The discharge line from Well No. 6 can be rerouted to direct the supply into this 24-inch line. However, the pump impeller size may need to be increased in the future to maintain production capacity as flows through the Bull Run Connection are increased. This is due to the increasing pressure in the pipeline.
5. Increase the size of the existing 16-inch pipe between the reservoir site and the intersection of Pine and Division Streets to 24 inches. This improvement should be installed concurrent with the installation of the 24-inch transmission main listed in Item 4.



CITY OF SHERWOOD

WATER SYSTEM MASTER PLAN UPDATE

FIG. 6-1

BOOKMAN-EDMONSTON ENGINEERING, INC.
BEAVERTON, OREGON

GRAVITY ZONE NETWORK
MODEL ONE

DATE
1/11/99

R:\..0590020\FIG6-1.DWG

LEGEND

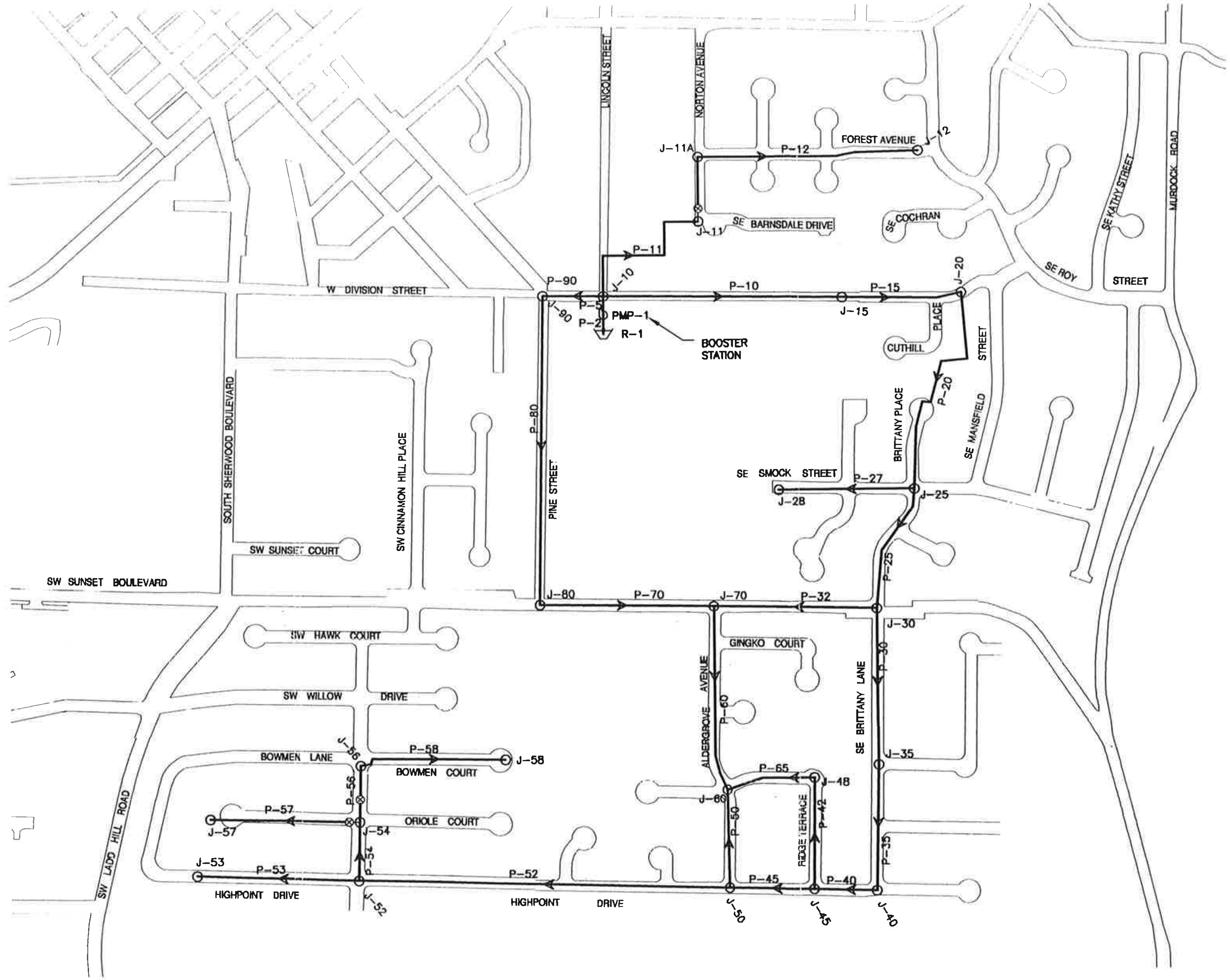
PIPE NUMBER DESIGNATION

P-10

J-10 J-15

NODE & NODE DESIGNATION

⊗ PRESSURE REDUCING VALVE

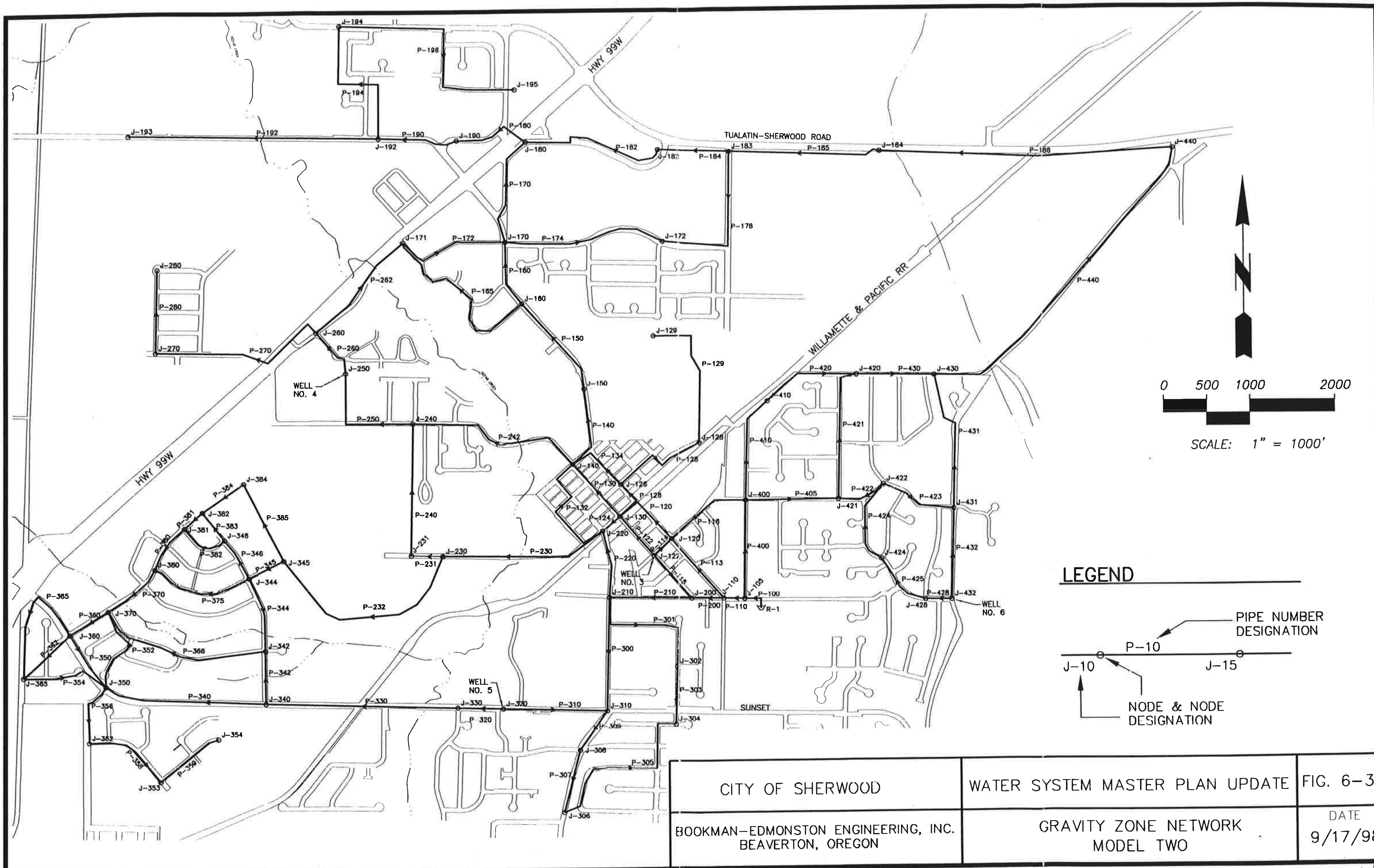


0 250 500 1000

SCALE: 1"=500'

CITY OF SHERWOOD	WATER SYSTEM MASTER PLAN UPDATE	FIGURE 6-2
BOOKMAN-EDMONSTON ENGINEERING, INC. BEAVERTON, OREGON	PRESSURE ZONE NETWORK MODEL ONE	DATE 9/17/98

R:\..0590020\FIG6-2.DWG



CITY OF SHERWOOD

BOOKMAN-EDMONSTON ENGINEERING, INC.
BEAVERTON, OREGON

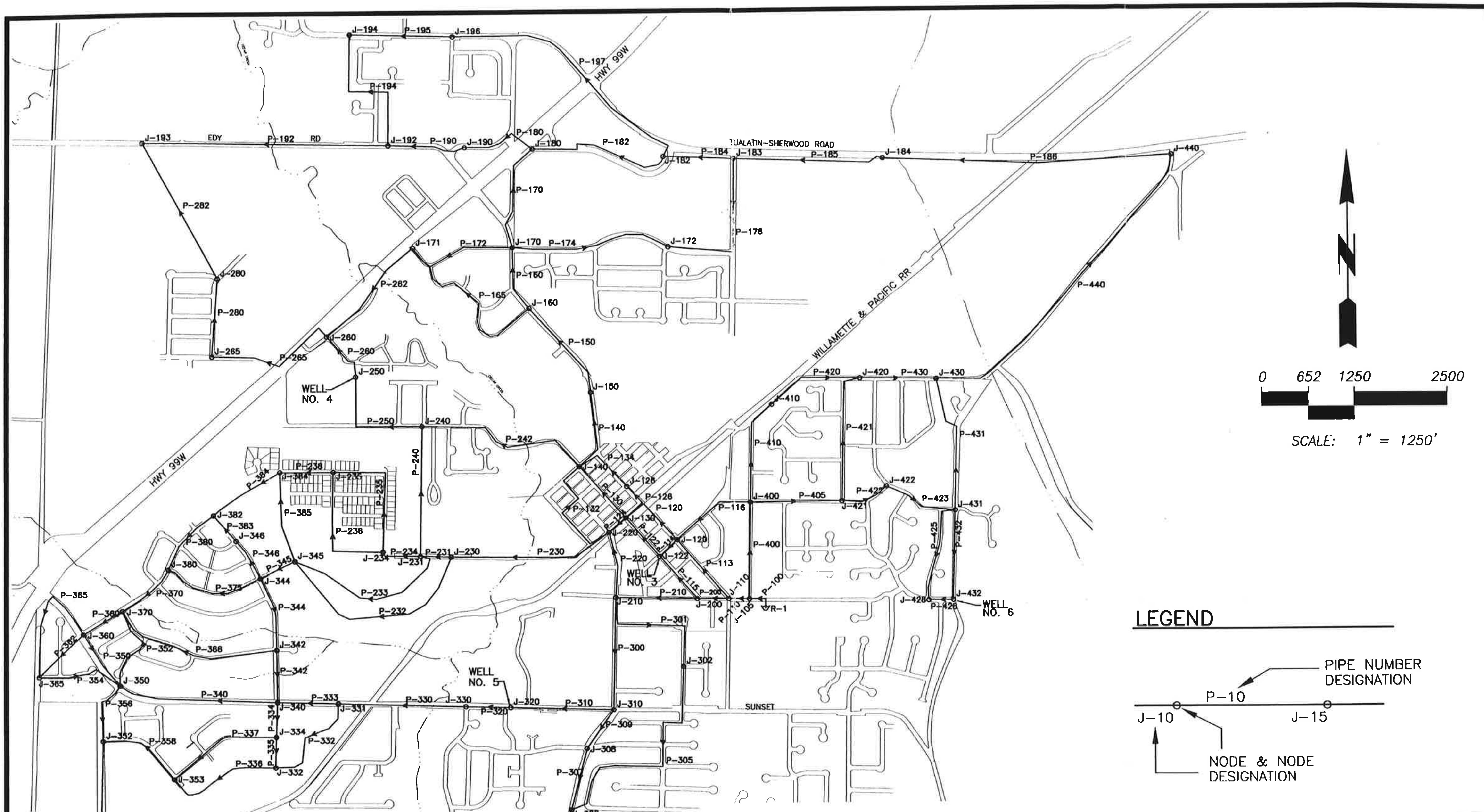
WATER SYSTEM MASTER PLAN UPDATE

GRAVITY ZONE NETWORK
MODEL TWO

FIG. 6-3

DATE
9/17/98

R:_0590020\FIG6-3.DWG



CITY OF SHERWOOD

BOOKMAN-EDMONSTON ENGINEERING, INC.
BEAVERTON, OREGON

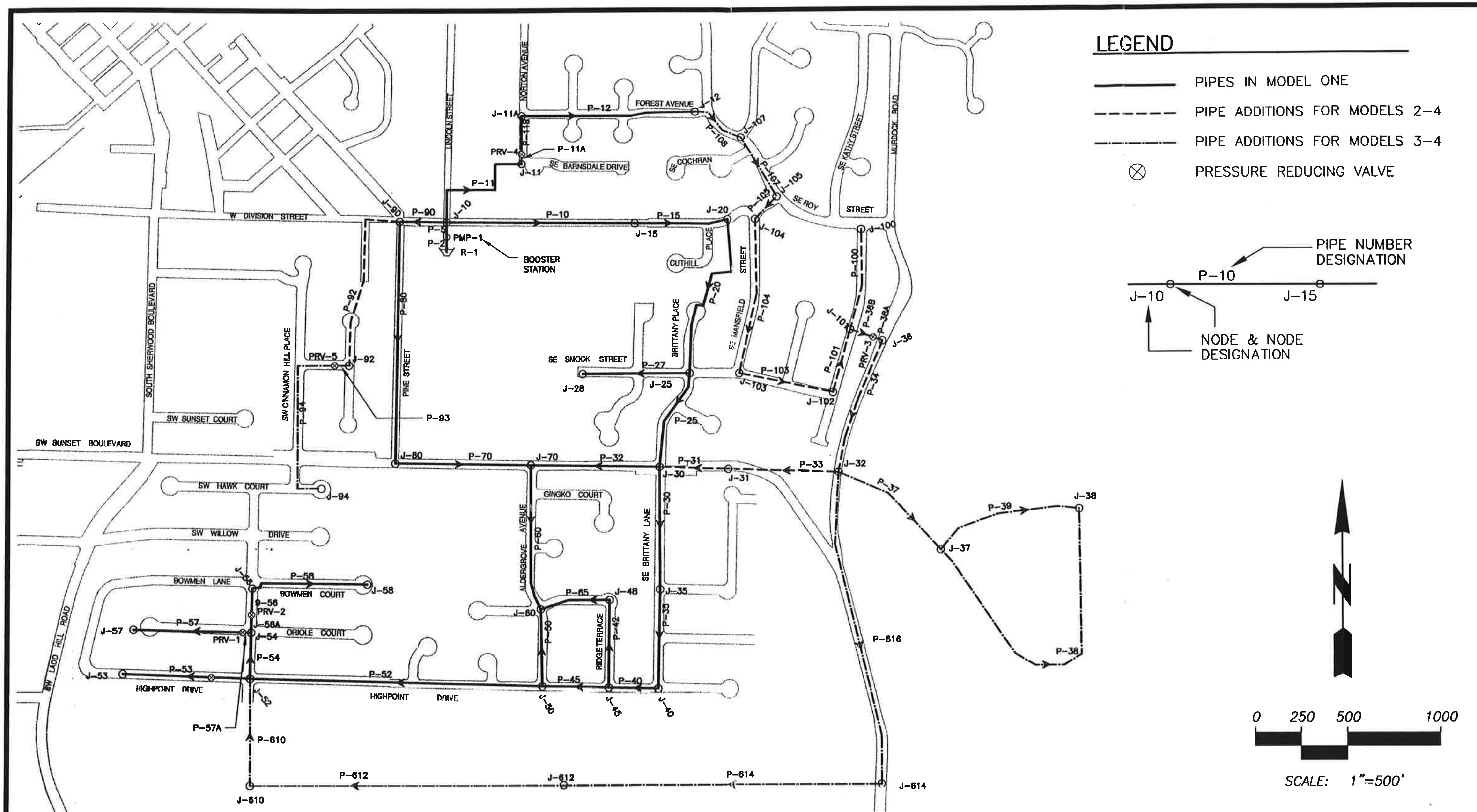
WATER SYSTEM MASTER PLAN UPDATE

GRAVITY ZONE NETWORK
MODELS THREE & FOUR

FIG. 6-4

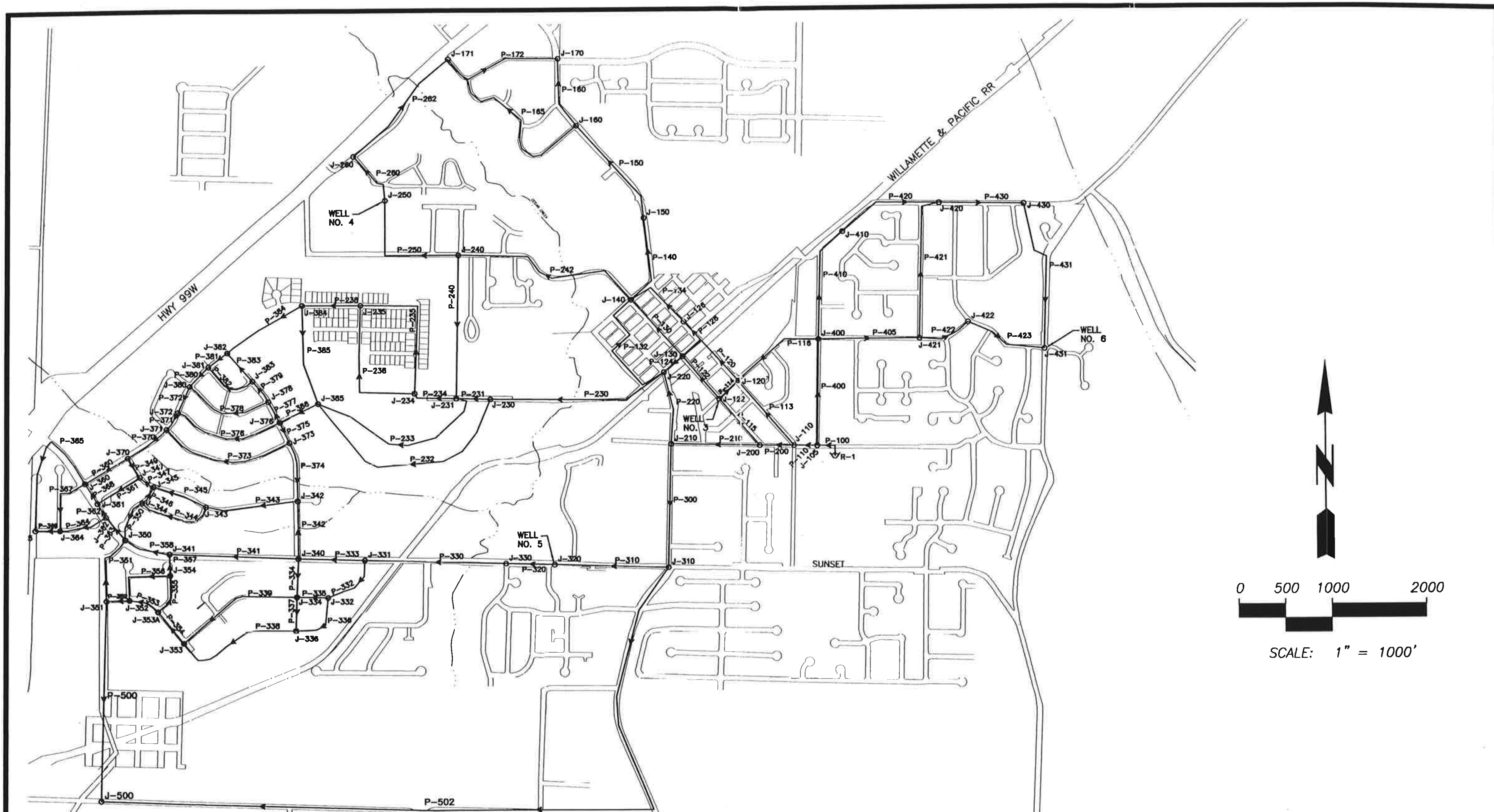
DATE
1/4/99

R:\..0590020\FIG6-7A.DWG



CITY OF SHERWOOD	WATER SYSTEM MASTER PLAN UPDATE	FIGURE 6–5
BOOKMAN–EDMONSTON ENGINEERING, INC. BEAVERTON, OREGON	PRESSURE ZONE NETWORK MODELS TWO THROUGH FOUR	DATE 1/4/99

R:\...\Figure 6–5.DWG



CITY OF SHERWOOD

WATER SYSTEM MASTER PLAN UPDATE

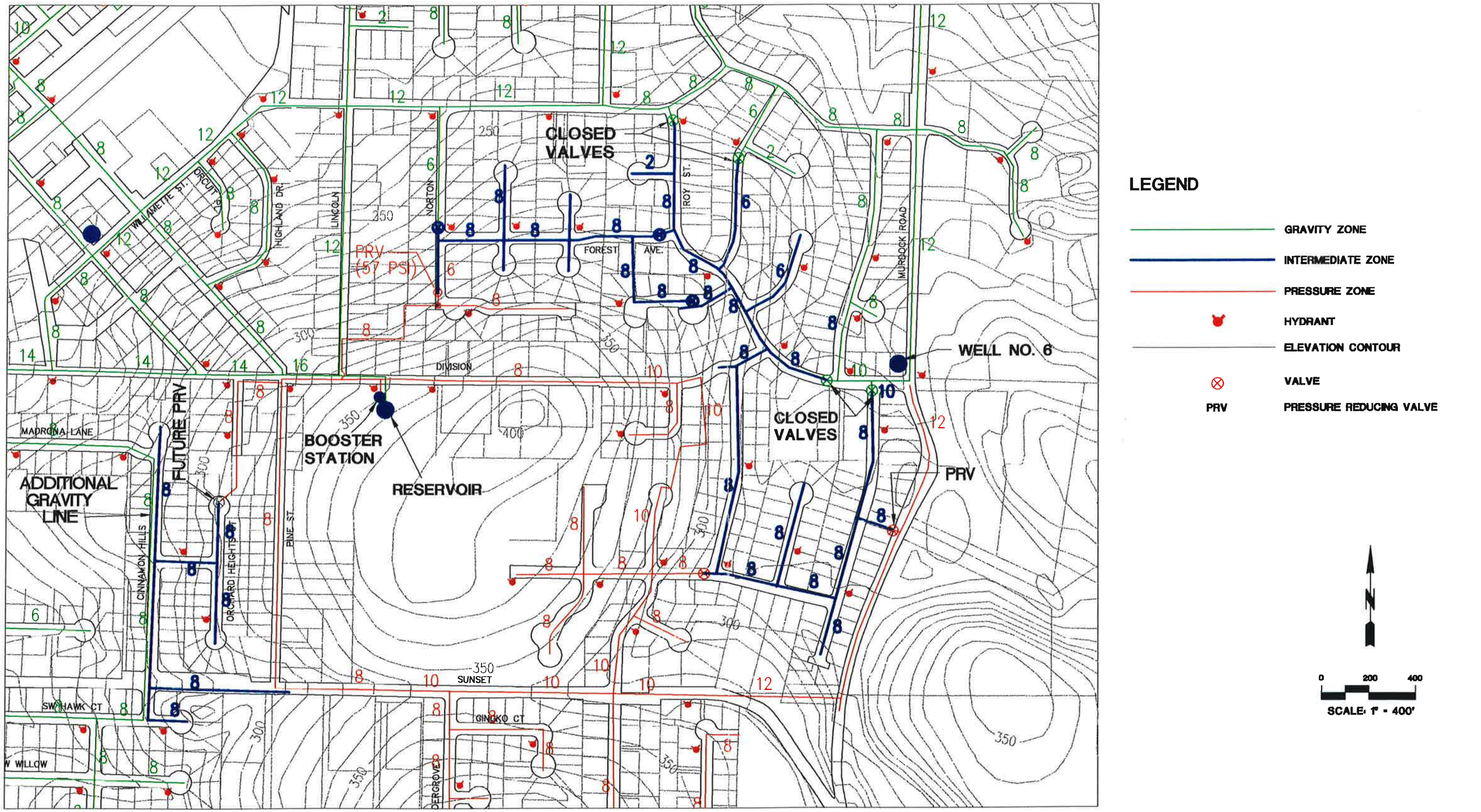
FIGURE 6-6

BOOKMAN-EDMONSTON ENGINEERING, INC.
BEAVERTON, OREGON

GRAVITY ZONE NETWORK
WOODHAVEN MODEL

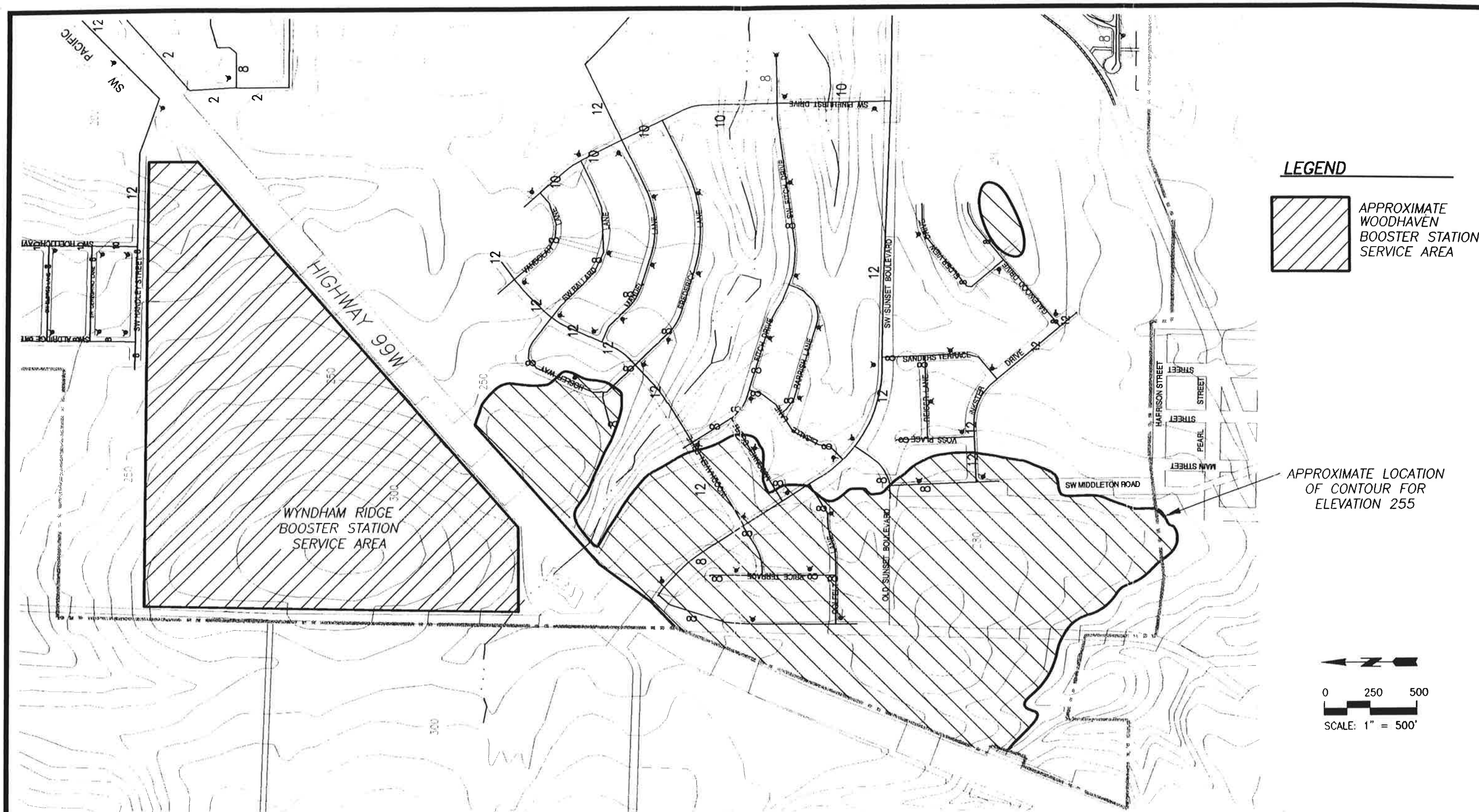
DATE
1/4/99

R:\..0590020\FIG6-9.DWG



CITY OF SHERWOOD	WATER SYSTEM MASTER PLAN UPDATE	FIGURE 6–7
BOOKMAN–EDMONSTON ENGINEERING, INC. BEAVERTON, OREGON	SUGGESTED SERVICE ZONE REVISIONS	DATE 10/1/98

R:\..\WATER SYSTEM CLOSE-UP2.DWG



CITY OF SHERWOOD	WATER SYSTEM MASTER PLAN UPDATE	FIGURE 6-8
BOOKMAN-EDMONSTON ENGINEERING, INC. BEAVERTON, OREGON	APPROXIMATE BOOSTER STATION SERVICE AREA	DATE 3/26/99

CITY OF SHERWOOD
WATER SYSTEM MASTER PLAN UPDATE
SECTION SEVEN

RECOMMENDATIONS AND PROBABLE PROJECT COSTS

GENERAL

This section summarizes the water system capital improvements recommended in the preceding sections and provides preliminary opinions of probable project costs. These improvements are needed to meet increasing customer demands and requirements for fire flow storage and delivery. General recommendations regarding alternative supply capacities and non-capital improvements are also presented.

RECOMMENDED IMPROVEMENTS

Summary of Capital Improvement Projects. The following is a summary of the recommended capital improvements projects broken down by system component.

A. Treated Water Storage

1. Initial Phase: A new concrete reservoir should be constructed on City-owned land adjacent to the existing reservoir to increase storage capacity. Using current projections, a 9.0 MG expansion is needed to provide adequate volume for peak demands and fire flows until the Year 2017. However, it may be more cost effective to construct a smaller tank at this time and increase capacity again in the near future. This would allow capacity requirements and siting issues to be reevaluated once development plans for the Urban Reserve have been formalized. The effects of water conservation could also be taken into account before the design capacity of a second tank is finalized.

   Therefore, it is recommended that the City construct a 6.0 MG tank initially and plan to add more capacity later. The probable project costs for both a 6.0 MG and a 9.0 MG reservoir are presented below for comparison.

   Given current demands and rapid growth rates, the design and construction of a reservoir should proceed as soon as possible. The new tank will operate at the same range of elevations as the existing tank.

2. Future Phase: Based on current projections, the City should plan to construct additional reservoir capacity sometime between 2005 and 2009. Therefore, planning efforts should be completed in 5 years (early 2004). Population and water use projections should be updated and a structural evaluation of the existing 2.0 MG reservoir should be completed. Consideration should also be given to alternative sites for a reservoir such as in the northwest part of the City or at higher elevations in the Urban Reserve.

At this time, it is projected that an additional 3.0 MG in storage volume will be needed to meet demands through 2017. Therefore, probable present worth costs are provided below for a 3.0 MG concrete tank.

B. Southwest Booster Station

A booster station should be installed that serves those portions of the Woodhaven Subdivision and adjacent areas that lie above elevation 245. A preliminary design report should be prepared to identify the specific service area and design peak demands for the booster station. Additionally, the report can investigate the specific nature of the structures in the service area to determine a design fire flow rate. Other issues to be addressed include the number and size of the pumps, pump design head, phasing of pump installation, and the potential for a pipeline intertie across Highway 99W.

C. Distribution and Transmission System

The following piping improvements are recommended to upgrade the water system. Figure 7-1 should be referred to for project locations. In some cases parallel pipes could be installed to increase capacity instead of replacement lines. This can reduce material costs, but it also would leave older pipes in service. A determination regarding the option of installing parallel pipes can be made during preliminary design. To be conservative, this report assumes replacement pipes will be installed.

1. Increase capacity of key water mains:

- Install 12-inch pipe from the Pressure Zone booster station through the park site to the intersection of Sunset Boulevard and Alder Grove Avenue. This transmission main will augment the existing Pressure Zone water line capacity. The upgrade is needed to deliver fire flows to the southerly portion of Alder Grove Avenue and the area of Highpoint Drive and Cascara Terrace. Approximately 1,600 lineal feet of pipe are included. See Item 1 on Figure 7-1.

- Install 12-inch piping to replace the 8-inch water mains under Pine Street from Willamette Street to Columbia Street and under Washington Street from Columbia Street to Railroad Avenue. Also install a 12-inch pipe under Columbia to connect the two segments described above. The work consists of approximately 900 lineal feet of pipe, including a bore under the railroad tracks. This upgrade is needed to replace aging pipe and to improve the transfer of water from the reservoir to the north and west parts of town. See Item 2 on Figure 7-1.

- Install 8-inch piping to replace the 6-inch water lines under Gleneagle Drive and Twelfth Street. The short section of pipe in Gleneagle between Twelfth and Highway 99W should be 12 inches in diameter. This upgrade should be scheduled in the next three years to increase line capacity for fire flows to the high school. Approximately 2,850 lineal feet of 8-inch pipe and 300 lineal feet of 12-inch pipe are included. See Item 3 on Figure 7-1.

- Install 12-inch piping to replace the 6-inch water lines under Lincoln and Oregon Streets between Willamette and Hall Streets. This work should be completed within the next three to five years to increase system capacity. Approximately 2,400 lineal feet of pipe are included. See Item 4 on Figure 7-1.

2. Installations of water lines to complete system loops:

   - Install a 12-inch pipe across Highway 99W under Tualatin-Sherwood and Tualatin-Scholls Roads to complete a loop to the north end of town. This pipe should be added in the next three years to improve system reliability for fire flow delivery north of Highway 99W. Approximately 3,000 lineal feet of pipe are included. See Item 5 on Figure 7-1.

   - Install a 12-inch pipe from the north end of Roellich Avenue to Edy Road. This pipe should be added as development increases along Edy Road to improve system looping. This pipe is not critical for maintaining adequate pressures due to low elevations in the area. Approximately 1,500 lineal feet of pipe are included. See Item 6 on Figure 7-1.

   - Extend a 12-inch pipe northwest from Highway 99W near Cedar Creek. This pipe will connect to a water line that will extend south from Edy Road as part of a planned development. Approximately 500 lineal feet of pipe are included. See Item 10 on Figure 7-1.

   - Extend a 12-inch pipe along Galbreath Road and Cipole Road to connect to existing and proposed water lines. This alignment will include a bore under the Willamette and Pacific Railroad at Cipole Road. Approximately 2,400 lineal feet of pipe are included. See Item 11 on Figure 7-1.

3. Increase transmission main capacity:

   - Install a 24-inch transmission main from the downstream end of the Bull Run Connection to the existing reservoir site. The pipe alignment can extend along Murdock Road and Division Street to allow the discharge line from Well No. 6 to be connected to this main. This project can be planned as a long-term improvement. It should be constructed once the Bull Run Connection has become the primary means for supplying water and flow rates through the line approach 4.0 MGD. This is projected to occur in about 9 to 14 years. It may be preferable to construct a first leg of this transmission line from Well No. 6 to the reservoir sooner. That would allow water from Well No. 6 to be blended with other water before it reaches customers. A total of approximately 5,000 lineal feet of pipe are included in the entire transmission main. The section from Well No. 6 to the reservoir would be approximately 2,350 lineal feet. See Item 7 on Figure 7-1.

- Replace the existing 16-inch Gravity Zone pipe between the existing reservoir site and the intersection of Lincoln and Division Streets with a 24-inch line. Approximately 300 lineal feet of pipe are included. This improvement should be constructed when the 24-inch transmission main described above is installed. See Item 8 on Figure 7-1.

4. Increase diameter of undersized water lines:

   - Replace 2-, 4- and 6-inch pipe lines with 8-inch pipe (see Figure 3-1). Approximately 11,300 lineal feet of pipe are included, not counting the sections identified above.

Other Recommended Projects. The following projects are not part of the recommended capital improvements, but should be initiated to ensure that the water system can meet the City's needs.

A. Production Wells

   As a separate phase in the Master Planning effort, the City needs to evaluate well production capacity and the impact the wells are having on groundwater levels. The feasibility of upgrading the wells so they can operate at their permitted capacities should be addressed. The City is currently having a geotechnical investigation of Well No. 3 completed as part of this effort.

B. Institute the recommended shifts in service zone boundaries as described in Section Six (See Figure 6-7).

C. An alternative water supply must be obtained to augment and potentially replace the municipal wells. The City should continue to participate in regional planning efforts to develop the alternative supply as soon as is practical. An additional supply capacity of about 6.2 MGD is needed by the Year 2017 based on current projections with the Urban Reserve included. This supply deficit assumes the well production rate would remain at the level reported for August of 1997.

D. A Water Conservation and Management Plan should be completed and an ongoing program of water conservation measures should be implemented. Water conservation can reduce reliance on the wells and alleviate near-term water shortages. It can also reduce future capital outlays by reducing supply and storage capacity requirements.

E. Evaluate the need to purchase a new pump for the Pressure Zone Booster Station in 3 to 4 years to replace the older 25-hp pump that is being installed.

F. Implement a systematic water meter inspection and replacement program to remove meters that no longer function properly.

G. Develop a schedule for periodically flushing fire hydrants throughout the system.

BASIS FOR OPINIONS OF PROBABLE PROJECT COSTS

Opinions of probable project costs have been developed using information available at the time this study was prepared. These estimates should be used for project evaluation and planning while noting that final costs will depend on current market conditions, final project scope, and other variable factors. Project feasibility, risks and funding needs should be carefully reviewed prior to making financial decisions or preparing specific project budgets.

The estimates of probable project costs presented below are considered Budget Estimates, defined as having an accuracy of +30 to -15 percent under the criteria established by the American Association of Cost Estimating Engineers. These preliminary project cost estimates are current to December 1998. The construction cost estimates used as the basis for the project costs include a 20 percent contingency factor. To establish the probable project costs, an additional 20 percent allowance was then added for engineering, bidding, and construction services.

OPINIONS OF PROBABLE PROJECT COSTS

A. **Treated Water Storage**

1. Current Phase: One 6.0-MG concrete reservoir: $ 3,800,000.

   Probable project costs assume the reservoir will be circular and mostly buried. The roof would be above grade and nearly flat. The wall height would be 32 feet and the overflow would be 30 feet above the tank floor. An inlet vault would be included with an altitude valve and isolation valves.

2. Future Phase: An additional storage capacity of 3.0 MG to provide adequate storage volume to accommodate the development of the Urban Reserve. The probable project cost for a single 3.0 MG concrete tank similar to the type described for the current phase would be approximately $ 2,300,000. The present worth cost would be $ 1,925,000, assuming the tank is constructed in 2005. This is based on a 3 percent discount rate.

3. Alternative: One 9.0-MG tank with the same features - $ 5,100,000).

B. Southwest Booster Station

Install a booster station that serves those portions of the Woodhaven Subdivision and adjacent areas that lie above elevation 245: $ 700,000. The addition of an 8-inch pipeline intertie across Highway 99W would add another $ 150,000.

C. Distribution System

1. Increase capacity of key water mains:

   - Install 1,600 lineal feet of 12-inch pipe through the park site to augment the 8-inch water mains under Pine Street and Sunset Boulevard from Division Street to Alder Grove Avenue: $ 125,000.
   - Install 900 lineal feet of 12-inch piping under Pine, Columbia and Washington Streets to replace existing 8-inch water mains and increase capacity: $ 140,000. This amount includes about $ 70,000 in project costs for a bore under the railroad.
   - Install 2,950 lineal feet of 8-inch piping and 300 lineal feet of 12-inch piping to replace the 6-inch water lines under Gleneagle Drive and Twelfth Street: $ 195,000.
   - Install 2,400 lineal feet of 12-inch piping to replace the 6-inch water lines under Lincoln and Oregon Streets: $ 220,000.

2. Installations of water lines to complete system loops:

   - Install 3,000 lineal feet of 12-inch pipe across Highway 99W under Tualatin-Sherwood and Tualatin-Scholls Roads: $ 385,000. This amount includes about $ 140,000 in project costs for a bore under Highway 99W.
   - Install 1,500 lineal feet of 12-inch pipe from the north end of Roellich Avenue to Edy Road: $ 130,000.
   - Install 500 lineal feet of 12-inch pipe northwest from Highway 99W near Cedar Creek: $ 40,000.
   - Install 2,400 lineal feet of 12-inch pipe along Galbreath and Cipole Roads to connect to existing and proposed water lines: $ 270,000. This amount includes about $ 70,000 in project costs for a bore under the railroad.

3. Increase transmission main capacity:

   - Install approximately 5,000 lineal feet of 24-inch pipe along Murdock Road and Division Street from the Bull Run Connection to the existing reservoir site: $ 790,000.
   - Install approximately 300 lineal feet of 24-inch pipe to replace the existing 16-inch Gravity Zone pipe between the existing reservoir site and the intersection of Lincoln and Division Streets: $ 50,000.

4. Increase diameter of undersized water lines:

   - Replace approximately 11,300 lineal feet of 2-, 4- and 6-inch pipe lines with 8-inch pipe: $ 690,000.

D. Total Capital Improvements for Storage and Distribution Systems

A summary of the probable project costs itemized above is tabulated in Appendix D. The total for the reservoir and distribution system capital improvements is $9,610,000. This assumes two reservoirs would be constructed in phases.

Record note: Water System Master Plan Update Figure 7-1, Water System Piping Improvements map kept in City Recorder's office.

CITY OF SHERWOOD
WATER SYSTEM MASTER PLAN UPDATE

APPENDIX A

COUNCIL RESOLUTION
AUTHORIZING MASTER PLAN UPDATE

City of Sherwood, Oregon

Resolution No. 97-717

A RESOLUTION AWARDING A BID TO RMI FOR THE PURPOSE OF UPDATING THE CITY'S WATER SYSTEM MASTER PLAN, AND ESTABLISHING AN EFFECTIVE DATE.

WHEREAS, the City of Sherwood's existing water system requires the implementation of a series of upgrades to improve service and meet the demands of a rapidly growing population; and

WHEREAS, it has become necessary for the City of Sherwood to revise the City's Water System Master Plan to assess the adequacy of both the City's water supply and distribution in order to meet present and future demands; and

WHEREAS, the City of Sherwood selected RMI-Bookman Edmonston Engineering Inc. of Beaverton, Oregon to provide professional services to update the City's Water System Master Plan; and

WHEREAS, RMI-Bookman Edmonston Engineering Inc. will develop a comprehensive program for expanding and improving the City's water system covering a 20 year period.

NOW, THEREFORE, THE CITY RESOLVES AS FOLLOWS:

Section 1: The contract is awarded to RMI-Bookman Edmonston Engineering Inc. of Beaverton, Oregon.

Section 2: The City Manager is hereby authorized to execute a contract with RMI-Bookman Edmonston Engineering Inc. in an amount not to exceed $35,000.00.

Section 3: This Resolution shall be effective upon its approval and adoption.

Duly passed by the City Council this 9th day of December 1997.

Ron Tobias, Mayor

ATTEST:

Jon Bormet, City Manager-Recorder

CITY OF SHERWOOD
WATER SYSTEM MASTER PLAN UPDATE

APPENDIX B

DEMAND DISTRIBUTIONS FOR NETWORK MODELS

MODEL ONE
PRESENT-TERM CONDITION 1997

SHERWOOD - TOTAL DOMESTIC WATER DEMAND

GRAVITY SYSTEM				
JUNCTION NODE	DWELLING UNITS	DEMAND (GPM)	INFLOW (GPM)	REMARKS
110	0	0.00		
120	55	34.65		
122	23	14.49	650.00	WELL #3
126	13	8.19		
128	23	14.49		
130	9	5.67		
140	20	12.60		
160	152	95.76		
170	155	97.65		
171	146	91.98		MOBILE HOMES
172	235	148.05		
192	29	18.27		
193	40	25.20		
194	65	40.95		
195	75	47.25		
200	33	20.79		
210	96	60.48		
220	20	12.60		
240	80	50.40		
250	96	60.48	180.00	WELL #4
260	68	42.84		
270	28	17.64		
280	27	17.01		
302	27	17.01		
304	27	17.01		
306	22	13.86		
308	32	20.16		
310	54	34.02		
320	43	27.09	475.00	WELL #5
330	45	28.35		
344	22	13.86		
350	36	22.68		
352	15	9.45		
354	32	20.16		
360	11	6.93		
365	13	8.19		
370	23	14.49		
380	38	23.94		
400	69	43.47		
410	52	32.76		
420	94	59.22		
421	54	34.02		
422	47	29.61		
424	33	20.79		
426	53	33.39		
428	102	64.26		
430	107	67.41		
431	63	39.69		
432			550.00	WELL #6
TOTAL=	2602	1,639.3	1855.00	

NOTE: 3044 DWELLING UNITS X 2.88 X 420 GPDPC X 75% = 1917.7 GPM FOR RESIDENTIAL

MODEL ONE
PRESENT-TERM CONDITION 1997

SHERWOOD - TOTAL DOMESTIC WATER DEMAND

PRESSURE SYSTEM			
JUNCTION NODE	DWELLING UNITS	DEMAND (GPM)	REMARKS
10	5	3.15	
11	28	17.64	
12	47	29.61	
15	6	3.78	
20	17	10.71	
22	6	3.78	
25	8	5.04	
28	23	14.49	
30	54	34.02	
35	33	20.79	
40	23	14.49	
45	12	7.56	
48	13	8.19	
50	26	16.38	
52	10	6.30	
53	6	3.78	
54	19	11.97	
56	7	4.41	
57	6	3.78	
58	7	4.41	
60	26	16.38	
70	38	23.94	
80	22	13.86	
TOTAL=	442	278.5	

NOTE: 3044 DWELLING UNITS X 2.88 X 420 GPDPC X 75% = 1917.7 GPM FOR RESIDENTIAL

MODEL ONE
PRESENT-TERM CONDITION 1997

SHERWOOD - TOTAL COMMERCIAL - INDUSTRIAL WATER DEMAND

COMMERCIAL - INDUSTRIAL			
JUNCTION NODE	AREA ACRE	DEMAND (GPM)	REMARKS
120	8	2.37	
122	0	0.00	WELL #3
126	4	1.18	
128	1	0.30	
130	4	1.18	
140	10	2.96	
150	0	0.00	
150	1	0.30	
160	1	0.30	
170	7	2.07	
171	11	3.25	
180	54	15.97	
182	16	4.73	
183	16	4.73	
184	86	25.44	
190	41	12.13	
192	41	12.13	
195	16	4.73	
220	3	0.89	
240	0	5.00	HIGH SCHOOL
250	16	4.73	WELL #4
260	0	0.00	
270	0	0.00	
310	0	5.00	ARCHER GLEN ELEM. SCHOOL
320	0	0.00	WELL #5
340	0	0.00	
370	0	0.00	
381	0	0.00	
410	7	2.07	
420	35	10.35	
430	35	10.35	
432	0	0.00	WELL #6
440	280	82.83	
TOTAL=	693	215.00	

MAJOR IRRIGATION USERS		
JUNCTION NODE	DEMAND (GPM)	REMARKS
	425.00	SEE ATTACHMENT

NOTE: 3044 DWELLING UNITS X 2.88 X 420 GPDPC X 25% = 639.2 GPM FOR COMMERCIAL/INDUSTRIAL

Major Irrigation Distribution

J-240 = 200 gpm (High School)

Total demand for other major users = 225 gpm

Node	Demand	Remarks
J-160	34.0	15%
J-170	34.0	15%
J-180	56.0	25%
J-182	22.5	10%
J-342	4.5	2%
J-344	4.5	2%
J-350	11.0	5%
J-360	27.0	12%
J-380	4.5	2%
J-381	4.5	2%
J-440	22.5	10%
Total	225	100%

MODEL TWO
IMMEDIATE FUTURE (1999)
HOUSING DEVELOPMENT CURRENTLY PLANNED
FROM 3/6/98 APPROVED HOUSING LIST - BUILDING PERMITS ISSUED
DEMAND DISTRIBUTION

SHERWOOD - TOTAL DOMESTIC WATER DEMAND

GRAVITY SYSTEM				
JUNCTION NODE	DWELLING UNITS	DEMAND (GPM)	INFLOW (GPM)	REMARKS
110	0	0.00		
120	55	34.90		
122	23	14.59	650.00	WELL #3
126	13	8.25		
128	23	14.59		
130	9	5.71		
140	20	12.69		
160	152	96.44		
170	155	98.35		
171	146	92.64		MOBILE HOMES
172	235	149.11		
192	29	18.40		
193	109	69.16		
194	80	50.76		
195	112	71.06		
200	33	20.94		
210	128	81.22		
220	20	12.69		
240	90	57.11		
250	96	60.91	180.00	WELL #4
260	68	43.15		
270	57	36.17		
280	57	36.17		
304	27	17.13		
306	32	20.30		
308	59	37.44		
310	54	34.26		
320	122	77.41	475.00	WELL #5
330	154	97.71		
342	26	16.50		
344	41	26.01		
345	5	3.17		
346	26	16.50		
350	57	36.17		
352	24	15.23		
354	52	32.99		
360	18	11.42		
365	21	13.32		
370	38	24.11		
380	72	45.68		
381	26	16.50		
382	13	8.25		
384	3	1.90		
400	69	43.78		
410	52	32.99		
420	94	59.64		
421	54	34.26		
422	47	29.82		
424	33	20.94		
428	104	65.99		
430	107	67.89		
431	63	39.97		
432		0.00	550.00	WELL #6
TOTAL=	3203	2,032.3	1855.00	

NOTE: 3770 DWELLING UNITS X 2.9 X 420 GPDPC X 75% = 2392 GPM FOR RESIDENTIAL

MODEL TWO
IMMEDIATE FUTURE (1999)
HOUSING DEVELOPMENT CURRENTLY PLANNED
FROM 3/6/98 APPROVED HOUSING LIST - BUILDING PERMITS ISSUED
DEMAND DISTRIBUTION

SHERWOOD - TOTAL DOMESTIC WATER DEMAND

PRESSURE SYSTEM			
JUNCTION NODE	DWELLING UNITS	DEMAND (GPM)	REMARKS
10	5	3.17	
11	28	17.77	
12	47	29.82	
15	6	3.81	
20	19	12.06	
25	8	5.08	
28	28	17.77	
30	54	34.26	
35	40	25.38	
40	23	14.59	
45	12	7.61	
48	13	8.25	
50	29	18.40	
52	15	9.52	
53	8	5.08	
54	28	17.77	
56	10	6.35	
57	8	5.08	
58	10	6.35	
60	26	16.50	
70	38	24.11	
80	22	13.96	
92	27	17.13	
103	63	39.97	
TOTAL=	567	359.8	

NOTE: 3770 DWELLING UNITS X 2.9 X 420 GPDPC X 75% = 2392 GPM FOR RESIDENTIAL

MODEL TWO
IMMEDIATE FUTURE (1999)
HOUSING DEVELOPMENT CURRENTLY PLANNED
FROM 3/6/98 APPROVED HOUSING LIST - BUILDING PERMITS ISSUED
DEMAND DISTRIBUTION

SHERWOOD - TOTAL COMMERCIAL - INDUSTRIAL WATER DEMAND

COMMERCIAL - INDUSTRIAL - INSTITUTIONAL

JUNCTION NODE	AREA ACRE	DEMAND (GPM)	REMARKS
120	8	3.07	
122	0	0.00	WELL #3
126	4	1.54	
128	1	0.38	
130	4	1.54	
140	10	3.84	
150	1	0.38	
160	1	0.38	
170	7	2.69	
171	11	4.22	
180	54	20.73	
182	16	6.14	
183	16	6.14	
184	86	33.01	
190	41	15.74	
192	41	15.74	
195	16	6.14	
220	3	1.15	
240		0.00	HIGH SCHOOL
250	16	6.14	WELL #4
310		0.00	ARCHER GLEN ELEMENTARY SCHOOL
320	0	0.00	WELL #5
410	7	2.69	
420	35	13.43	
430	35	13.43	
432	0	0.00	WELL #6
440	280	107.46	
TOTAL=	693	266.0	

MAJOR IRRIGATION USERS

JUNCTION NODE	DEMAND (GPM)	REMARKS
	531	SEE ATTACHMENT

NOTE: 3770 DWELLING UNITS X 2.9 X 420 GPDPC X 25% = 797 GPM FOR COMMERCIAL, INDUSTRIAL, INSTITUTIONAL & IRRIGATIONS.

MODEL TWO
IMMEDIATE FUTURE (1999)
HOUSING DEVELOPMENT CURRENTLY PLANNED
FROM 3/6/98 APPROVED HOUSING LIST - BUILDING PERMITS ISSUED
DEMAND DISTRIBUTION

Major Irrigation Distribution

J-240 = 200 gpm (High School)

Total demand for other major users = 331 gpm

Node	Demand	Remarks
J-160	49.65	15%
J-170	49.65	15%
J-180	82.75	25%
J-182	33.1	10%
J-342	6.62	2%
J-344	6.62	2%
J-350	16.55	5%
J-360	39.72	12%
J-380	6.62	2%
J-381	6.62	2%
J-440	33.1	10%
total	331	100%

MODEL THREE
NEAR-TERM FUTURE (CIRCA 2002)
ALL APPROVED HOUSING DEVELOPMENTS (3/98)
DEMAND DISTRIBUTION

SHERWOOD - TOTAL DOMESTIC WATER DEMAND

GRAVITY SYSTEM				
JUNCTION NODE	DWELLING UNITS	DEMAND (GPM)	INFLOW (GPM)	REMARKS
110	0	0.00		
120	55	34.90		
122	23	14.59	800.00	WELL #3
126	13	8.25		
130	9	5.71		
140	20	12.69		
160	152	96.44		
170	155	98.35		
171	146	92.64		MOBILE HOMES
172	363	230.32		
182	50	31.73		
183	50	31.73		
192	38	24.11		
193	129	81.85		
194	129	81.85		
195	118	74.87		
196	0	0.00		
200	33	20.94		
210	128	81.22		
220	20	12.69		
230	45	28.55		
231	29	18.40		
234	50	31.73		
235	91	57.74		
240	90	57.11		
250	96	60.91	325.00	WELL #4
260	68	43.15		
265	129	81.85		
280	60	38.07		
304	0	0.00		
306	32	20.30		
308	62	39.34		
310	54	34.26		
320	125	79.31	450.00	WELL #5
330	158	100.25		
331	21	13.32		
332	51	32.36		
334	18	11.42		
336	119	75.51		
340	164	104.06		
342	33	20.94		
344	42	26.65		
345	64	40.61		
346	29	18.40		
350	59	37.44		
352	58	36.80		
360	19	12.06		
365	22	13.96		
370	39	24.75		
380	74	46.95		
382	75	47.59		
384	70	44.42		
400	69	43.78		
410	52	32.99		
420	94	59.64		
421	54	34.26		
422	47	29.82		
428	104	65.99		
430	107	67.89	125.00	BULL RUN
431	66	41.88		
432	0	0.00	550.00	WELL #6
500	159	100.89		
TOTAL=	4429	2,810.20	2250.00	

NOTE: 5310 DWELLING UNITS X 2.9 X 420 GPDPC X 75% = 3369 GPM FOR RESIDENTIAL

B-10

MODEL THREE
NEAR-TERM FUTURE (CIRCA 2002)
ALL APPROVED HOUSING DEVELOPMENTS (3/98)
DEMAND DISTRIBUTION

SHERWOOD - TOTAL DOMESTIC WATER DEMAND

PRESSURE SYSTEM			
JUNCTION NODE	DWELLING UNITS	DEMAND	REMARKS
10	5	3.17	
11	28	17.77	
12	47	29.82	
15	6	3.81	
20	19	12.06	
25	8	5.08	
28	23	14.59	
30	54	34.26	
35	40	25.38	
37	38	24.11	
38	38	24.11	
40	23	14.59	
41	0	0.00	
45	12	7.61	
48	13	8.25	
50	29	18.40	
52	15	9.52	
53	8	5.08	
54	28	17.77	
56	10	6.35	
57	8	5.08	
58	10	6.35	
60	26	16.50	
70	38	24.11	
80	22	13.96	
92	27	17.13	
94	27	17.13	
102	44	27.92	
103	43	27.28	
107	33	20.94	
610	159	100.89	
612	0	0.00	
614	0	0.00	
TOTAL=	881	559.0	

NOTE: 5310 DWELLING UNITS X 2.9 X 420 GPDPC X 75% = 3369 GPM FOR RESIDENTIAL

MODEL THREE
NEAR-TERM FUTURE (CIRCA 2002)
ALL APPROVED HOUSING DEVELOPMENTS (3/98)
DEMAND DISTRIBUTION

SHERWOOD - TOTAL COMMERCIAL - INDUSTRIAL WATER DEMAND

COMMERCIAL - INDUSTRIAL - INSTITUTIONAL			
JUNCTION NODE	AREA ACRE	DEMAND (GPM)	REMARKS
120	8	3.99	
122	0	0.00	WELL #3
126	5	2.50	
130	4	2.00	
140	10	4.99	
150	1	0.50	
160	1	0.50	
170	7	3.50	
171	11	5.49	
180	54	26.96	
182	16	7.99	
183	16	7.99	
184	86	42.94	
190	41	20.47	
192	41	20.47	
195	16	7.99	
220	3	1.50	
240		0.00	HIGH SCHOOL IRRIGATION
250	16	7.99	WELL #4
260	7	3.50	
265	23	11.48	
310		0.00	ARCHER GLEN ELEM. SCHOOL IRRIG.
320	0	0.00	WELL #5
340	6	3.00	
370	10	4.99	
384	10	4.99	
410	7	3.50	
420	35	17.48	
430	35	17.48	
432	0	0.00	WELL #6
440	280	139.81	
TOTAL=	749	374.00	

MAJOR IRRIGATION USERS		
JUNCTION NODE	DEMAND (GPM)	REMARKS
	749	SEE ATTACHMENT

NOTE: 5310 DWELLING UNITS X 2.9 X 420 GPDPC X 25% = 1123 GPM FOR COMMERCIAL, INDUSTRIAL, INSTITUTIONAL & IRRIGATION.

MODEL THREE
NEAR-TERM FUTURE (CIRCA 2002)
ALL APPROVED HOUSING DEVELOPMENTS (3/98)
DEMAND DISTRIBUTION

Major Irrigation Distribution

J-240 = 200 gpm (High School)

Total demand for other major users = 549 gpm

Node	Demand	Remarks
J-160	82.35	15%
J-170	82.35	15%
J-180	137.25	25%
J-182	54.9	10%
J-342	10.98	2%
J-344	10.98	2%
J-350	27.45	5%
J-360	65.88	12%
J-380	21.96	4%
J-440	54.9	10%
total	549	100%

MODEL FOUR
FUTURE-TERM (CIRCA 2017)
URBAN GROWTH RESERVE
DEMAND DISTRIBUTION

SHERWOOD - TOTAL DOMESTIC WATER DEMAND

GRAVITY SYSTEM				
JUNCTION NODE	DWELLING UNITS	DEMAND (GPM)	INFLOW (GPM)	REMARKS
110	0	0.00		
120	56	31.17		
122	23	12.80	900.00	WELL #3
126	13	7.24		
130	9	5.01		
140	20	11.13		
160	152	84.60		
170	155	86.27		
171	266	148.05		MOBILE HOMES
172	363	202.03		
182	50	27.83		
183	50	27.83		
192	246	136.92		
193	200	111.31		
194	455	253.24		
196	300	166.97		
200	32	17.81		
210	128	71.24		
220	20	11.13		
230	45	25.05		
231	29	16.14		
234	50	27.83		
235	151	84.04		
240	150	83.48		
250	96	53.43	350.00	WELL #4
260	68	37.85		
265	479	266.59		
280	135	75.14		
306	32	17.81		
308	62	34.51		
310	54	30.05		
320	125	69.57	450.00	WELL #5
330	158	87.94		
331	21	11.69		
332	51	28.38		
334	18	10.02		
336	119	66.23		
340	164	91.28		
342	33	18.37		
344	42	23.38		
345	64	35.62		
346	29	16.14		
350	59	32.84		
352	474	263.81		
360	19	10.57		
365	22	12.24		
370	39	21.71		
380	74	41.19		
382	75	41.74		
384	158	87.94		
400	69	38.40		
410	52	28.94		
420	94	52.32		
421	54	30.05		
422	47	26.16		
428	104	57.88		
430	107	59.55		
431	66	36.73		
432	13	7.24	550.00	WELL #6
500	1378	766.95		URBAN GROWTH RESERVE
TOTAL=	7617	4239.36	2,250.00	

NOTE: 9069 DWELLING UNITS X 2.65 X 420 GPDPC X 72% = 5047 GPM FOR RESIDENTIAL

GRAVITY = 3504.64 GPM
URBAN RESERVE (GRAVITY) = 1986 GPM
COMMERCIAL, INDUSTRIAL, INSTITUTIONAL = 438 GPM

PRESSURE = 442.36 GPM
URBAN RESERVE (PRESSURE) = 1041 GPM

MODEL FOUR
FUTURE-TERM (CIRCA 2017)
URBAN GROWTH RESERVE
DEMAND DISTRIBUTION

SHERWOOD - TOTAL DOMESTIC WATER DEMAND

PRESSURE SYSTEM			
JUNCTION NODE	DWELLING UNITS	DEMAND	REMARKS
10	5	2.78	
11	28	15.58	
12	47	26.16	
15	6	3.34	
20	19	10.57	
25	8	4.45	
28	23	12.80	
30	54	30.05	
35	40	22.26	
36	41	22.82	
37	38	21.15	
38	38	21.15	
40	23	12.80	
45	12	6.68	
48	13	7.24	
50	29	16.14	
52	15	8.35	
53	8	4.45	
54	28	15.58	
56	10	5.57	
57	8	4.45	
58	10	5.57	
60	26	14.47	
70	38	21.15	
80	22	12.24	
92	27	15.03	
94	27	15.03	
102	44	24.49	
103	43	23.93	
107	33	18.37	
610	409	227.65	URBAN GROWTH RESERVE
612	140	77.92	URBAN GROWTH RESERVE
614	140	77.92	URBAN GROWTH RESERVE
TOTAL=	1452	808	

NOTE: 9069 DWELLING UNITS X 2.65 X 420 GPDPC X 72% = 5047 GPM FOR RESIDENTIAL

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DEC. 98 REVISION

MODEL FOUR
FUTURE-TERM (CIRCA 2017)
URBAN GROWTH RESERVE
DEMAND DISTRIBUTION

SHERWOOD - TOTAL COMMERCIAL - INDUSTRIAL WATER DEMAND

COMMERCIAL - INDUSTRIAL - INSTITUTIONAL			
JUNCTION NODE	AREA ACRE	DEMAND (GPM)	REMARKS
120	8	11.99	
122	0	0.00	WELL #3
126	4	6.00	
130	4	6.00	
140	10	14.99	
		0.00	
160	2	3.00	
170	7	10.49	
171	11	16.49	
180	54	80.94	
182	16	23.98	
183	16	23.98	
184	86	128.90	
190	41	61.45	
192	41	61.45	
195	16	23.98	
220	3	4.50	
240		0.00	HIGH SCHOOL
250	16	23.98	WELL #4
260	7	10.49	
265	23	34.47	
310		0.00	ARCHER GLEN ELEM. SCHOOL
320	0	0.00	WELL #5
340	6	8.99	
370	10	14.99	
382	10	14.99	
410	7	10.49	
420	35	52.46	
430	35	52.46	
432	0	0.00	WELL #6
440	280	419.69	
TOTAL=	748	1121	

MAJOR IRRIGATION USERS		
JUNCTION NODE	DEMAND (GPM)	REMARKS
	842	SEE ATTACHMENT

NOTE: 9069 DWELLING UNITS X 2.65 X 420 GPDPC X 28% = 1963 GPM FOR COMMERCIAL, INDUSTRIAL, INSTITUTIONAL & IRRIGATION.

MODEL FOUR
FUTURE-TERM (CIRCA 2017)
URBAN GROWTH RESERVE
DEMAND DISTRIBUTION

Major Irrigation Distribution

J-240 = 200 gpm (High School)

Total demand for other major users = 642 gpm

Node	Demand	Remarks
J-160	96.5	15%
J-170	96.5	15%
J-180	160.5	25%
J-182	64.0	10%
J-342	13.0	2%
J-344	13.0	2%
J-350	32.0	5%
J-360	77.0	12%
J-380	26.0	4%
J-440	64.0	10%
total	642.5	100%

CITY OF SHERWOOD
WATER SYSTEM MASTER PLAN UPDATE

APPENDIX C

NETWORK ANALYSIS MODELING RESULTS FOR BASE CONDITIONS

CITY OF SHERWOOD
MODEL ONE - GRAVITY ZONE

Scenario Summary Report

Base

Scenario Summary	
Demand Alternative	Peak Day
Physical Alternative	Base-Physical
Initial Settings Alternative	Base-Initial Settings
Operational Alternative	Base-Operational
Age Alternative	Base-Age Alternative
Constituent Alternative	Base-Constituent
Trace Alternative	Base-Trace Alternative
Fire Flow Alternative	Base-Fire Flow

Hydraulic Analysis Summary	
Analysis	Steady State
Friction Method	Hazen-Williams Formula
Accuracy	0.010000
Trials	25

Calibration			
Demand Operation	<none>	Roughness Operation	<none>
Demand	0.00	Roughness	0.00

GRAVITY ZONE
MODEL ONE - 1997
BASE RUN
NO FIRE FLOW
RESERVOIR @ 375'
ALL WELLS OFF

HYDRAULIC STATUS:

Hydraulic status for steady-state conditions

Balanced	Trials = 5, Accuracy = 0.00227
Flow Supplied	2,279.22 gpm
Flow Demanded	2,279.22 gpm
Flow Stored	0.00 gpm
R-1	Reservoir: Emptying

Scenario: Base
Steady State Analysis
Reservoir Report

Node Label	Reservoir Surface Elevation (ft)	Reservoir Inflow (gpm)	Calculated Hydraulic Grade (ft)
R-1	375	2,279.22	375.00

Scenario: Base
Steady State Analysis
Pipe Report

Link Label	Start Node	End Node	Length (ft)	Diameter (in)	Material	Roughness	Discharge (gpm)	End Calculated Hydraulic Grade (ft)	Headloss (ft)	Friction Slope (ft/1000ft)
P-100	R-1	J-110	450	16	Ductile Iron	100.0	2,279.22	372.73	2.27	5.05
P-113	J-120	J-110	920	8	Ductile Iron	100.0	-277.69	372.73	2.57	2.79
P-114	J-120	J-122	280	12	Ductile Iron	100.0	-40.47	370.16	0.31e-2	0.01
P-115	J-122	J-200	670	8	Ductile Iron	100.0	-275.99	372.01	1.85	2.76
P-116	J-400	J-120	970	12	Ductile Iron	100.0	-71.90	370.16	0.03	0.03
P-120	J-130	J-120	660	8	Ductile Iron	100.0	-209.24	370.16	1.11	1.69
P-122	J-130	J-122	610	8	Ductile Iron	100.0	-221.06	370.16	1.12	1.83
P-124	J-130	J-220	260	6	Ductile Iron	100.0	-131.90	369.79	0.74	2.86
P-126	J-126	J-130	370	8	Ductile Iron	100.0	-141.89	369.04	0.31	0.83
P-128	J-126	J-128	1,140	10	Ductile Iron	130.0	14.79	368.73	0.3e-2	0.26e-2
P-129	J-128	J-129	1,360	8	Ductile Iron	130.0	0.00	368.73	0.00	0.00
P-130	J-140	J-130	820	12	Ductile Iron	100.0	-413.45	369.04	0.66	0.81
P-132	J-220	J-140	850	8	Ductile Iron	100.0	202.56	368.38	1.41	1.65
P-134	J-140	J-126	610	8	Ductile Iron	100.0	-117.73	368.74	0.36	0.58
P-140	J-140	J-150	980	12	Ductile Iron	100.0	490.57	367.29	1.09	1.12
P-150	J-160	J-150	1,260	12	Ductile Iron	100.0	-490.27	367.29	1.40	1.11
P-160	J-160	J-170	780	12	Ductile Iron	100.0	360.21	365.40	0.49	0.63
P-170	J-170	J-180	1,190	12	Ductile Iron	100.0	150.71	365.25	0.15	0.13
P-172	J-170	J-171	1,190	6	Ductile Iron	100.0	17.04	365.32	0.08	0.07
P-174	J-172	J-170	1,840	10	Ductile Iron	130.0	-58.73	365.40	0.06	0.03
P-178	J-183	J-172	1,300	10	Ductile Iron	130.0	89.32	365.34	0.09	0.07
P-180	J-180	J-190	810	12	Ductile Iron	130.0	160.66	365.18	0.07	0.09
P-182	J-180	J-181	530	10	Ductile Iron	130.0	-81.92	365.28	0.03	0.06
P-183	J-181	J-182	1,040	10	Ductile Iron	130.0	-81.92	365.35	0.06	0.06
P-184	J-182	J-183	830	10	Ductile Iron	130.0	-109.15	365.43	0.09	0.10
P-185	J-183	J-184	1,760	10	Ductile Iron	130.0	-203.19	366.01	0.58	0.33
P-186	J-184	J-440	3,420	12	Ductile Iron	130.0	-228.63	366.58	0.57	0.17
P-190	J-190	J-192	910	10	Ductile Iron	130.0	148.53	365.01	0.17	0.18
P-192	J-192	J-193	2,910	10	Ductile Iron	130.0	25.20	364.99	0.02	0.01
P-194	J-192	J-194	1,390	10	Ductile Iron	130.0	92.93	364.90	0.11	0.08
P-196	J-194	J-195	2,180	10	Ductile Iron	130.0	51.98	364.84	0.06	0.03
P-200	J-110	J-200	370	14	Ductile Iron	130.0	1,292.07	372.01	0.72	1.93
P-210	J-200	J-210	960	14	Ductile Iron	130.0	995.30	370.86	1.15	1.19
P-220	J-220	J-210	770	12	Ductile Iron	100.0	-554.98	370.86	1.08	1.40
P-230	J-230	J-220	1,890	14	Ductile Iron	130.0	-207.03	369.79	0.12	0.07
P-232	J-230	J-344	2,260	12	Ductile Iron	130.0	-41.63	369.68	0.02	0.01
P-240	J-240	J-230	1,900	8	Ductile Iron	130.0	-248.67	369.66	2.66	1.40
P-242	J-240	J-140	2,100	10	Ductile Iron	100.0	-227.62	368.38	1.38	0.66
P-250	J-240	J-250	1,360	8	Ductile Iron	130.0	220.89	365.45	1.55	1.14
P-260	J-260	J-250	590	12	Ductile Iron	130.0	-155.68	365.45	0.05	0.08
P-262	J-171	J-260	1,450	10	Ductile Iron	130.0	-78.19	365.40	0.08	0.06
P-270	J-260	J-270	1,880	12	Ductile Iron	130.0	34.65	365.39	0.01	0.01
P-280	J-270	J-280	970	8	Ductile Iron	130.0	17.01	365.38	0.01	0.01
P-300	J-210	J-310	1,320	12	Ductile Iron	130.0	304.33	370.49	0.37	0.28
P-301	J-210	J-302	1,340	8	Ductile Iron	130.0	75.51	370.65	0.21	0.16
P-303	J-302	J-304	720	8	Ductile Iron	130.0	58.50	370.58	0.07	0.10
P-305	J-306	J-304	1,650	8	Ductile Iron	130.0	-41.49	370.58	0.09	0.05
P-307	J-308	J-306	750	12	Ductile Iron	130.0	-27.63	370.49	0.25e-2	0.33e-2
P-309	J-310	J-308	560	12	Ductile Iron	130.0	-7.47	370.49	0.15e-3	0.27e-3
P-310	J-310	J-320	1,210	12	Ductile Iron	130.0	272.77	370.21	0.28	0.23

C-5

Scenario: Base
Steady State Analysis
Pipe Report

Link Label	Start Node	End Node	Length (ft)	Diameter (in)	Material	Roughness	Discharge (gpm)	End Calculated Hydraulic Grade (ft)	Headloss (ft)	Friction Slope (ft/1000ft)
P-320	J-330	J-320	530	12	Ductile Iror	130.0	-245.68	370.21	0.10	0.19
P-330	J-330	J-340	2,220	12	Ductile Iror	130.0	217.33	369.77	0.34	0.15
P-340	J-340	J-350	1,870	12	Ductile Iror	130.0	116.88	369.68	0.09	0.05
P-342	J-342	J-340	620	10	Ductile Iror	130.0	-100.46	369.77	0.05	0.09
P-344	J-344	J-342	860	10	Ductile Iror	130.0	-70.53	369.72	0.04	0.05
P-346	J-344	J-346	530	10	Ductile Iror	130.0	5.94	369.68	0.24e-3	0.46e-3
P-350	J-350	J-360	740	12	Ductile Iror	130.0	32.80	369.68	0.32e-2	0.44e-2
P-352	J-370	J-350	880	8	Ductile Iror	130.0	-10.22	369.68	0.34e-2	0.38e-2
P-354	J-365	J-350	960	8	Ductile Iror	130.0	-10.57	369.68	0.13e-2	0.13e-2
P-356	J-350	J-352	680	8	Ductile Iror	130.0	29.61	369.67	0.02	0.03
P-358	J-352	J-353	830	6	Ductile Iror	130.0	20.16	369.62	0.05	0.05
P-359	J-353	J-354	670	6	Ductile Iror	130.0	20.16	369.58	0.04	0.05
P-360	J-370	J-360	530	6	Ductile Iror	130.0	-1.25	369.68	0.15e-3	0.29e-3
P-365	J-360	J-365	730	8	Ductile Iror	130.0	-2.38	369.68	0.2e-2	0.27e-2
P-366	J-370	J-342	1,880	8	Ductile Iror	130.0	-25.43	369.72	0.04	0.02
P-370	J-380	J-370	730	12	Ductile Iror	130.0	-22.41	369.68	0.17e-2	0.23e-2
P-375	J-344	J-380	1,100	8	Ductile Iror	130.0	4.59	369.68	0.95e-3	0.86e-3
P-380	J-381	J-380	580	12	Ductile Iror	130.0	1.44	369.68	0.31e-4	0.53e-4
P-382	J-346	J-381	480	8	Ductile Iror	130.0	5.94	369.68	0.67e-3	0.14e-2
P-400	J-110	J-400	1,160	12	Ductile Iror	100.0	709.45	370.13	2.60	2.24
P-405	J-400	J-421	1,080	12	Ductile Iror	130.0	655.11	368.87	1.26	1.17
P-410	J-410	J-400	1,180	6	Ductile Iror	100.0	-82.78	370.13	1.43	1.21
P-420	J-420	J-410	1,090	6	Ductile Iror	100.0	-47.95	368.70	0.48	0.44
P-421	J-421	J-420	1,460	12	Ductile Iror	130.0	383.38	368.22	0.65	0.44
P-422	J-421	J-422	550	8	Ductile Iror	130.0	237.71	368.15	0.72	1.30
P-423	J-422	J-431	860	8	Ductile Iror	130.0	109.36	367.89	0.27	0.31
P-424	J-422	J-424	860	8	Ductile Iror	130.0	98.74	367.93	0.22	0.26
P-425	J-424	J-428	690	8	Ductile Iror	130.0	77.95	367.82	0.11	0.16
P-426	J-428	J-426	960	8	Ductile Iror	130.0	16.71	367.81	0.01	0.01
P-427	J-426	J-428	960	8	Ductile Iror	130.0	-16.68	367.82	0.01	0.01
P-428	J-432	J-428	300	6	Ductile Iror	130.0	19.70	367.82	0.02	0.05
P-430	J-430	J-420	900	12	Ductile Iror	130.0	-361.75	368.22	0.35	0.39
P-431	J-431	J-430	1,570	12	Ductile Iror	130.0	49.97	367.87	0.02	0.01
P-432	J-431	J-432	1,050	6	Ductile Iror	130.0	19.70	367.83	0.05	0.05
P-440	J-440	J-430	3,840	12	Ductile Iror	130.0	-333.96	367.87	1.29	0.34

Scenario: Base
Steady State Analysis
Junction Report

Node Label	Elevation (ft)	Demand Type	Demand (gpm)	Demand Pattern	Calculated Demand (gpm)	Calculated Hydraulic Grade (ft)	Pressure (psi)
J-110	305	Demand	0.00	Fixed	0.00	372.73	29.29
J-120	195	Demand	37.02	Fixed	37.02	370.16	75.74
J-122	195	Demand	14.46	Fixed	14.46	370.16	75.75
J-126	195	Demand	9.37	Fixed	9.37	368.74	75.13
J-128	200	Demand	14.79	Fixed	14.79	368.73	72.97
J-129	205	Demand	0.00	Fixed	0.00	368.73	70.80
J-130	195	Demand	6.85	Fixed	6.85	369.04	75.26
J-140	190	Demand	15.56	Fixed	15.56	368.38	77.14
J-150	200	Demand	0.30	Fixed	0.30	367.29	72.34
J-160	200	Demand	130.06	Fixed	130.06	365.89	71.74
J-170	210	Demand	133.72	Fixed	133.72	365.40	67.20
J-171	190	Demand	95.23	Fixed	95.23	365.32	75.81
J-172	210	Demand	148.05	Fixed	148.05	365.34	67.17
J-180	210	Demand	71.97	Fixed	71.97	365.25	67.13
J-181	210	Demand	0.00	Fixed	0.00	365.28	67.15
J-182	205	Demand	27.23	Fixed	27.23	365.35	69.34
J-183	200	Demand	4.73	Fixed	4.73	365.43	71.54
J-184	150	Demand	25.44	Fixed	25.44	366.01	93.41
J-190	205	Demand	12.13	Fixed	12.13	365.18	69.27
J-192	195	Demand	30.40	Fixed	30.40	365.01	73.52
J-193	190	Demand	25.20	Fixed	25.20	364.99	75.67
J-194	190	Demand	40.95	Fixed	40.95	364.90	75.63
J-195	205	Demand	51.98	Fixed	51.98	364.84	69.12
J-200	275	Demand	20.79	Fixed	20.79	372.01	41.95
J-210	190	Demand	60.48	Fixed	60.48	370.86	78.21
J-220	190	Demand	13.49	Fixed	13.49	369.79	77.75
J-230	195	Demand	0.00	Fixed	0.00	369.68	75.53
J-240	210	Demand	255.40	Fixed	255.40	367.00	67.89
J-250	200	Demand	65.21	Fixed	65.21	365.45	71.55
J-260	192	Demand	42.84	Fixed	42.84	365.40	74.98
J-270	225	Demand	17.64	Fixed	17.64	365.39	60.71
J-280	200	Demand	17.01	Fixed	17.01	365.38	71.52
J-302	310	Demand	17.01	Fixed	17.01	370.65	26.23
J-304	270	Demand	17.01	Fixed	17.01	370.58	43.49
J-306	240	Demand	13.86	Fixed	13.86	370.49	56.43
J-308	215	Demand	20.16	Fixed	20.16	370.49	67.24
J-310	215	Demand	39.02	Fixed	39.02	370.49	67.24
J-320	190	Demand	27.09	Fixed	27.09	370.21	77.93
J-330	195	Demand	28.35	Fixed	28.35	370.11	75.72
J-340	225	Demand	0.00	Fixed	0.00	369.77	62.61
J-342	215	Demand	4.50	Fixed	4.50	369.72	66.91
J-344	220	Demand	18.36	Fixed	18.36	369.68	64.73
J-346	220	Demand	0.00	Fixed	0.00	369.68	64.73
J-350	245	Demand	33.68	Fixed	33.68	369.68	53.92
J-352	262	Demand	9.45	Fixed	9.45	369.67	46.56
J-353	240	Demand	0.00	Fixed	0.00	369.62	56.05
J-354	255	Demand	20.16	Fixed	20.16	369.58	49.55
J-360	270	Demand	33.93	Fixed	33.93	369.68	43.11
J-365	290	Demand	8.19	Fixed	8.19	369.68	34.46
J-370	250	Demand	14.49	Fixed	14.49	369.68	51.75

Scenario: Base
Steady State Analysis
Junction Report

Node Label	Elevation (ft)	Demand Type	Demand (gpm)	Demand Pattern	Calculated Demand (gpm)	Calculated Hydraulic Grade (ft)	Pressure (psi)
J-380	245	Demand	28.44	Fixed	28.44	369.68	53.92
J-381	235	Demand	4.50	Fixed	4.50	369.68	58.24
J-400	210	Demand	43.47	Fixed	43.47	370.13	69.24
J-410	195	Demand	34.83	Fixed	34.83	368.70	75.11
J-420	185	Demand	69.57	Fixed	69.57	368.22	79.23
J-421	245	Demand	34.02	Fixed	34.02	368.87	53.56
J-422	235	Demand	29.61	Fixed	29.61	368.15	57.58
J-424	302	Demand	20.79	Fixed	20.79	367.93	28.51
J-426	285	Demand	33.39	Fixed	33.39	367.81	35.81
J-428	257	Demand	64.26	Fixed	64.26	367.82	47.92
J-430	160	Demand	77.76	Fixed	77.76	367.87	89.89
J-431	220	Demand	39.69	Fixed	39.69	367.89	63.95
J-432	250	Demand	0.00	Fixed	0.00	367.83	50.95
J-440	185	Demand	105.33	Fixed	105.33	366.58	78.52

CITY OF SHERWOOD
MODEL ONE - PRESSURE ZONE

Scenario Summary Report

Base

Scenario Summary	
Demand Alternative	Base-Average Daily
Physical Alternative	Base-Physical
Initial Settings Alternative	Base-Initial Settings
Operational Alternative	Base-Operational
Age Alternative	Base-Age Alternative
Constituent Alternative	Base-Constituent
Trace Alternative	Base-Trace Alternative
Fire Flow Alternative	Base-Fire Flow

Hydraulic Analysis Summary	
Analysis	Steady State
Friction Method	Hazen-Williams Formula
Accuracy	0.010000
Trials	25

Calibration			
Demand Operation	<none>	Roughness Operation	<none>
Demand	0.00	Roughness	0.00

MODEL ONE - PRESSURE ZONE
BASE RUN
NO FIRE FLOW
ALL WELLS OFF
ONE BIG PUMP ON
RESERVOIR @ 375'
NO PRVs INCLUDED

HYDRAULIC STATUS:

Hydraulic status for steady-state conditions

Balanced	Trials = 5, Accuracy = 0.001642
Flow Supplied	278.46 gpm
Flow Demanded	278.46 gpm
Flow Stored	0.00 gpm
R-1	Reservoir: Emptying
PMP-1	Pump: On

Scenario: Base
Steady State Analysis
Reservoir Report

Node Label	Reservoir Surface Elevation (ft)	Reservoir Inflow (gpm)	Calculated Hydraulic Grade (ft)
R-1	375.00	-278.46	375.00

Scenario: Base
Steady State Analysis
Pump Report

Link Label	Shutoff Head (ft)	Shutoff Discharge (gpm)	Design Head (ft)	Design Discharge (gpm)	Maximum Operating Head (ft)	Maximum Operating Discharge (gpm)	Current Status	Start Calculated Hydraulic Grade (ft)	End Calculated Hydraulic Grade (ft)	Discharge (gpm)	Pump Head (ft)
PMP-1	163.68	0.00	128.59	1,063.07	0.00	2,126.14	On	375.00	536.89	278.46	161.89

Scenario: Base
Steady State Analysis
Pipe Report

Link Label	Start Node	End Node	Length (ft)	Diameter (in)	Material	Roughness	Discharge (gpm)	End Calculated Hydraulic Grade (ft)	Headloss (ft)	Friction Slope (ft/1000ft)
P-2	PMP-1-In	R-1	47	16	Ductile Iron	130.0	-278.46	375.00	0.28e-2	0.06
P-5	J-10	PMP-1-Ou	53	16	Ductile Iron	130.0	-278.46	536.89	0.31e-2	0.06
P-10	J-10	J-15	1,220	16	Ductile Iron	130.0	144.25	536.87	0.02	0.02
P-11	J-11	J-10	510	8	Ductile Iron	130.0	-47.25	536.89	0.03	0.06
P-12	J-12	J-11	990	8	Ductile Iron	130.0	-29.61	536.86	0.03	0.03
P-15	J-15	J-20	300	10	Ductile Iron	130.0	140.47	536.82	0.05	0.16
P-20	J-20	J-25	850	10	Ductile Iron	130.0	125.98	536.70	0.11	0.13
P-22	J-25	J-28	570	8	Ductile Iron	130.0	14.49	536.70	0.42e-2	0.01
P-25	J-30	J-25	530	8	Ductile Iron	130.0	-106.45	536.70	0.15	0.29
P-30	J-35	J-30	670	8	Ductile Iron	130.0	-57.42	536.55	0.06	0.09
P-32	J-30	J-70	690	10	Ductile Iron	130.0	15.01	536.55	0.18e-2	0.27e-2
P-35	J-40	J-35	520	8	Ductile Iron	130.0	-36.63	536.49	0.02	0.04
P-40	J-45	J-40	260	8	Ductile Iron	130.0	-22.14	536.47	0.42e-2	0.02
P-42	J-48	J-45	470	8	Ductile Iron	130.0	8.14	536.46	0.12e-2	0.26e-2
P-45	J-45	J-50	360	8	Ductile Iron	130.0	22.72	536.46	0.01	0.02
P-50	J-50	J-60	420	8	Ductile Iron	130.0	-28.31	536.47	0.01	0.03
P-52	J-50	J-52	1,570	8	Ductile Iron	130.0	34.65	536.40	0.06	0.04
P-53	J-52	J-53	660	8	Ductile Iron	130.0	3.78	536.40	0.43e-3	0.65e-3
P-54	J-52	J-54	250	8	Ductile Iron	130.0	24.57	536.39	0.49e-2	0.02
P-56	J-54	J-56	240	8	Ductile Iron	130.0	8.82	536.39	0.67e-3	0.28e-2
P-57	J-54	J-57	590	8	Ductile Iron	130.0	3.78	536.39	0.31e-3	0.52e-3
P-58	J-56	J-58	620	8	Ductile Iron	130.0	4.41	536.39	0.49e-3	0.79e-3
P-60	J-70	J-60	780	8	Ductile Iron	130.0	61.02	536.47	0.08	0.10
P-65	J-60	J-48	370	8	Ductile Iron	130.0	16.33	536.46	0.34e-2	0.01
P-70	J-80	J-70	730	8	Ductile Iron	130.0	69.95	536.55	0.10	0.13
P-80	J-90	J-80	1,290	8	Ductile Iron	130.0	83.81	536.65	0.24	0.19
P-90	J-10	J-90	250	16	Ductile Iron	130.0	83.81	536.89	0.16e-2	0.01

Scenario: Base
Steady State Analysis
Junction Report

Node Label	Elevation (ft)	Demand Type	Demand (gpm)	Demand Pattern	Calculated Demand (gpm)	Calculated Hydraulic Grade (ft)	Pressure (psi)
J-10	335.00	Demand	3.15	Fixed	3.15	536.89	87.30
J-11	315.00	Demand	17.64	Fixed	17.64	536.86	95.94
J-12	295.00	Demand	29.61	Fixed	29.61	536.83	104.57
J-15	385.00	Demand	3.78	Fixed	3.78	536.87	65.67
J-20	310.00	Demand	14.49	Fixed	14.49	536.82	98.08
J-25	365.00	Demand	5.04	Fixed	5.04	536.70	74.25
J-28	375.00	Demand	14.49	Fixed	14.49	536.70	69.92
J-30	325.00	Demand	34.02	Fixed	34.02	536.55	91.48
J-35	380.00	Demand	20.79	Fixed	20.79	536.49	67.67
J-40	375.00	Demand	14.49	Fixed	14.49	536.47	69.82
J-45	395.00	Demand	7.56	Fixed	7.56	536.46	61.17
J-48	405.00	Demand	8.19	Fixed	8.19	536.46	56.85
J-50	410.00	Demand	16.38	Fixed	16.38	536.46	54.68
J-52	360.00	Demand	6.30	Fixed	6.30	536.40	76.28
J-53	285.00	Demand	3.78	Fixed	3.78	536.40	108.71
J-54	315.00	Demand	11.97	Fixed	11.97	536.39	95.74
J-56	285.00	Demand	4.41	Fixed	4.41	536.39	108.71
J-57	250.00	Demand	3.78	Fixed	3.78	536.39	123.85
J-58	290.00	Demand	4.41	Fixed	4.41	536.39	106.55
J-60	420.00	Demand	16.38	Fixed	16.38	536.47	50.36
J-70	350.00	Demand	23.94	Fixed	23.94	536.55	80.67
J-80	300.00	Demand	13.86	Fixed	13.86	536.65	102.33
J-90	310.00	Demand	0.00	Fixed	0.00	536.89	98.11

CITY OF SHERWOOD
MODEL TWO - GRAVITY ZONE

Scenario Summary Report

BASE @ 375

Scenario Summary

Demand Alternative	BASE RUN
Physical Alternative	BASE WITH RESERVOIR @ 375
Initial Settings Alternative	Base-Initial Settings
Operational Alternative	Base-Operational
Age Alternative	Base-Age Alternative
Constituent Alternative	Base-Constituent
Trace Alternative	Base-Trace Alternative
Fire Flow Alternative	Base-Fire Flow

Hydraulic Analysis Summary

Analysis	Steady State
Friction Method	Hazen-Williams Formula
Accuracy	0.001000
Trials	40

Calibration

Demand Operation	<none>	Roughness Operation	<none>
Demand	0.00	Roughness	0.00

Created: 12/29/98

CITY OF SHERWOOD-MODEL TWO
GRAVITY ZONE
IMMEDIATE FUTURE (1999)

BASE RUN
* NO FIRE FLOW
* RESERVOIR AT 375
* ALL WELLS OFF

HYDRAULIC STATUS:

Hydraulic status for steady-state conditions

Balanced	Trials = 6, Accuracy = 0.000698
Flow Supplied	2,829.27 gpm
Flow Demanded	2,829.27 gpm
Flow Stored	0.00 gpm
R-1	Reservoir: Emptying

Scenario: BASE @ 375
Steady State Analysis
Reservoir Report

Node Label	Reservoir Surface Elevation (ft)	Reservoir Inflow (gpm)	Calculated Hydraulic Grade (ft)
R-1	375	2,829.27	375.00

  Resource Management Int'l Inc
37 Brookside Road Waterbury, CT 06708 USA (203) 755-1666

Scenario: BASE @ 375
Steady State Analysis
Pipe Report

Link Label	Start Node	End Node	Length (ft)	Diameter (in)	Material	Roughness	Discharge (gpm)	End Calculated Hydraulic Grade (ft)	Headloss (ft)	Friction Slope (ft/1000ft)
P-100	R-1	J-105	300	16	Ductile Iron	100.0	2,829.27	372.90	2.10	6.99
P-110	J-105	J-110	250	16	Ductile Iron	100.0	1,925.93	371.93	0.98	3.90
P-113	J-120	J-110	920	8	Ductile Iron	100.0	-303.00	371.93	3.02	3.28
P-114	J-120	J-122	280	12	Ductile Iron	100.0	24.14	368.91	0.12e-2	0.41e-2
P-115	J-122	J-200	670	8	Ductile Iron	100.0	-282.24	370.83	1.93	2.88
P-116	J-400	J-120	970	12	Ductile Iron	100.0	35.68	368.91	0.01	0.01
P-120	J-130	J-120	660	8	Ductile Iron	100.0	-276.57	368.91	1.87	2.83
P-122	J-130	J-122	610	8	Ductile Iron	100.0	-291.79	368.91	1.87	3.06
P-124	J-130	J-220	260	8	Ductile Iron	100.0	-136.06	367.24	0.19	0.75
P-126	J-126	J-130	370	8	Ductile Iron	100.0	-175.58	367.04	0.46	1.24
P-128	J-126	J-128	1,140	10	Ductile Iron	130.0	14.98	366.58	0.31e-2	0.27e-2
P-129	J-128	J-129	1,360	8	Ductile Iron	130.0	0.00	366.58	0.00	0.00
P-130	J-140	J-130	820	12	Ductile Iron	100.0	-521.59	367.04	1.02	1.24
P-132	J-220	J-140	850	8	Ductile Iron	100.0	187.10	366.02	1.21	1.43
P-134	J-140	J-126	610	8	Ductile Iron	100.0	-150.82	366.58	0.56	0.92
P-140	J-140	J-150	980	12	Ductile Iron	100.0	591.04	364.48	1.54	1.58
P-150	J-160	J-150	1,260	12	Ductile Iron	100.0	-590.66	364.48	1.97	1.57
P-160	J-160	J-170	780	12	Ductile Iron	100.0	400.36	361.91	0.60	0.77
P-165	J-160	J-171	1,870	6	Ductile Iron	130.0	43.82	362.08	0.43	0.23
P-170	J-170	J-180	1,190	12	Ductile Iron	100.0	227.93	361.59	0.32	0.27
P-172	J-170	J-171	1,190	6	Ductile Iron	100.0	-26.03	362.08	0.17	0.14
P-174	J-172	J-170	1,840	10	Ductile Iron	130.0	-47.79	361.91	0.04	0.02
P-178	J-183	J-172	1,300	10	Ductile Iron	130.0	101.32	361.87	0.12	0.09
P-180	J-180	J-190	810	12	Ductile Iron	130.0	247.00	361.42	0.16	0.20
P-182	J-180	J-182	1,740	10	Ductile Iron	130.0	-122.55	361.81	0.22	0.13
P-184	J-182	J-183	830	10	Ductile Iron	130.0	-161.79	361.98	0.18	0.21
P-185	J-183	J-184	1,760	10	Ductile Iron	130.0	-269.25	362.96	0.97	0.55
P-186	J-184	J-440	3,420	12	Ductile Iron	130.0	-302.26	363.91	0.95	0.28
P-190	J-190	J-192	910	10	Ductile Iron	130.0	231.26	361.04	0.38	0.42
P-192	J-192	J-193	2,910	10	Ductile Iron	130.0	69.16	360.92	0.13	0.04
P-194	J-192	J-194	1,390	10	Ductile Iron	130.0	127.96	360.85	0.20	0.14
P-196	J-194	J-195	2,180	10	Ductile Iron	130.0	77.20	360.73	0.12	0.05
P-200	J-110	J-200	370	14	Ductile Iron	130.0	1,622.93	370.83	1.09	2.95
P-210	J-200	J-210	960	14	Ductile Iron	130.0	1,319.75	368.90	1.93	2.01
P-220	J-220	J-210	770	12	Ductile Iron	100.0	-703.20	368.90	1.67	2.17
P-230	J-230	J-220	1,890	14	Ductile Iron	130.0	-366.19	367.24	0.36	0.19
P-231	J-230	J-231	370	14	Ductile Iron	130.0	266.78	366.84	0.04	0.11
P-232	J-230	J-345	2,500	12	Ductile Iron	130.0	99.41	366.79	0.09	0.04
P-240	J-231	J-240	1,530	8	Ductile Iron	130.0	266.78	364.36	2.48	1.62
P-242	J-240	J-140	2,100	10	Ductile Iron	100.0	-251.94	366.02	1.66	0.79
P-250	J-240	J-250	1,360	8	Ductile Iron	130.0	261.62	362.23	2.13	1.56
P-260	J-260	J-250	590	12	Ductile Iron	130.0	-194.57	362.23	0.07	0.12
P-262	J-171	J-260	1,450	10	Ductile Iron	130.0	-79.08	362.16	0.08	0.06
P-270	J-260	J-270	1,880	12	Ductile Iron	130.0	72.34	362.12	0.04	0.02
P-280	J-270	J-280	970	8	Ductile Iron	130.0	36.17	362.08	0.04	0.04
P-300	J-210	J-310	1,320	12	Ductile Iron	130.0	448.01	368.14	0.76	0.58
P-301	J-210	J-302	1,560	8	Ductile Iron	130.0	87.33	368.59	0.32	0.20
P-303	J-302	J-304	720	8	Ductile Iron	130.0	87.33	368.44	0.15	0.20
P-305	J-306	J-304	2,130	8	Ductile Iron	130.0	-70.20	368.44	0.29	0.14
P-307	J-308	J-306	750	12	Ductile Iron	130.0	-49.90	368.15	0.01	0.01

Scenario: BASE @ 375
Steady State Analysis
Pipe Report

Link Label	Start Node	End Node	Length (ft)	Diameter (in)	Material	Roughness	Discharge (gpm)	End Calculated Hydraulic Grade (ft)	Headloss (ft)	Friction Slope (ft/1000ft)
P-309	J-310	J-308	560	12	Ductile Iron	130.0	-12.46	368.14	0.43e-3	0.76e-3
P-310	J-310	J-320	1,210	12	Ductile Iron	130.0	426.20	367.50	0.64	0.53
P-320	J-330	J-320	530	12	Ductile Iron	130.0	-348.79	367.50	0.19	0.36
P-330	J-330	J-340	2,220	12	Ductile Iron	130.0	251.08	366.87	0.44	0.20
P-340	J-340	J-350	1,870	12	Ductile Iron	130.0	140.32	366.74	0.13	0.07
P-342	J-342	J-340	620	10	Ductile Iron	130.0	-110.76	366.87	0.07	0.11
P-344	J-344	J-342	860	10	Ductile Iron	130.0	-55.16	366.81	0.03	0.03
P-345	J-345	J-344	450	12	Ductile Iron	130.0	71.21	366.78	0.01	0.02
P-346	J-344	J-346	530	10	Ductile Iron	130.0	61.63	366.76	0.02	0.04
P-350	J-350	J-360	740	12	Ductile Iron	130.0	26.29	366.74	0.22e-2	0.3e-2
P-352	J-370	J-350	880	8	Ductile Iron	130.0	-3.59	366.74	0.34e-3	0.38e-3
P-354	J-365	J-350	800	8	Ductile Iron	130.0	-9.50	366.74	0.23e-2	0.28e-2
P-356	J-350	J-352	680	8	Ductile Iron	130.0	48.22	366.70	0.05	0.07
P-358	J-352	J-353	830	12	Ductile Iron	130.0	32.99	366.69	0.38e-2	0.46e-2
P-359	J-353	J-354	670	12	Ductile Iron	130.0	32.99	366.69	0.31e-2	0.46e-2
P-360	J-370	J-360	530	12	Ductile Iron	130.0	28.67	366.74	0.19e-2	0.36e-2
P-362	J-360	J-365	650	8	Ductile Iron	130.0	0.24	366.74	0.31e-4	0.47e-4
P-365	J-360	J-365	730	8	Ductile Iron	130.0	3.58	366.74	0.31e-4	0.42e-4
P-366	J-370	J-342	1,880	8	Ductile Iron	130.0	-32.48	366.81	0.06	0.03
P-370	J-380	J-370	730	12	Ductile Iron	130.0	16.71	366.74	0.95e-3	0.13e-2
P-375	J-344	J-380	1,100	8	Ductile Iron	130.0	32.10	366.75	0.04	0.03
P-380	J-381	J-380	580	12	Ductile Iron	130.0	36.90	366.75	0.33e-2	0.01
P-381	J-382	J-381	310	8	Ductile Iron	130.0	31.10	366.75	0.01	0.03
P-382	J-346	J-381	480	8	Ductile Iron	130.0	28.92	366.75	0.01	0.03
P-383	J-346	J-382	390	8	Ductile Iron	130.0	16.21	366.76	0.35e-2	0.01
P-384	J-384	J-382	770	8	Ductile Iron	130.0	23.14	366.76	0.01	0.02
P-385	J-384	J-345	890	8	Ductile Iron	130.0	-25.04	366.79	0.02	0.02
P-400	J-400	J-105	1,160	12	Ductile Iron	100.0	-903.34	372.90	3.98	3.43
P-405	J-400	J-421	1,080	12	Ductile Iron	130.0	732.11	367.37	1.55	1.43
P-410	J-410	J-400	1,180	6	Ductile Iron	100.0	-91.77	368.92	1.73	1.46
P-420	J-420	J-410	1,090	6	Ductile Iron	100.0	-56.09	367.19	0.64	0.59
P-421	J-421	J-420	1,460	12	Ductile Iron	130.0	436.10	366.55	0.82	0.56
P-422	J-421	J-422	550	8	Ductile Iron	130.0	261.74	366.51	0.86	1.56
P-423	J-422	J-431	860	8	Ductile Iron	130.0	129.52	366.15	0.30	0.42
P-424	J-422	J-424	860	8	Ductile Iron	130.0	102.40	366.28	0.24	0.28
P-425	J-424	J-428	690	8	Ductile Iron	130.0	81.46	366.15	0.12	0.18
P-428	J-432	J-428	300	10	Ductile Iron	130.0	-15.47	366.15	0.82e-3	0.27e-2
P-430	J-430	J-420	900	12	Ductile Iron	130.0	-419.12	366.55	0.46	0.51
P-431	J-431	J-430	1,570	12	Ductile Iron	130.0	105.02	366.09	0.06	0.04
P-432	J-431	J-432	1,050	12	Ductile Iron	130.0	-15.47	366.15	0.12e-2	0.11e-2
P-440	J-440	J-430	3,840	12	Ductile Iron	130.0	-442.82	366.09	2.18	0.57

Scenario: BASE @ 375
Steady State Analysis
Junction Report

Node Label	Elevation (ft)	Demand Type	Demand (gpm)	Demand Pattern	Calculated Demand (gpm)	Calculated Hydraulic Grade (ft)	Pressure (psi)
J-105	330	Demand	0.00	Fixed	0.00	372.90	18.55
J-110	305	Demand	0.00	Fixed	0.00	371.93	28.94
J-120	195	Demand	37.97	Fixed	37.97	368.91	75.20
J-122	195	Demand	14.59	Fixed	14.59	368.91	75.20
J-126	195	Demand	9.78	Fixed	9.78	366.58	74.20
J-128	200	Demand	14.98	Fixed	14.98	366.58	72.04
J-129	205	Demand	0.00	Fixed	0.00	366.58	69.87
J-130	195	Demand	7.25	Fixed	7.25	367.04	74.40
J-140	190	Demand	16.53	Fixed	16.53	366.02	76.12
J-150	200	Demand	0.38	Fixed	0.38	364.48	71.13
J-160	200	Demand	146.48	Fixed	146.48	362.51	70.27
J-170	210	Demand	150.68	Fixed	150.68	361.91	65.69
J-171	190	Demand	96.86	Fixed	96.86	362.08	74.41
J-172	210	Demand	149.11	Fixed	149.11	361.87	65.67
J-180	210	Demand	103.48	Fixed	103.48	361.59	65.55
J-182	205	Demand	39.24	Fixed	39.24	361.81	67.81
J-183	200	Demand	6.14	Fixed	6.14	361.98	70.05
J-184	150	Demand	33.01	Fixed	33.01	362.96	92.09
J-190	205	Demand	15.74	Fixed	15.74	361.42	67.64
J-192	195	Demand	34.14	Fixed	34.14	361.04	71.80
J-193	190	Demand	69.16	Fixed	69.16	360.92	73.91
J-194	190	Demand	50.76	Fixed	50.76	360.85	73.88
J-195	205	Demand	77.20	Fixed	77.20	360.73	67.34
J-200	275	Demand	20.94	Fixed	20.94	370.83	41.44
J-210	190	Demand	81.22	Fixed	81.22	368.90	77.36
J-220	190	Demand	13.84	Fixed	13.84	367.24	76.64
J-230	195	Demand	0.00	Fixed	0.00	366.88	74.33
J-231	195	Demand	0.00	Fixed	0.00	366.84	74.31
J-240	210	Demand	257.11	Fixed	257.11	364.36	66.75
J-250	200	Demand	67.05	Fixed	67.05	362.23	70.15
J-260	192	Demand	43.15	Fixed	43.15	362.16	73.58
J-270	225	Demand	36.17	Fixed	36.17	362.12	59.30
J-280	200	Demand	36.17	Fixed	36.17	362.08	70.09
J-302	285	Demand	0.00	Fixed	0.00	368.59	36.15
J-304	270	Demand	17.13	Fixed	17.13	368.44	42.57
J-306	240	Demand	20.30	Fixed	20.30	368.15	55.42
J-308	215	Demand	37.44	Fixed	37.44	368.14	66.22
J-310	215	Demand	34.26	Fixed	34.26	368.14	66.22
J-320	190	Demand	77.41	Fixed	77.41	367.50	76.76
J-330	195	Demand	97.71	Fixed	97.71	367.31	74.51
J-340	225	Demand	0.00	Fixed	0.00	366.87	61.35
J-342	215	Demand	23.12	Fixed	23.12	366.81	65.65
J-344	220	Demand	32.63	Fixed	32.63	366.78	63.47
J-345	210	Demand	3.17	Fixed	3.17	366.79	67.80
J-346	220	Demand	16.50	Fixed	16.50	366.76	63.46
J-350	245	Demand	52.72	Fixed	52.72	366.74	52.65
J-352	262	Demand	15.23	Fixed	15.23	366.70	45.27
J-353	240	Demand	0.00	Fixed	0.00	366.69	54.79
J-354	255	Demand	32.99	Fixed	32.99	366.69	48.30
J-360	270	Demand	51.14	Fixed	51.14	366.74	41.83

Scenario: BASE @ 375
Steady State Analysis
Junction Report

Node Label	Elevation (ft)	Demand Type	Demand (gpm)	Demand Pattern	Calculated Demand (gpm)	Calculated Hydraulic Grade (ft)	Pressure (psi)
J-365	290	Demand	13.32	Fixed	13.32	366.74	33.19
J-370	250	Demand	24.11	Fixed	24.11	366.74	50.48
J-380	245	Demand	52.30	Fixed	52.30	366.75	52.65
J-381	235	Demand	23.12	Fixed	23.12	366.75	56.97
J-382	230	Demand	8.25	Fixed	8.25	366.76	59.14
J-384	225	Demand	1.90	Fixed	1.90	366.77	61.31
J-400	210	Demand	43.78	Fixed	43.78	368.92	68.72
J-410	195	Demand	35.68	Fixed	35.68	367.19	74.46
J-420	185	Demand	73.08	Fixed	73.08	366.55	78.51
J-421	245	Demand	34.26	Fixed	34.26	367.37	52.92
J-422	235	Demand	29.82	Fixed	29.82	366.51	56.87
J-424	302	Demand	20.94	Fixed	20.94	366.28	27.80
J-428	257	Demand	65.99	Fixed	65.99	366.15	47.20
J-430	160	Demand	81.32	Fixed	81.32	366.09	89.12
J-431	220	Demand	39.97	Fixed	39.97	366.15	63.20
J-432	250	Demand	0.00	Fixed	0.00	366.15	50.23
J-440	185	Demand	140.56	Fixed	140.56	363.91	77.37

CITY OF SHERWOOD
MODEL FOUR - GRAVITY ZONE

Scenario Summary Report

CASE ONE - NO TRANSMISSION MAIN

4170 gpm INFLOW @ J-430

Scenario Summary

Demand Alternative	4170 IN @ J-430
Physical Alternative	Base-Physical
Initial Settings Alternative	Base-Initial Settings
Operational Alternative	Base-Operational
Age Alternative	Base-Age Alternative
Constituent Alternative	Base-Constituent
Trace Alternative	Base-Trace Alternative
Fire Flow Alternative	Base-Fire Flow

Hydraulic Analysis Summary

Analysis	Steady State
Friction Method	Hazen-Williams Formula
Accuracy	0.001000
Trials	25

Calibration

Demand Operation	<none>	Roughness Operation	<none>
Demand	0.00	Roughness	0.00

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CITY OF SHERWOOD-MODEL FOUR
GRAVITY ZONE
FUTURE - URBAN RESERVE (2017)
WATER SUPPLY FROM BULL RUN
BASE RUN
NO FIRE FLOW
ALL WELLS OFF
RESERVOIR AT 375'
P-100 & P-110 (24")
J-430 INFLOW = 4170 gpm (6 MGD)
NO TRANSMISSON MAIN TO RESERVOIR

HYDRAULIC STATUS:

Hydraulic status for steady-state conditions

Balanced	Trials = 5, Accuracy = 0.000108
Flow Supplied	6,082.95 gpm
Flow Demanded	6,082.95 gpm
Flow Stored	0.00 gpm
R-1	Reservoir: Emptying

Scenario: 4170 gpm INFLOW @ J-430
Steady State Analysis
Reservoir Report

Node Label	Reservoir Surface Elevation (ft)	Reservoir Inflow (gpm)	Calculated Hydraulic Grade (ft)
R-1	375	2,024.96	375.00

Scenario: 4170 gpm INFLOW @ J-430
Steady State Analysis
Pipe Report

Link Label	Start Node	End Node	Length (ft)	Diameter (in)	Material	Roughness	Discharge (gpm)	End Calculated Hydraulic Grade (ft)	Headloss (ft)	Friction Slope (ft/1000ft)
P-100	R-1	J-105	300	24	Ductile Iron	130.0	2,024.96	374.90	0.10	0.32
P-110	J-105	J-110	250	24	Ductile Iron	130.0	2,959.61	374.74	0.16	0.65
P-113	J-120	J-110	920	8	Ductile Iron	100.0	-351.58	374.74	3.97	4.32
P-114	J-120	J-122	280	12	Ductile Iron	100.0	297.41	370.65	0.12	0.44
P-115	J-122	J-200	670	8	Ductile Iron	100.0	-243.89	372.12	1.47	2.20
P-116	J-400	J-120	1,030	12	Ductile Iron	100.0	1,276.36	370.77	6.75	6.55
P-120	J-130	J-120	840	12	Ductile Iron	100.0	1,287.37	370.77	5.72	6.81
P-122	J-130	J-122	610	8	Ductile Iron	100.0	-528.51	370.65	5.60	9.18
P-124	J-130	J-220	260	8	Ductile Iron	100.0	418.37	363.50	1.55	5.96
P-126	J-126	J-130	530	8	Ductile Iron	100.0	-297.48	365.05	1.72	3.25
P-130	J-140	J-130	820	12	Ductile Iron	100.0	1,089.02	365.05	3.98	4.85
P-132	J-220	J-140	1,530	8	Ductile Iron	100.0	201.06	361.07	2.43	1.59
P-134	J-140	J-126	760	8	Ductile Iron	100.0	-284.25	363.32	2.26	2.97
P-140	J-160	J-140	2,240	12	Ductile Iron	130.0	1,135.02	361.07	7.22	3.23
P-160	J-160	J-170	780	12	Ductile Iron	100.0	728.90	352.03	1.81	2.33
P-165	J-171	J-160	2,170	8	Ductile Iron	130.0	-222.01	353.84	2.46	1.14
P-170	J-170	J-180	1,300	12	Ductile Iron	100.0	470.77	350.68	1.34	1.03
P-172	J-170	J-171	1,340	8	Ductile Iron	100.0	107.11	351.38	0.65	0.48
P-174	J-172	J-170	1,910	10	Ductile Iron	130.0	42.24	352.03	0.03	0.02
P-178	J-183	J-172	1,870	10	Ductile Iron	130.0	244.27	352.06	0.87	0.46
P-180	J-180	J-190	920	12	Ductile Iron	130.0	362.74	350.31	0.38	0.41
P-182	J-180	J-182	1,740	10	Ductile Iron	130.0	-133.42	350.94	0.26	0.15
P-184	J-182	J-183	830	10	Ductile Iron	130.0	-597.64	352.93	1.99	2.39
P-185	J-183	J-184	1,780	10	Ductile Iron	130.0	-893.72	361.98	9.05	5.08
P-186	J-184	J-440	3,420	12	Ductile Iron	130.0	1,022.62	371.07	9.09	2.66
P-190	J-190	J-192	930	10	Ductile Iron	130.0	301.29	349.68	0.63	0.68
P-192	J-192	J-193	2,910	10	Ductile Iron	130.0	7.15	349.68	0.19e-2	0.66e-3
P-194	J-192	J-194	1,770	10	Ductile Iron	130.0	95.77	349.53	0.15	0.08
P-195	J-194	J-196	1,220	10	Ductile Iron	130.0	-157.46	349.78	0.25	0.20
P-196	J-196	J-195	1,520	10	Ductile Iron	130.0	23.98	349.77	0.01	0.01
P-197	J-196	J-182	3,210	12	Ductile Iron	130.0	-348.42	350.94	1.16	0.36
P-200	J-110	J-200	370	14	Ductile Iron	130.0	2,608.03	372.12	2.63	7.09
P-210	J-200	J-210	960	14	Ductile Iron	130.0	2,346.33	366.52	5.60	5.83
P-220	J-220	J-210	780	12	Ductile Iron	100.0	-962.40	366.52	3.02	3.87
P-230	J-230	J-220	1,960	14	Ductile Iron	130.0	1,164.08	363.50	3.15	1.61
P-231	J-230	J-231	370	14	Ductile Iron	130.0	735.94	360.06	0.28	0.76
P-232	J-345	J-230	2,510	12	Ductile Iron	130.0	-403.09	360.34	1.19	0.48
P-233	J-231	J-345	2,030	8	Ductile Iron	130.0	134.39	359.15	0.91	0.45
P-234	J-234	J-231	470	14	Ductile Iron	130.0	-281.61	360.06	0.05	0.12
P-235	J-235	J-234	1,550	8	Ductile Iron	130.0	-126.22	360.01	0.62	0.40
P-236	J-234	J-235	1,520	8	Ductile Iron	130.0	127.56	359.39	0.62	0.41
P-238	J-384	J-235	630	8	Ductile Iron	130.0	-169.74	359.39	0.44	0.69
P-240	J-231	J-240	1,530	8	Ductile Iron	130.0	303.81	356.91	3.15	2.06
P-242	J-240	J-140	2,100	10	Ductile Iron	100.0	-413.20	361.07	4.15	1.98
P-250	J-240	J-250	1,360	8	Ductile Iron	130.0	441.52	351.30	5.61	4.12
P-260	J-260	J-250	630	12	Ductile Iron	130.0	-364.11	351.30	0.25	0.39
P-262	J-171	J-260	1,460	10	Ductile Iron	130.0	164.60	351.06	0.32	0.22
P-265	J-260	J-265	1,610	12	Ductile Iron	130.0	480.37	350.00	1.06	0.66
P-280	J-265	J-280	970	10	Ductile Iron	130.0	179.30	349.75	0.25	0.26
P-282	J-193	J-280	1,850	12	Ductile Iron	130.0	-104.16	349.75	0.07	0.04

Scenario: 4170 gpm INFLOW @ J-430
Steady State Analysis
Pipe Report

Link Label	Start Node	End Node	Length (ft)	Diameter (in)	Material	Roughness	Discharge (gpm)	End Calculated Hydraulic Grade (ft)	Headloss (ft)	Friction Slope (ft/1000ft)
P-300	J-210	J-310	1,320	12	Ductile Iron	130.0	1,102.96	362.48	4.04	3.06
P-301	J-210	J-304	1,580	8	Ductile Iron	130.0	209.72	364.90	1.61	1.02
P-305	J-304	J-306	2,800	8	Ductile Iron	130.0	209.72	362.04	2.86	1.02
P-307	J-308	J-306	750	12	Ductile Iron	130.0	318.46	362.04	0.23	0.31
P-309	J-310	J-308	560	12	Ductile Iron	130.0	352.97	362.27	0.21	0.37
P-310	J-310	J-320	1,210	12	Ductile Iron	130.0	719.94	360.80	1.68	1.39
P-320	J-330	J-320	530	12	Ductile Iron	130.0	-650.37	360.80	0.61	1.15
P-330	J-330	J-331	1,510	12	Ductile Iron	130.0	562.44	358.86	1.33	0.88
P-332	J-332	J-331	1,250	8	Ductile Iron	130.0	-155.44	358.86	0.73	0.59
P-333	J-331	J-340	710	12	Ductile Iron	130.0	395.31	358.53	0.33	0.46
P-334	J-340	J-334	410	8	Ductile Iron	130.0	203.52	358.14	0.40	0.97
P-335	J-334	J-332	360	8	Ductile Iron	130.0	33.24	358.12	0.01	0.03
P-336	J-336	J-332	1,370	8	Ductile Iron	130.0	-160.29	358.12	0.85	0.62
P-337	J-336	J-334	1,390	8	Ductile Iron	130.0	-160.27	358.14	0.86	0.62
P-340	J-340	J-350	1,890	12	Ductile Iron	130.0	189.61	358.31	0.22	0.12
P-342	J-342	J-340	620	10	Ductile Iron	130.0	98.10	358.53	0.05	0.08
P-344	J-344	J-342	880	10	Ductile Iron	130.0	187.97	358.58	0.25	0.28
P-345	J-345	J-344	450	12	Ductile Iron	130.0	463.00	358.83	0.32	0.71
P-346	J-344	J-346	530	10	Ductile Iron	130.0	136.61	358.75	0.08	0.16
P-350	J-350	J-360	740	12	Ductile Iron	130.0	-78.51	358.33	0.02	0.02
P-352	J-370	J-350	1,090	8	Ductile Iron	130.0	50.09	358.31	0.08	0.07
P-354	J-365	J-350	1,040	8	Ductile Iron	130.0	12.68	358.31	0.01	0.01
P-356	J-350	J-352	760	8	Ductile Iron	130.0	266.05	357.06	1.25	1.65
P-358	J-352	J-336	1,030	12	Ductile Iron	130.0	-254.33	357.27	0.22	0.21
P-360	J-370	J-360	530	12	Ductile Iron	130.0	191.00	358.33	0.06	0.12
P-362	J-360	J-365	650	6	Ductile Iron	130.0	10.77	358.32	0.01	0.02
P-365	J-360	J-365	1,580	8	Ductile Iron	130.0	14.14	358.32	0.01	0.01
P-366	J-370	J-342	2,020	8	Ductile Iron	130.0	-58.50	358.58	0.20	0.10
P-370	J-380	J-370	740	12	Ductile Iron	130.0	219.28	358.39	0.11	0.15
P-375	J-344	J-380	1,210	8	Ductile Iron	130.0	102.05	358.50	0.33	0.27
P-380	J-380	J-382	850	12	Ductile Iron	130.0	-184.41	358.60	0.10	0.11
P-383	J-346	J-382	410	8	Ductile Iron	130.0	120.47	358.60	0.15	0.37
P-384	J-382	J-384	960	8	Ductile Iron	130.0	-120.67	358.95	0.35	0.37
P-385	J-384	J-345	1,080	6	Ductile Iron	130.0	-38.86	359.15	0.20	0.18
P-400	J-400	J-105	1,160	12	Ductile Iron	130.0	934.65	374.90	2.61	2.25
P-405	J-400	J-421	1,080	12	Ductile Iron	130.0	1,372.34	382.46	4.95	4.58
P-410	J-410	J-400	1,250	12	Ductile Iron	100.0	877.06	377.52	4.08	3.27
P-420	J-420	J-410	1,160	12	Ductile Iron	100.0	916.49	381.60	4.11	3.55
P-421	J-421	J-420	1,590	12	Ductile Iron	130.0	-875.53	385.71	3.25	2.04
P-422	J-421	J-422	580	8	Ductile Iron	130.0	-526.86	385.76	3.29	5.68
P-423	J-422	J-431	890	8	Ductile Iron	130.0	-553.02	391.27	5.52	6.20
P-426	J-431	J-428	1,080	8	Ductile Iron	130.0	19.26	391.26	0.01	0.01
P-428	J-428	J-432	290	10	Ductile Iron	130.0	-38.62	391.26	0.44e-2	0.02
P-430	J-430	J-420	900	12	Ductile Iron	130.0	1,896.80	385.71	7.51	8.34
P-431	J-431	J-430	1,650	12	Ductile Iron	130.0	-654.87	393.22	1.95	1.18
P-432	J-431	J-432	1,050	12	Ductile Iron	130.0	45.86	391.26	0.01	0.01
P-440	J-440	J-430	4,050	12	Ductile Iron	130.0	1,506.31	393.22	22.15	5.47
P-500	J-306	J-500	7,380	12	Ductile Iron	130.0	510.37	356.61	5.43	0.74
P-502	J-500	J-352	2,150	12	Ductile Iron	130.0	-256.58	357.06	0.44	0.21

Scenario: 4170 gpm INFLOW @ J-430
Steady State Analysis
Junction Report

Node Label	Elevation (ft)	Demand Type	Demand (gpm)	Demand Pattern	Calculated Demand (gpm)	Calculated Hydraulic Grade (ft)	Pressure (psi)
J-105	330	Demand	0.00	Fixed	0.00	374.90	19.42
J-110	305	Demand	0.00	Fixed	0.00	374.74	30.16
J-120	195	Demand	43.16	Fixed	43.16	370.77	76.01
J-122	195	Demand	12.80	Fixed	12.80	370.65	75.95
J-126	195	Demand	13.23	Fixed	13.23	363.32	72.79
J-130	195	Demand	11.00	Fixed	11.00	365.05	73.53
J-140	190	Demand	26.12	Fixed	26.12	361.07	73.97
J-160	200	Demand	184.10	Fixed	184.10	353.84	66.53
J-170	210	Demand	193.26	Fixed	193.26	352.03	61.42
J-171	190	Demand	164.53	Fixed	164.53	351.38	69.79
J-172	210	Demand	202.03	Fixed	202.03	352.06	61.43
J-180	210	Demand	241.44	Fixed	241.44	350.68	60.84
J-182	205	Demand	115.81	Fixed	115.81	350.94	63.11
J-183	200	Demand	51.81	Fixed	51.81	352.93	66.13
J-184	150	Demand	128.90	Fixed	128.90	361.98	91.67
J-190	205	Demand	61.45	Fixed	61.45	350.31	62.84
J-192	195	Demand	198.37	Fixed	198.37	349.68	66.89
J-193	190	Demand	111.31	Fixed	111.31	349.68	69.05
J-194	190	Demand	253.24	Fixed	253.24	349.53	68.99
J-195	205	Demand	23.98	Fixed	23.98	349.77	62.60
J-196	200	Demand	166.97	Fixed	166.97	349.78	64.77
J-200	275	Demand	17.81	Fixed	17.81	372.12	42.00
J-210	190	Demand	71.24	Fixed	71.24	366.52	76.33
J-220	190	Demand	15.63	Fixed	15.63	363.50	75.03
J-230	195	Demand	25.05	Fixed	25.05	360.34	71.50
J-231	200	Demand	16.14	Fixed	16.14	360.06	69.22
J-234	210	Demand	27.83	Fixed	27.83	360.01	64.87
J-235	225	Demand	84.04	Fixed	84.04	359.39	58.11
J-240	210	Demand	275.48	Fixed	275.48	356.91	63.53
J-250	200	Demand	77.41	Fixed	77.41	351.30	65.43
J-260	192	Demand	48.34	Fixed	48.34	351.06	68.78
J-265	205	Demand	301.07	Fixed	301.07	350.00	62.70
J-280	195	Demand	75.14	Fixed	75.14	349.75	66.92
J-304	285	Demand	0.00	Fixed	0.00	364.90	34.55
J-306	240	Demand	17.81	Fixed	17.81	362.04	52.77
J-308	215	Demand	34.51	Fixed	34.51	362.27	63.68
J-310	215	Demand	30.05	Fixed	30.05	362.48	63.77
J-320	190	Demand	69.57	Fixed	69.57	360.80	73.86
J-330	195	Demand	87.94	Fixed	87.94	360.19	71.43
J-331	210	Demand	11.69	Fixed	11.69	358.86	64.37
J-332	230	Demand	28.38	Fixed	28.38	358.12	55.41
J-334	235	Demand	10.02	Fixed	10.02	358.14	53.25
J-336	255	Demand	66.23	Fixed	66.23	357.27	44.23
J-340	225	Demand	100.27	Fixed	100.27	358.53	57.74
J-342	215	Demand	31.37	Fixed	31.37	358.58	62.09
J-344	220	Demand	36.37	Fixed	36.37	358.83	60.04
J-345	210	Demand	35.62	Fixed	35.62	359.15	64.50
J-346	220	Demand	16.14	Fixed	16.14	358.75	60.00
J-350	245	Demand	64.84	Fixed	64.84	358.31	49.00
J-352	262	Demand	263.81	Fixed	263.81	357.06	41.11

Scenario: 4170 gpm INFLOW @ J-430
Steady State Analysis
Junction Report

Node Label	Elevation (ft)	Demand Type	Demand (gpm)	Demand Pattern	Calculated Demand (gpm)	Calculated Hydraulic Grade (ft)	Pressure (psi)
J-360	270	Demand	87.57	Fixed	87.57	358.33	38.20
J-365	290	Demand	12.24	Fixed	12.24	358.32	29.54
J-370	250	Demand	36.69	Fixed	36.69	358.39	46.87
J-380	245	Demand	67.18	Fixed	67.18	358.50	49.08
J-382	230	Demand	56.73	Fixed	56.73	358.60	55.61
J-384	230	Demand	87.94	Fixed	87.94	358.95	55.76
J-400	210	Demand	38.40	Fixed	38.40	377.52	72.44
J-410	195	Demand	39.43	Fixed	39.43	381.60	80.69
J-420	185	Demand	104.78	Fixed	104.78	385.71	86.80
J-421	245	Demand	30.05	Fixed	30.05	382.46	59.44
J-422	235	Demand	26.16	Fixed	26.16	385.76	65.19
J-428	255	Demand	57.88	Fixed	57.88	391.26	58.92
J-430	160	Inflow	4,057.99	Fixed	-4,057.99	393.22	100.85
J-431	220	Demand	36.73	Fixed	36.73	391.27	74.06
J-432	250	Demand	7.24	Fixed	7.24	391.26	61.09
J-440	185	Demand	483.69	Fixed	483.69	371.07	80.46
J-500	210	Demand	766.95	Fixed	766.95	356.61	63.40

CITY OF SHERWOOD

Scenario Summary Report

MODEL TWO

PRESSURE ZONE

Scenario Summary

Demand Alternative	Demand-12-98 REVISED
Physical Alternative	Base-Physical
Initial Settings Alternative	Base-Initial Settings
Operational Alternative	Base-Operational
Age Alternative	Base-Age Alternative
Constituent Alternative	Base-Constituent
Trace Alternative	Base-Trace Alternative
Fire Flow Alternative	Base-Fire Flow

Hydraulic Analysis Summary

Analysis	Steady State
Friction Method	Hazen-Williams Formula
Accuracy	0.001000
Trials	40

Calibration

Demand Operation	<none>	Roughness Operation	<none>
Demand	0.00	Roughness	0.00

Created: 12/29/98 02:41:18 PM

CITY OF SHERWOOD-MODEL TWO
PRESSURE ZONE
IMMEDIATE FUTURE (1999)

NO FIRE FLOW
RESERVOIR @ 375'
SMALL PUMP
W/ 12" IMPELLER

P-70 = 8"
P-80 = 8"

HYDRAULIC STATUS:

Hydraulic status for steady-state conditions

Balanced	Trials = 5, Accuracy = 0.000606
Flow Supplied	359.79 gpm
Flow Demanded	359.79 gpm
Flow Stored	0.00 gpm
R-1	Reservoir: Emptying
PMP-1	Pump: On
PRV-1	PRV: Active, Setting = 49.77 psi
PRV-2	PRV: Active, Setting = 54.10 psi
PRV-3	PRV: Active, Setting = 64.92 psi
PRV-4	PRV: Active, Setting = 49.77 psi

Scenario: PRESSURE ZONE, 12/98
Steady State Analysis
Reservoir Report

Node Label	Reservoir Surface Elevation (ft)	Reservoir Inflow (gpm)	Calculated Hydraulic Grade (ft)
R-1	375	-359.79	375.00

Scenario: PRESSURE ZONE, 12/98
Steady State Analysis
Valve Report

Link Label	End Elevation (ft)	Diameter (in)	Minor Loss	Initial Grade Setting (ft)	Initial Status	Current Status	Discharge (gpm)	End Calculated Hydraulic Grade (ft)	Headloss (ft)
PRV-1	310	8	0.00	425.00	Active	Throttling	5.08	425.10	93.78
PRV-2	305	8	0.00	430.00	Active	Throttling	12.70	430.11	88.77
PRV-3	275	8	0.00	425.00	Active	Throttling	39.97	425.13	93.99
PRV-4	305	6	0.00	420.00	Active	Throttling	29.82	420.10	99.61

Scenario: PRESSURE ZONE, 12/98
Steady State Analysis
Pump Report

Link Label	Shutoff Head (ft)	Shutoff Discharge (gpm)	Design Head (ft)	Design Discharge (gpm)	Maximum Operating Head (ft)	Maximum Operating Discharge (gpm)	Current Status	Start Calculated Hydraulic Grade (ft)	End Calculated Hydraulic Grade (ft)	Discharge (gpm)	Pump Head (ft)
PMP-1	148.72	0.00	134.93	518.22	0.00	1,036.45	On	374.99	519.77	359.79	144.78

Scenario: PRESSURE ZONE, 12/98
Steady State Analysis
Pipe Report

Link Label	Start Node	End Node	Length (ft)	Diameter (in)	Material	Roughness	Discharge (gpm)	End Calculated Hydraulic Grade (ft)	Headloss (ft)	Friction Slope (ft/1000ft)
P-2	PMP-1-In	R-1	80	16	Ductile Iron	130.0	-359.79	375.00	0.01	0.09
P-5	J-10	PMP-1-Ou	80	16	Ductile Iron	130.0	-359.79	519.77	0.01	0.09
P-10	J-10	J-15	1,020	16	Ductile Iron	130.0	178.21	519.73	0.03	0.03
P-11	J-11	J-10	720	8	Ductile Iron	130.0	-47.59	519.76	0.05	0.07
P-11A	J-11	PRV-4-In	70	6	Ductile Iron	130.0	29.82	519.71	0.01	0.11
P-11B	PRV-4-Ou	J-11A	210	6	Ductile Iron	130.0	29.82	420.08	0.02	0.11
P-12	J-11A	J-12	940	8	Ductile Iron	130.0	29.82	420.05	0.03	0.03
P-15	J-15	J-20	500	10	Ductile Iron	130.0	174.40	519.61	0.12	0.25
P-20	J-20	J-25	980	10	Ductile Iron	130.0	162.34	519.40	0.21	0.21
P-25	J-30	J-25	550	8	Ductile Iron	130.0	-139.49	519.40	0.26	0.48
P-27	J-25	J-28	570	8	Ductile Iron	130.0	17.77	519.40	0.01	0.01
P-30	J-35	J-30	660	8	Ductile Iron	130.0	-68.25	519.14	0.08	0.13
P-31	J-30	J-31	500	10	Ductile Iron	130.0	39.97	519.13	0.01	0.02
P-32	J-30	J-70	690	10	Ductile Iron	130.0	-2.99	519.14	0.12e-3	0.18e-3
P-33	J-31	J-32	470	12	Ductile Iron	130.0	39.97	519.13	0.31e-2	0.01
P-34	J-32	J-36	750	12	Ductile Iron	130.0	39.97	519.12	0.49e-2	0.01
P-35	J-40	J-35	530	8	Ductile Iron	130.0	-42.87	519.05	0.03	0.05
P-36A	J-36	PRV-3-In	80	8	Ductile Iron	130.0	39.97	519.12	0.38e-2	0.05
P-36B	PRV-3-Ou	J-101	100	8	Ductile Iron	130.0	39.97	425.13	0.48e-2	0.05
P-40	J-45	J-40	260	8	Ductile Iron	130.0	-28.28	519.02	0.01	0.03
P-42	J-48	J-45	470	8	Ductile Iron	130.0	10.53	519.02	0.19e-2	0.4e-2
P-45	J-45	J-50	360	8	Ductile Iron	130.0	31.21	519.01	0.01	0.03
P-50	J-50	J-60	410	8	Ductile Iron	130.0	-37.34	519.02	0.02	0.04
P-52	J-50	J-52	1,570	8	Ductile Iron	130.0	50.15	518.89	0.11	0.07
P-53	J-52	J-53	690	8	Ductile Iron	130.0	5.08	518.89	0.67e-3	0.97e-3
P-54	J-52	J-54	250	8	Ductile Iron	130.0	35.55	518.88	0.01	0.04
P-56	PRV-2-Ou	J-56	140	8	Ductile Iron	130.0	12.70	430.11	0.79e-3	0.01
P-56A	J-54	PRV-2-In	100	8	Ductile Iron	130.0	12.70	518.88	0.61e-3	0.01
P-57	PRV-1-Ou	J-57	600	8	Ductile Iron	130.0	5.08	425.10	0.61e-3	0.1e-2
P-57A	J-54	PRV-1-In	40	8	Ductile Iron	130.0	5.08	518.88	0.61e-4	0.15e-2
P-58	J-56	J-58	630	8	Ductile Iron	130.0	6.35	430.11	0.98e-3	0.16e-2
P-60	J-70	J-60	790	8	Ductile Iron	130.0	72.63	519.02	0.11	0.14
P-65	J-60	J-48	380	8	Ductile Iron	130.0	18.78	519.02	0.45e-2	0.01
P-70	J-80	J-70	730	8	Ductile Iron	130.0	99.73	519.14	0.19	0.26
P-80	J-90	J-80	1,310	8	Ductile Iron	130.0	113.69	519.33	0.43	0.33
P-90	J-10	J-90	250	16	Ductile Iron	130.0	130.82	519.76	0.37e-2	0.01
P-92	J-90	J-92	1,310	8	Ductile Iron	130.0	17.13	519.74	0.01	0.01
P-100	J-101	J-100	549	8	Ductile Iron	130.0	0.00	425.13	0.00	0.00
P-101	J-102	J-101	351	8	Ductile Iron	130.0	-39.97	425.13	0.02	0.05
P-103	J-103	J-102	520	8	Ductile Iron	130.0	-39.97	425.11	0.02	0.05
P-104	J-104	J-103	840	8	Ductile Iron	130.0	0.00	425.08	0.00	0.00

Scenario: PRESSURE ZONE, 12/98
Steady State Analysis
Junction Report

Node Label	Elevation (ft)	Demand Type	Demand (gpm)	Demand Pattern	Calculated Demand (gpm)	Calculated Hydraulic Grade (ft)	Pressure (psi)
J-10	335	Demand	3.17	Fixed	3.17	519.76	79.90
J-11	315	Demand	17.77	Fixed	17.77	519.71	88.53
J-11A	280	Demand	0.00	Fixed	0.00	420.08	60.57
J-12	295	Demand	29.82	Fixed	29.82	420.05	54.08
J-15	385	Demand	3.81	Fixed	3.81	519.73	58.26
J-20	310	Demand	12.06	Fixed	12.06	519.61	90.64
J-25	365	Demand	5.08	Fixed	5.08	519.40	66.77
J-28	375	Demand	17.77	Fixed	17.77	519.40	62.44
J-30	325	Demand	34.26	Fixed	34.26	519.14	83.95
J-31	330	Demand	0.00	Fixed	0.00	519.13	81.79
J-32	280	Demand	0.00	Fixed	0.00	519.13	103.41
J-35	380	Demand	25.38	Fixed	25.38	519.05	60.13
J-36	275	Demand	0.00	Fixed	0.00	519.12	105.57
J-40	375	Demand	14.59	Fixed	14.59	519.02	62.28
J-45	395	Demand	7.61	Fixed	7.61	519.02	53.63
J-48	405	Demand	8.25	Fixed	8.25	519.02	49.31
J-50	410	Demand	18.40	Fixed	18.40	519.01	47.14
J-52	360	Demand	9.52	Fixed	9.52	518.89	68.71
J-53	285	Demand	5.08	Fixed	5.08	518.89	101.14
J-54	315	Demand	17.77	Fixed	17.77	518.88	88.17
J-56	285	Demand	6.35	Fixed	6.35	430.11	62.75
J-57	250	Demand	5.08	Fixed	5.08	425.10	75.72
J-58	290	Demand	6.35	Fixed	6.35	430.11	60.59
J-60	420	Demand	16.50	Fixed	16.50	519.02	42.82
J-70	350	Demand	24.11	Fixed	24.11	519.14	73.14
J-80	300	Demand	13.96	Fixed	13.96	519.33	94.84
J-90	310	Demand	0.00	Fixed	0.00	519.76	90.71
J-92	310	Demand	17.13	Fixed	17.13	519.74	90.70
J-100	255	Demand	0.00	Fixed	0.00	425.13	73.57
J-101	275	Demand	0.00	Fixed	0.00	425.13	64.92
J-102	280	Demand	0.00	Fixed	0.00	425.11	62.75
J-103	285	Demand	39.97	Fixed	39.97	425.08	60.58
J-104	285	Demand	0.00	Fixed	0.00	425.08	60.58

CITY OF SHERWOOD
MODEL THREE - GRAVITY ZONE

Scenario Summary Report

Base

Scenario Summary

Demand Alternative	Peak Day
Physical Alternative	Base-Physical
Initial Settings Alternative	Base-Initial Settings
Operational Alternative	Base-Operational
Age Alternative	Base-Age Alternative
Constituent Alternative	Base-Constituent
Trace Alternative	Base-Trace Alternative
Fire Flow Alternative	Base-Fire Flow

Hydraulic Analysis Summary

Analysis	Steady State
Friction Method	Hazen-Williams Formula
Accuracy	0.001000
Trials	25

Calibration

Demand Operation	<none>	Roughness Operation	<none>
Demand	0.00	Roughness	0.00

Created 12/98, DAG

CITY OF SHERWOOD-MODEL THREE
GRAVITY ZONE
NEAR-TERM (2002)

BASE RUN
NO FIRE FLOW
ALL WELLS OFF
RESERVOIR AT 375'
BULL RUN @ 125 GPM (J-430)

HYDRAULIC STATUS:

Hydraulic status for steady-state conditions

Balanced	Trials = 6, Accuracy = 0.000441
Flow Supplied	3,847.82 gpm
Flow Demanded	3,847.82 gpm
Flow Stored	0.00 gpm
R-1	Reservoir: Emptying

Scenario: Base
Steady State Analysis
Reservoir Report

Node Label	Reservoir Surface Elevation (ft)	Reservoir Inflow (gpm)	Calculated Hydraulic Grade (ft)
R-1	375	3,808.19	375.00

Scenario: Base
Steady State Analysis
Pipe Report

Link Label	Start Node	End Node	Length (ft)	Diameter (in)	Material	Roughness	Discharge (gpm)	End Calculated Hydraulic Grade (ft)	Headloss (ft)	Friction Slope (ft/1000ft)
P-100	R-1	J-105	300	16	Ductile Iron	100.0	3,808.19	371.36	3.64	12.12
P-110	J-105	J-110	250	16	Ductile Iron	100.0	2,459.67	370.01	1.35	5.40
P-113	J-120	J-110	920	8	Ductile Iron	100.0	-379.63	370.01	4.58	4.98
P-114	J-120	J-122	280	12	Ductile Iron	100.0	-19.40	365.44	0.79e-3	0.28e-2
P-115	J-122	J-200	670	8	Ductile Iron	100.0	-348.78	368.29	2.85	4.25
P-116	J-400	J-120	1,030	12	Ductile Iron	100.0	398.44	365.44	0.78	0.76
P-120	J-130	J-120	840	12	Ductile Iron	100.0	-758.58	365.44	2.15	2.55
P-122	J-130	J-122	610	8	Ductile Iron	100.0	-314.79	365.44	2.15	3.52
P-124	J-130	J-220	260	8	Ductile Iron	100.0	119.27	363.14	0.15	0.58
P-126	J-126	J-130	530	8	Ductile Iron	100.0	-203.93	363.29	0.86	1.62
P-130	J-140	J-130	820	12	Ductile Iron	100.0	-742.45	363.29	1.96	2.39
P-132	J-220	J-140	1,530	8	Ductile Iron	100.0	171.36	361.33	1.81	1.18
P-134	J-140	J-126	760	8	Ductile Iron	100.0	-193.18	362.43	1.10	1.45
P-140	J-140	J-150	980	12	Ductile Iron	100.0	770.63	358.81	2.52	2.58
P-150	J 160	J-150	1,260	12	Ductile Iron	100.0	-770.13	358.81	3.22	2.56
P-160	J-160	J-170	780	12	Ductile Iron	100.0	473.90	354.77	0.82	1.05
P-165	J-171	J-160	2,170	8	Ductile Iron	130.0	-116.94	355.58	0.75	0.35
P-170	J-170	J-180	1,300	12	Ductile Iron	100.0	220.90	354.44	0.33	0.25
P-172	J-170	J-171	1,340	8	Ductile Iron	100.0	-31.00	354.83	0.07	0.05
P-174	J-172	J-170	1,910	10	Ductile Iron	130.0	-99.82	354.77	0.17	0.09
P-178	J-183	J-172	1,870	10	Ductile Iron	130.0	130.50	354.60	0.27	0.15
P-180	J-180	J-190	920	12	Ductile Iron	130.0	99.63	354.40	0.03	0.04
P-182	J-180	J-182	1,740	10	Ductile Iron	130.0	-42.95	354.47	0.03	0.02
P-184	J-182	J-183	830	10	Ductile Iron	130.0	-252.51	354.87	0.40	0.49
P-185	J-183	J-184	1,780	10	Ductile Iron	130.0	-422.72	357.13	2.26	1.27
P-186	J-184	J-440	3,420	12	Ductile Iron	130.0	465.66	359.20	2.12	0.62
P-190	J-190	J-192	930	10	Ductile Iron	130.0	79.16	354.35	0.05	0.06
P-192	J-192	J-193	2,910	10	Ductile Iron	130.0	-15.18	354.36	0.01	0.27e-2
P-194	J-192	J-194	1,770	10	Ductile Iron	130.0	49.76	354.30	0.04	0.02
P-195	J-194	J-196	1,220	10	Ductile Iron	130.0	-32.09	354.32	0.01	0.01
P-196	J-196	J-195	1,520	10	Ductile Iron	130.0	82.86	354.22	0.09	0.06
P-197	J-196	J-182	3,210	12	Ductile Iron	130.0	-114.95	354.47	0.15	0.05
P-200	J-110	J-200	370	14	Ductile Iron	130.0	2,080.04	368.29	1.73	4.67
P-210	J-200	J-210	960	14	Ductile Iron	130.0	1,710.32	365.17	3.12	3.25
P-220	J-220	J-210	780	12	Ductile Iron	100.0	-776.54	365.17	2.03	2.60
P-230	J-230	J-220	1,960	14	Ductile Iron	130.0	-710.26	363.14	1.26	0.64
P-231	J-230	J-231	370	14	Ductile Iron	130.0	500.22	361.74	0.14	0.37
P-232	J-345	J-230	2,510	12	Ductile Iron	130.0	-181.49	361.87	0.27	0.11
P-233	J-231	J-345	2,030	8	Ductile Iron	130.0	47.94	361.60	0.14	0.07
P-234	J-234	J-231	470	14	Ductile Iron	130.0	-139.28	361.74	0.01	0.03
P-235	J-235	J-234	1,550	8	Ductile Iron	130.0	-53.49	361.72	0.13	0.08
P-236	J-234	J-235	1,520	8	Ductile Iron	130.0	54.06	361.60	0.13	0.08
P-238	J-384	J-235	630	8	Ductile Iron	130.0	-49.81	361.60	0.05	0.07
P-240	J-231	J-240	1,530	8	Ductile Iron	130.0	294.59	358.76	2.98	1.94
P-242	J-240	J-140	2,100	10	Ductile Iron	100.0	-318.69	361.33	2.57	1.22
P-250	J-240	J-250	1,360	8	Ductile Iron	130.0	356.17	354.99	3.77	2.77
P-260	J-260	J-250	630	12	Ductile Iron	130.0	-287.27	354.99	0.16	0.25
P-262	J-171	J-260	1,460	10	Ductile Iron	130.0	-12.20	354.83	0.26e-2	0.18e-2
P-265	J-260	J-265	1,610	12	Ductile Iron	130.0	228.43	354.57	0.27	0.17
P-280	J-265	J-280	970	10	Ductile Iron	130.0	135.09	354.42	0.15	0.15

Scenario: Base
Steady State Analysis
Pipe Report

Link Label	Start Node	End Node	Length (ft)	Diameter (in)	Material	Roughness	Discharge (gpm)	End Calculated Hydraulic Grade (ft)	Headloss (ft)	Friction Slope (ft/1000ft)
P-282	J-193	J-280	1,850	12	Ductile Iron	130.0	-97.02	354.42	0.06	0.03
P-300	J-210	J-310	1,320	12	Ductile Iron	130.0	719.09	363.34	1.83	1.39
P-301	J-210	J-304	1,580	8	Ductile Iron	130.0	133.47	364.47	0.70	0.44
P-305	J-304	J-306	2,800	8	Ductile Iron	130.0	133.47	363.23	1.24	0.44
P-307	J-308	J-306	750	12	Ductile Iron	130.0	140.20	363.23	0.05	0.07
P-309	J-310	J-308	560	12	Ductile Iron	130.0	179.54	363.28	0.06	0.11
P-310	J-310	J-320	1,210	12	Ductile Iron	130.0	505.30	362.46	0.87	0.72
P-320	J-330	J-320	530	12	Ductile Iron	130.0	-425.98	362.46	0.28	0.53
P-330	J-330	J-331	1,510	12	Ductile Iron	130.0	325.73	361.70	0.48	0.32
P-332	J-332	J-331	1,250	8	Ductile Iron	130.0	-62.35	361.70	0.14	0.11
P-333	J-331	J-340	710	12	Ductile Iron	130.0	250.07	361.56	0.14	0.20
P-334	J-340	J-334	410	8	Ductile Iron	130.0	0.12	361.56	0.00	0.00
P-335	J-334	J-332	360	8	Ductile Iron	130.0	-18.34	361.57	0.41e-2	0.01
P-336	J-336	J-332	1,370	8	Ductile Iron	130.0	-11.64	361.57	0.01	0.49e-2
P 337	J-336	J-334	1,390	8	Ductile Iron	130.0	-7.04	361.56	0.26e-2	0.19e-2
P-340	J-340	J-350	1,890	12	Ductile Iron	130.0	97.80	361.50	0.07	0.03
P-342	J-342	J-340	620	10	Ductile Iron	130.0	-45.10	361.56	0.01	0.02
P-344	J-344	J-342	880	10	Ductile Iron	130.0	18.44	361.55	0.34e-2	0.38e-2
P-345	J-345	J-344	450	12	Ductile Iron	130.0	170.15	361.55	0.05	0.11
P-346	J-344	J-346	530	10	Ductile Iron	130.0	71.22	361.53	0.02	0.05
P-350	J-350	J-360	740	12	Ductile Iron	130.0	59.14	361.49	0.01	0.01
P-352	J-370	J-350	1,090	8	Ductile Iron	130.0	-15.79	361.50	0.01	0.01
P-354	J-365	J-350	1,040	8	Ductile Iron	130.0	-16.84	361.50	0.01	0.01
P-356	J-350	J-352	760	8	Ductile Iron	130.0	-58.86	361.57	0.08	0.10
P-358	J-352	J-336	1,030	12	Ductile Iron	130.0	56.83	361.56	0.01	0.01
P-360	J-370	J-360	530	12	Ductile Iron	130.0	15.92	361.49	0.64e-3	0.12e-2
P-362	J-360	J-365	650	6	Ductile Iron	130.0	-1.90	361.49	0.61e-4	0.94e-4
P-365	J-360	J-365	1,580	8	Ductile Iron	130.0	-0.98	361.49	0.61e-4	0.39e-4
P-366	J-370	J-342	2,020	8	Ductile Iron	130.0	-31.62	361.55	0.06	0.03
P-370	J-380	J-370	740	12	Ductile Iron	130.0	-1.75	361.49	0.00	0.00
P-375	J-344	J-380	1,210	8	Ductile Iron	130.0	42.86	361.49	0.07	0.05
P-380	J-380	J-382	850	12	Ductile Iron	130.0	-49.60	361.50	0.01	0.01
P-383	J-346	J-382	410	8	Ductile Iron	130.0	52.81	361.50	0.03	0.08
P-384	J-382	J-384	960	8	Ductile Iron	130.0	-44.38	361.55	0.06	0.06
P-385	J-384	J-345	1,080	6	Ductile Iron	130.0	-18.67	361.60	0.05	0.05
P-400	J-400	J-105	1,160	12	Ductile Iron	130.0	1,348.52	371.36	5.15	4.44
P-405	J-400	J-421	1,080	12	Ductile Iron	130.0	547.43	365.31	0.90	0.84
P-410	J-410	J-400	1,250	12	Ductile Iron	100.0	-358.87	366.22	0.78	0.63
P-420	J-420	J-410	1,160	12	Ductile Iron	100.0	-322.38	365.44	0.59	0.51
P-421	J-421	J-420	1,590	12	Ductile Iron	130.0	309.34	364.84	0.47	0.30
P-422	J-421	J-422	580	8	Ductile Iron	130.0	203.83	364.75	0.57	0.98
P-423	J-422	J-431	890	8	Ductile Iron	130.0	174.01	364.10	0.65	0.73
P-426	J-431	J-428	1,080	8	Ductile Iron	130.0	20.39	364.08	0.01	0.01
P-428	J-428	J-432	290	10	Ductile Iron	130.0	-45.60	364.09	0.01	0.02
P-430	J-430	J-420	900	12	Ductile Iron	130.0	-554.60	364.84	0.77	0.86
P-431	J-431	J-430	1,650	12	Ductile Iron	130.0	66.14	364.07	0.03	0.02
P-432	J-431	J-432	1,050	12	Ductile Iron	130.0	45.60	364.09	0.01	0.01
P-440	J-440	J-430	4,050	12	Ductile Iron	130.0	-660.37	364.07	4.81	1.19
P-500	J-306	J-500	7,380	12	Ductile Iron	130.0	253.37	361.74	1.49	0.20
P-502	J-500	J-352	2,150	12	Ductile Iron	130.0	152.48	361.57	0.17	0.08

Scenario: Base
Steady State Analysis
Junction Report

Node Label	Elevation (ft)	Demand Type	Demand (gpm)	Demand Pattern	Calculated Demand (gpm)	Calculated Hydraulic Grade (ft)	Pressure (psi)
J-105	330	Demand	0.00	Fixed	0.00	371.36	17.89
J-110	305	Demand	0.00	Fixed	0.00	370.01	28.11
J-120	195	Demand	38.89	Fixed	38.89	365.44	73.70
J-122	195	Demand	14.59	Fixed	14.59	365.44	73.70
J-126	195	Demand	10.75	Fixed	10.75	362.43	72.40
J-130	195	Demand	7.71	Fixed	7.71	363.29	72.77
J-140	190	Demand	17.68	Fixed	17.68	361.33	74.09
J-150	200	Demand	0.50	Fixed	0.50	358.81	68.67
J-160	200	Demand	179.29	Fixed	179.29	355.58	67.28
J-170	210	Demand	184.19	Fixed	184.19	354.77	62.60
J-171	190	Demand	98.13	Fixed	98.13	354.83	71.28
J-172	210	Demand	230.32	Fixed	230.32	354.60	62.53
J-180	210	Demand	164.21	Fixed	164.21	354.44	62.46
J-182	205	Demand	94.61	Fixed	94.61	354.47	64.63
J-183	200	Demand	39.71	Fixed	39.71	354.87	66.97
J-184	150	Demand	42.94	Fixed	42.94	357.13	89.57
J-190	205	Demand	20.47	Fixed	20.47	354.40	64.61
J-192	195	Demand	44.58	Fixed	44.58	354.35	68.91
J-193	190	Demand	81.85	Fixed	81.85	354.36	71.07
J-194	190	Demand	81.85	Fixed	81.85	354.30	71.05
J-195	205	Demand	82.86	Fixed	82.86	354.22	64.53
J-196	200	Demand	0.00	Fixed	0.00	354.32	66.73
J-200	275	Demand	20.94	Fixed	20.94	368.29	40.34
J-210	190	Demand	81.22	Fixed	81.22	365.17	75.75
J-220	190	Demand	14.19	Fixed	14.19	363.14	74.87
J-230	195	Demand	28.55	Fixed	28.55	361.87	72.16
J-231	200	Demand	18.40	Fixed	18.40	361.74	69.94
J-234	210	Demand	31.73	Fixed	31.73	361.72	65.61
J-235	225	Demand	57.74	Fixed	57.74	361.60	59.07
J-240	210	Demand	257.11	Fixed	257.11	358.76	64.33
J-250	200	Demand	68.90	Fixed	68.90	354.99	67.02
J-260	192	Demand	46.64	Fixed	46.64	354.83	70.42
J-265	205	Demand	93.34	Fixed	93.34	354.57	64.68
J-280	195	Demand	38.07	Fixed	38.07	354.42	68.94
J-304	285	Demand	0.00	Fixed	0.00	364.47	34.36
J-306	240	Demand	20.30	Fixed	20.30	363.23	53.29
J-308	215	Demand	39.34	Fixed	39.34	363.28	64.12
J-310	215	Demand	34.26	Fixed	34.26	363.34	64.15
J-320	190	Demand	79.31	Fixed	79.31	362.46	74.58
J-330	195	Demand	100.25	Fixed	100.25	362.18	72.30
J-331	210	Demand	13.32	Fixed	13.32	361.70	65.60
J-332	230	Demand	32.36	Fixed	32.36	361.57	56.89
J-334	235	Demand	11.42	Fixed	11.42	361.56	54.73
J-336	255	Demand	75.51	Fixed	75.51	361.56	46.08
J-340	225	Demand	107.05	Fixed	107.05	361.56	59.05
J-342	215	Demand	31.92	Fixed	31.92	361.55	63.37
J-344	220	Demand	37.63	Fixed	37.63	361.55	61.21
J-345	210	Demand	40.61	Fixed	40.61	361.60	65.56
J-346	220	Demand	18.40	Fixed	18.40	361.53	61.20
J-350	245	Demand	64.89	Fixed	64.89	361.50	50.38

Scenario: Base
Steady State Analysis
Junction Report

Node Label	Elevation (ft)	Demand Type	Demand (gpm)	Demand Pattern	Calculated Demand (gpm)	Calculated Hydraulic Grade (ft)	Pressure (psi)
J-352	262	Demand	36.80	Fixed	36.80	361.57	43.06
J-360	270	Demand	77.94	Fixed	77.94	361.49	39.56
J-365	290	Demand	13.96	Fixed	13.96	361.49	30.91
J-370	250	Demand	29.74	Fixed	29.74	361.49	48.21
J-380	245	Demand	94.21	Fixed	94.21	361.49	50.37
J-382	230	Demand	47.59	Fixed	47.59	361.50	56.86
J-384	230	Demand	24.11	Fixed	24.11	361.55	56.89
J-400	210	Demand	43.78	Fixed	43.78	366.22	67.55
J-410	195	Demand	36.49	Fixed	36.49	365.44	73.70
J-420	185	Demand	77.12	Fixed	77.12	364.84	77.77
J-421	245	Demand	34.26	Fixed	34.26	365.31	52.03
J-422	235	Demand	29.82	Fixed	29.82	364.75	56.11
J-428	255	Demand	65.99	Fixed	65.99	364.08	47.17
J-430	160	Inflow	39.63	Fixed	-39.63	364.07	88.25
J-431	220	Demand	41.88	Fixed	41.88	364.10	62.31
J-432	250	Demand	0.00	Fixed	0.00	364.09	49.34
J-440	185	Demand	194.71	Fixed	194.71	359.26	75.35
J-500	210	Demand	100.89	Fixed	100.89	361.74	65.62

Resource Management Int'l Inc
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CITY OF SHERWOOD
MODEL THREE - PRESSURE ZONE

Scenario Summary Report

Base

Scenario Summary	
Demand Alternative	Base-Average Daily
Physical Alternative	Base-Physical
Initial Settings Alternative	Base-Initial Settings
Operational Alternative	Base-Operational
Age Alternative	Base-Age Alternative
Constituent Alternative	Base-Constituent
Trace Alternative	Base-Trace Alternative
Fire Flow Alternative	Base-Fire Flow

Hydraulic Analysis Summary	
Analysis	Steady State
Friction Method	Hazen-Williams Formula
Accuracy	0.010000
Trials	25

Calibration			
Demand Operation	<none>	Roughness Operation	<none>
Demand	0.00	Roughness	0.00

12/98

CITY OF SHERWOOD-MODEL THREE
PRESSURE ZONE
NEAR-TERM (2002)

NO FIRE FLOW
RESERVOIR @ 375
ONE 50 HP PUMP

P-70 = 12"
P-80 = 12"

HYDRAULIC STATUS:

Hydraulic status for steady-state conditions

Balanced	Trials = 4, Accuracy = 0.008077
Flow Supplied	559.02 gpm
Flow Demanded	559.02 gpm
Flow Stored	0.00 gpm
R-1	Reservoir: Emptying
PMP-1	Pump: On
PRV-1	PRV: Active, Setting = 49.99 psi
PRV-2	PRV: Active, Setting = 54.17 psi
PRV-3	PRV: Active, Setting = 64.99 psi
PRV-4	PRV: Active, Setting = 52.00 psi
PRV-5	PRV: Active, Setting = 59.99 psi

Scenario: Base
Steady State Analysis
Reservoir Report

Node Label	Reservoir Surface Elevation (ft)	Reservoir Inflow (gpm)	Calculated Hydraulic Grade (ft)
R-1	375	-559.02	375.00

Scenario: Base
Steady State Analysis
Pump Report

Link Label	Shutoff Head (ft)	Shutoff Discharge (gpm)	Design Head (ft)	Design Discharge (gpm)	Maximum Operating Head (ft)	Maximum Operating Discharge (gpm)	Current Status	Start Calculated Hydraulic Grade (ft)	End Calculated Hydraulic Grade (ft)	Discharge (gpm)	Pump Head (ft)
PMP-1	163.68	0.00	128.59	1,063.07	0.00	2,126.14	On	374.98	530.25	559.02	155.27

Scenario: Base
Steady State Analysis
Valve Report

Link Label	End Elevation (ft)	Diameter (in)	Minor Loss	Initial Grade Setting (ft)	Initial Status	Current Status	Discharge (gpm)	End Calculated Hydraulic Grade (ft)	Headloss (ft)
PRV-1	310	8	0.00	425.50	Active	Throttling	5.08	425.60	103.64
PRV-2	305	8	0.00	430.15	Active	Throttling	12.70	430.26	98.98
PRV-3	275	8	0.00	425.15	Active	Throttling	69.78	425.28	104.07
PRV-4	305	6	0.00	425.15	Active	Throttling	36.18	425.25	104.91
PRV-5	300	8	0.00	438.60	Active	Throttling	17.13	438.72	91.46

Scenario: Base
Steady State Analysis
Pipe Report

Link Label	Start Node	End Node	Length (ft)	Diameter (in)	Material	Roughness	Discharge (gpm)	End Calculated Hydraulic Grade (ft)	Headloss (ft)	Friction Slope (ft/1000ft)
P-2	PMP-1-In	R-1	80	16	Ductile Iror	130.0	-559.02	375.00	0.02	0.21
P-5	J-10	PMP-1-Ou	80	16	Ductile Iror	130.0	-559.02	530.25	0.02	0.21
P-10	J-10	J-15	1,020	16	Ductile Iror	130.0	180.04	530.21	0.03	0.03
P-11	J-11	J-10	720	8	Ductile Iror	130.0	-53.95	530.23	0.06	0.08
P-11A	PRV-4-In	J-11	70	6	Ductile Iror	130.0	-36.18	530.17	0.01	0.16
P-11B	J-11A	PRV-4-Ou	210	6	Ductile Iror	130.0	-36.18	425.25	0.03	0.16
P-11C	J-12	J-11A	940	8	Ductile Iror	130.0	-36.18	425.22	0.04	0.04
P-15	J-15	J-20	500	10	Ductile Iror	130.0	176.23	530.08	0.12	0.25
P-20	J-20	J-25	980	10	Ductile Iror	130.0	164.17	529.87	0.21	0.22
P-25	J-30	J-25	550	8	Ductile Iror	130.0	-144.50	529.87	0.28	0.51
P-27	J-25	J-28	570	8	Ductile Iror	130.0	14.59	529.86	0.42e-2	0.01
P-30	J-35	J-30	660	8	Ductile Iror	130.0	-62.44	529.58	0.07	0.11
P-31	J-30	J-31	500	10	Ductile Iror	130.0	196.99	529.43	0.15	0.31
P-32	J-30	J-70	690	10	Ductile Iror	130.0	-149.19	529.71	0.13	0.18
P-33	J-31	J-32	470	12	Ductile Iror	130.0	196.99	529.37	0.06	0.13
P-34	J-32	J-36	750	12	Ductile Iror	130.0	69.78	529.36	0.01	0.02
P-35	J-40	J-35	530	8	Ductile Iror	130.0	-37.06	529.51	0.02	0.04
P-36A	J-36	PRV-3-In	80	8	Ductile Iror	130.0	69.78	529.35	0.01	0.13
P-36B	PRV-3-Out	J-101	100	8	Ductile Iror	130.0	69.78	425.27	0.01	0.13
P-37	J-32	J-37	710	8	Ductile Iror	130.0	48.22	529.32	0.05	0.07
P-38	J-37	J-38	1,870	8	Ductile Iror	130.0	10.58	529.32	0.29e-2	0.15e-2
P-39	J-38	J-37	820	8	Ductile Iror	130.0	-13.53	529.32	0.29e-2	0.35e-2
P-40	J-45	J-40	260	8	Ductile Iror	130.0	-22.47	529.49	0.43e-2	0.02
P-42	J-48	J-45	470	8	Ductile Iror	130.0	22.34	529.49	0.01	0.02
P-45	J-45	J-50	360	8	Ductile Iror	130.0	37.20	529.47	0.01	0.04
P-50	J-50	J-60	410	8	Ductile Iror	130.0	-53.25	529.50	0.03	0.08
P-52	J-50	J-52	1,570	8	Ductile Iror	130.0	72.05	529.25	0.22	0.14
P-53	J-52	J-53	690	8	Ductile Iror	130.0	5.08	529.25	0.73e-3	0.11e-2
P-54	J-52	J-54	250	8	Ductile Iror	130.0	35.55	529.24	0.01	0.04
P-56	PRV-2-Out	J-56	140	8	Ductile Iror	130.0	12.70	430.26	0.79e-3	0.01
P-56A	J-54	PRV-2-In	100	8	Ductile Iror	130.0	12.70	529.24	0.61e-3	0.01
P-57	PRV-1-Out	J-57	600	8	Ductile Iror	130.0	5.08	425.60	0.64e-3	0.11e-2
P-57A	J-54	PRV-1-In	40	8	Ductile Iror	130.0	5.08	529.24	0.61e-4	0.15e-2
P-58	J-56	J-58	630	8	Ductile Iror	130.0	6.35	430.26	0.1e-2	0.16e-2
P-60	J-70	J-60	790	8	Ductile Iror	130.0	100.34	529.50	0.21	0.26
P-65	J-60	J-48	380	8	Ductile Iror	130.0	30.59	529.49	0.01	0.03
P-70	J-80	J-70	730	12	Ductile Iror	130.0	273.64	529.71	0.17	0.23
P-80	J-90	J-80	1,310	12	Ductile Iror	130.0	287.60	529.88	0.33	0.25
P-90	J-10	J-90	250	16	Ductile Iror	130.0	321.86	530.21	0.02	0.08
P-92	J-90	J-92	980	8	Ductile Iror	130.0	34.26	530.18	0.04	0.04
P-93	J-92	PRV-5-In	110	8	Ductile Iror	130.0	17.13	530.18	0.11e-2	0.01
P-94	PRV-5-Out	J-94	950	8	Ductile Iror	130.0	17.13	438.71	0.01	0.01
P-100	J-101	J-100	549	8	Ductile Iror	130.0	0.00	425.27	0.00	0.00
P-101	J-102	J-101	351	8	Ductile Iror	130.0	-69.78	425.27	0.05	0.13
P-103	J-103	J-102	520	8	Ductile Iror	130.0	-41.86	425.22	0.03	0.05
P-104	J-104	J-103	840	8	Ductile Iror	130.0	-14.58	425.19	0.01	0.01
P-105	J-104	J-105	170	8	Ductile Iror	130.0	14.58	425.19	0.13e-2	0.01
P-107	J-105	J-107	390	8	Ductile Iror	130.0	14.58	425.18	0.29e-2	0.01
P-108	J-12	J-107	310	8	Ductile Iror	130.0	6.36	425.18	0.49e-3	0.16e-2
P-610	J-52	J-610	700	12	Ductile Iror	130.0	21.90	529.25	0.15e-2	0.22e-2

Scenario: Base
Steady State Analysis
Pipe Report

Link Label	Start Node	End Node	Length (ft)	Diameter (in)	Material	Roughness	Discharge (gpm)	End Calculated Hydraulic Grade (ft)	Headloss (ft)	Friction Slope (ft/1000ft)
P-612	J-610	J-612	1,570	12	Ductile Iron	130.0	-78.99	529.28	0.04	0.02
P-614	J-612	J-614	1,840	12	Ductile Iron	130.0	-78.99	529.33	0.04	0.02
P-616	J-614	J-32	1,900	12	Ductile Iron	130.0	-78.99	529.37	0.04	0.02

Scenario: Base
Steady State Analysis
Junction Report

Node Label	Elevation (ft)	Demand Type	Demand (gpm)	Demand Pattern	Calculated Demand (gpm)	Calculated Hydraulic Grade (ft)	Pressure (psi)
J-10	335	Demand	3.17	Fixed	3.17	530.23	84.43
J-11	315	Demand	17.77	Fixed	17.77	530.17	93.05
J-11A	280	Demand	0.00	Fixed	0.00	425.22	62.80
J-12	295	Demand	29.82	Fixed	29.82	425.18	56.30
J-15	385	Demand	3.81	Fixed	3.81	530.21	62.79
J-20	310	Demand	12.06	Fixed	12.06	530.08	95.17
J-25	365	Demand	5.08	Fixed	5.08	529.87	71.29
J-28	375	Demand	14.59	Fixed	14.59	529.86	66.97
J-30	325	Demand	34.26	Fixed	34.26	529.58	88.47
J-31	330	Demand	0.00	Fixed	0.00	529.43	86.24
J-32	280	Demand	0.00	Fixed	0.00	529.37	107.84
J-35	380	Demand	25.38	Fixed	25.38	529.51	64.65
J-36	275	Demand	0.00	Fixed	0.00	529.36	109.99
J-37	335	Demand	24.11	Fixed	24.11	529.32	84.03
J-38	320	Demand	24.11	Fixed	24.11	529.32	90.52
J-40	375	Demand	14.59	Fixed	14.59	529.49	66.81
J-45	395	Demand	7.61	Fixed	7.61	529.49	58.16
J-48	405	Demand	8.25	Fixed	8.25	529.49	53.84
J-50	410	Demand	18.40	Fixed	18.40	529.47	51.66
J-52	360	Demand	9.52	Fixed	9.52	529.25	73.19
J-53	285	Demand	5.08	Fixed	5.08	529.25	105.62
J-54	315	Demand	17.77	Fixed	17.77	529.24	92.64
J-56	285	Demand	6.35	Fixed	6.35	430.26	62.81
J-57	250	Demand	5.08	Fixed	5.08	425.60	75.94
J-58	290	Demand	6.35	Fixed	6.35	430.26	60.65
J-60	420	Demand	16.50	Fixed	16.50	529.50	47.35
J-70	350	Demand	24.11	Fixed	24.11	529.71	77.71
J-80	300	Demand	13.96	Fixed	13.96	529.88	99.41
J-90	310	Demand	0.00	Fixed	0.00	530.21	95.23
J-92	310	Demand	17.13	Fixed	17.13	530.18	95.21
J-94	285	Demand	17.13	Fixed	17.13	438.71	66.47
J-100	255	Demand	0.00	Fixed	0.00	425.27	73.63
J-101	275	Demand	0.00	Fixed	0.00	425.27	64.98
J-102	280	Demand	27.92	Fixed	27.92	425.22	62.80
J-103	285	Demand	27.28	Fixed	27.28	425.19	60.62
J-104	285	Demand	0.00	Fixed	0.00	425.19	60.62
J-105	268	Demand	0.00	Fixed	0.00	425.19	67.97
J-107	302	Demand	20.94	Fixed	20.94	425.18	53.27
J-610	400	Demand	100.89	Fixed	100.89	529.25	55.89
J-612	345	Demand	0.00	Fixed	0.00	529.28	79.69
J-614	340	Demand	0.00	Fixed	0.00	529.33	81.87

CITY OF SHERWOOD
MODEL FOUR - GRAVITY ZONE

Scenario Summary Report

CASE TWO - TRANSMISSION MAIN ADDED

Base

Scenario Summary

Demand Alternative	Peak Day
Physical Alternative	Base-Physical
Initial Settings Alternative	Base-Initial Settings
Operational Alternative	Base-Operational
Age Alternative	Base-Age Alternative
Constituent Alternative	Base-Constituent
Trace Alternative	Base-Trace Alternative
Fire Flow Alternative	Base-Fire Flow

Hydraulic Analysis Summary

Analysis	Steady State
Friction Method	Hazen-Williams Formula
Accuracy	0.001000
Trials	25

Calibration

Demand Operation	<none>	Roughness Operation	<none>
Demand	0.00	Roughness	0.00

Created 12/98, DAG

CITY OF SHERWOOD-MODEL FOUR
FUTURE - URBAN RESERVE (2017)
GRAVITY ZONE
WATER SUPPLY FROM BULL RUN - 6.0 MGD
*BASE RUN
*NO FIRE FLOW
*ALL WELLS OFF
*RESERVOIR AT 375'
*P-100 & P-110 (24")
*P-900 TRANSMISSION MAIN TO RESERVOIR (24"), Q=4170 gpm (6 MGD)

HYDRAULIC STATUS:

Hydraulic status for steady-state conditions

Balanced	Trials = 5, Accuracy = 0.000518
Flow Supplied	6,194.96 gpm
Flow Demanded	6,194.96 gpm
Flow Stored	0.00 gpm
R-1	Reservoir: Emptying

Scenario: Base
Steady State Analysis
Reservoir Report

Node Label	Reservoir Surface Elevation (ft)	Reservoir Inflow (gpm)	Calculated Hydraulic Grade (ft)
R-1	375	2,024.96	375.00

Scenario: Base
Steady State Analysis
Pipe Report

Link Label	Start Node	End Node	Length (ft)	Diameter (in)	Material	Roughness	Discharge (gpm)	End Calculated Hydraulic Grade (ft)	Headloss (ft)	Friction Slope (ft/1000ft)
P-100	R-1	J-105	300	24	Ductile Iron	130.0	6,194.96	374.24	0.76	2.55
P-110	J-105	J-110	250	24	Ductile Iron	130.0	4,112.03	373.94	0.30	1.19
P-113	J-120	J-110	920	8	Ductile Iron	100.0	-652.24	373.94	12.46	13.55
P-114	J-120	J-122	280	12	Ductile Iron	100.0	-91.60	361.49	0.01	0.05
P-115	J-122	J-200	670	8	Ductile Iron	100.0	-610.08	369.51	8.02	11.97
P-116	J-400	J-120	1,030	12	Ductile Iron	100.0	515.07	361.48	1.26	1.22
P-120	J-130	J-120	840	12	Ductile Iron	100.0	1,215.76	361.48	5.15	6.13
P-122	J-130	J-122	610	8	Ductile Iron	100.0	-505.68	361.49	5.16	8.46
P-124	J-130	J-220	260	8	Ductile Iron	100.0	139.60	356.13	0.20	0.78
P-126	J-126	J-130	530	8	Ductile Iron	100.0	-336.17	356.33	2.16	4.08
P-130	J-140	J-130	820	12	Ductile Iron	100.0	1,234.68	356.33	5.02	6.12
P-132	J-220	J-140	1,530	8	Ductile Iron	100.0	290.67	351.31	4.82	3.15
P-134	J-140	J-126	760	8	Ductile Iron	100.0	-322.94	354.17	2.86	3.76
P-140	J-160	J-140	2,240	12	Ductile Iron	130.0	1,396.41	351.31	10.60	4.73
P-160	J-160	J-170	780	12	Ductile Iron	100.0	961.27	337.68	3.03	3.88
P-165	J-171	J-160	2,170	8	Ductile Iron	130.0	-251.04	340.71	3.09	1.43
P-170	J-170	J-180	1,300	12	Ductile Iron	100.0	541.39	335.94	1.74	1.34
P-172	J-170	J-171	1,340	8	Ductile Iron	100.0	30.66	337.62	0.06	0.05
P-174	J-172	J-170	1,910	10	Ductile Iron	130.0	-195.96	337.68	0.58	0.31
P-178	J-183	J-172	1,870	10	Ductile Iron	130.0	6.07	337.09	0.95e-3	0.51e-3
P-180	J-180	J-190	920	12	Ductile Iron	130.0	321.02	335.64	0.30	0.33
P-182	J-180	J-182	1,740	10	Ductile Iron	130.0	-21.07	335.95	0.01	0.49e-2
P-184	J-182	J-183	830	10	Ductile Iron	130.0	-444.71	337.10	1.15	1.38
P-185	J-183	J-184	1,780	10	Ductile Iron	130.0	-502.59	340.21	3.12	1.75
P-186	J-184	J-440	3,420	12	Ductile Iron	130.0	-631.49	343.94	3.73	1.09
P-190	J-190	J-192	930	10	Ductile Iron	130.0	259.57	335.16	0.48	0.51
P-192	J-192	J-193	2,910	10	Ductile Iron	130.0	-75.16	335.31	0.15	0.05
P-194	J-192	J-194	1,770	10	Ductile Iron	130.0	136.36	334.88	0.28	0.16
P-195	J-194	J-196	1,220	10	Ductile Iron	130.0	-116.88	335.02	0.14	0.12
P-196	J-196	J-195	1,520	10	Ductile Iron	130.0	23.98	335.01	0.01	0.01
P-197	J-196	J-182	3,210	12	Ductile Iron	130.0	-307.83	335.95	0.93	0.29
P-200	J-110	J-200	370	14	Ductile Iron	130.0	3,459.78	369.51	4.43	11.97
P-210	J-200	J-210	960	14	Ductile Iron	130.0	2,831.89	361.58	7.93	8.26
P-220	J-220	J-210	780	12	Ductile Iron	100.0	1,324.83	361.58	5.45	6.99
P-230	J-230	J-220	1,960	14	Ductile Iron	130.0	1,158.13	356.13	3.12	1.59
P-231	J-230	J-231	370	14	Ductile Iron	130.0	788.11	352.68	0.32	0.87
P-232	J-345	J-230	2,510	12	Ductile Iron	130.0	-344.97	353.00	0.89	0.36
P-233	J-231	J-345	2,030	8	Ductile Iron	130.0	104.69	352.11	0.57	0.28
P-234	J-234	J-231	470	14	Ductile Iron	130.0	-246.30	352.68	0.04	0.09
P-235	J-235	J-234	1,550	8	Ductile Iron	130.0	-108.66	352.64	0.47	0.30
P-236	J-234	J-235	1,520	8	Ductile Iron	130.0	109.81	352.17	0.47	0.31
P-238	J-384	J-235	630	8	Ductile Iron	130.0	-134.43	352.17	0.28	0.45
P-240	J-231	J-240	1,530	8	Ductile Iron	130.0	420.99	346.92	5.76	3.77
P-242	J-240	J-140	2,100	10	Ductile Iron	100.0	-425.75	351.31	4.39	2.09
P-250	J-240	J-250	1,360	8	Ductile Iron	130.0	571.26	337.88	9.04	6.65
P-260	J-260	J-250	630	12	Ductile Iron	130.0	-493.85	337.88	0.44	0.69
P-262	J-171	J-260	1,460	10	Ductile Iron	130.0	117.17	337.44	0.17	0.12
P-265	J-260	J-265	1,610	12	Ductile Iron	130.0	562.68	336.03	1.42	0.88
P-280	J-265	J-280	970	10	Ductile Iron	130.0	261.61	335.52	0.50	0.52
P-282	J-193	J-280	1,850	12	Ductile Iron	130.0	-186.47	335.52	0.21	0.11

Scenario: Base
Steady State Analysis
Pipe Report

Link Label	Start Node	End Node	Length (ft)	Diameter (in)	Material	Roughness	Discharge (gpm)	End Calculated Hydraulic Grade (ft)	Headloss (ft)	Friction Slope (ft/1000ft)
P-300	J-210	J-310	1,320	12	Ductile Iron	130.0	1,207.08	356.81	4.77	3.61
P-301	J-210	J-304	1,580	8	Ductile Iron	130.0	228.74	359.68	1.90	1.20
P-305	J-304	J-306	2,800	8	Ductile Iron	130.0	228.74	356.32	3.36	1.20
P-307	J-308	J-306	750	12	Ductile Iron	130.0	337.00	356.32	0.26	0.34
P-309	J-310	J-308	560	12	Ductile Iron	130.0	371.51	356.58	0.23	0.41
P-310	J-310	J-320	1,210	12	Ductile Iron	130.0	805.52	354.74	2.07	1.71
P-320	J-330	J-320	530	12	Ductile Iron	130.0	-735.95	354.74	0.77	1.45
P-330	J-330	J-331	1,510	12	Ductile Iron	130.0	648.02	352.24	1.73	1.14
P-332	J-332	J-331	1,250	8	Ductile Iron	130.0	-161.82	352.24	0.79	0.63
P-333	J-331	J-340	710	12	Ductile Iron	130.0	474.51	351.79	0.46	0.64
P-334	J-340	J-334	410	8	Ductile Iron	130.0	184.15	351.46	0.33	0.80
P-335	J-334	J-332	360	8	Ductile Iron	130.0	20.69	351.45	0.01	0.01
P-336	J-336	J-332	1,370	8	Ductile Iron	130.0	-154.12	351.45	0.79	0.58
P-337	J-336	J-334	1,390	8	Ductile Iron	130.0	-153.45	351.46	0.80	0.57
P-340	J-340	J-350	1,890	12	Ductile Iron	130.0	212.55	351.51	0.28	0.15
P-342	J-342	J-340	620	10	Ductile Iron	130.0	22.46	351.79	0.34e-2	0.01
P-344	J-344	J-342	880	10	Ductile Iron	130.0	117.60	351.79	0.10	0.12
P-345	J-345	J-344	450	12	Ductile Iron	130.0	372.73	351.90	0.21	0.47
P-346	J-344	J-346	530	10	Ductile Iron	130.0	128.04	351.82	0.07	0.14
P-350	J-350	J-360	740	12	Ductile Iron	130.0	-49.16	351.52	0.01	0.01
P-352	J-370	J-350	1,090	8	Ductile Iron	130.0	38.81	351.51	0.05	0.05
P-354	J-365	J-350	1,040	8	Ductile Iron	130.0	5.82	351.51	0.12e-2	0.11e-2
P-356	J-350	J-352	760	8	Ductile Iron	130.0	241.49	350.47	1.05	1.38
P-358	J-352	J-336	1,030	12	Ductile Iron	130.0	-241.34	350.66	0.20	0.19
P-360	J-370	J-360	530	12	Ductile Iron	130.0	154.79	351.52	0.04	0.08
P-362	J-360	J-365	650	6	Ductile Iron	130.0	7.77	351.51	0.01	0.01
P-365	J-360	J-365	1,580	8	Ductile Iron	130.0	10.29	351.51	0.01	0.38e-2
P-366	J-370	J-342	2,020	8	Ductile Iron	130.0	-63.77	351.79	0.23	0.11
P-370	J-380	J-370	740	12	Ductile Iron	130.0	166.52	351.56	0.07	0.09
P-375	J-344	J-380	1,210	8	Ductile Iron	130.0	90.73	351.63	0.26	0.22
P-380	J-380	J-382	850	12	Ductile Iron	130.0	-142.98	351.69	0.06	0.07
P-383	J-346	J-382	410	8	Ductile Iron	130.0	111.90	351.69	0.13	0.32
P-384	J-382	J-384	960	8	Ductile Iron	130.0	-87.80	351.89	0.20	0.20
P-385	J-384	J-345	1,080	6	Ductile Iron	130.0	-41.31	352.11	0.22	0.20
P-400	J-400	J-105	1,160	12	Ductile Iron	130.0	2,082.93	374.24	11.50	9.92
P-405	J-400	J-421	1,080	12	Ductile Iron	130.0	923.47	360.35	2.38	2.20
P-410	J-410	J-400	1,250	12	Ductile Iron	100.0	-605.99	362.73	2.06	1.65
P-420	J-420	J-410	1,160	12	Ductile Iron	100.0	-566.56	360.67	1.69	1.46
P-421	J-421	J-420	1,590	12	Ductile Iron	130.0	549.73	358.98	1.37	0.86
P-422	J-421	J-422	580	8	Ductile Iron	130.0	343.69	358.86	1.49	2.57
P-423	J-422	J-431	890	8	Ductile Iron	130.0	317.53	356.89	1.97	2.22
P-426	J-431	J-428	1,080	8	Ductile Iron	130.0	19.26	356.87	0.01	0.01
P-428	J-428	J-432	290	10	Ductile Iron	130.0	-38.62	356.88	0.44e-2	0.02
P-430	J-430	J-420	900	12	Ductile Iron	130.0	1,011.51	358.98	2.35	2.61
P-431	J-431	J-430	1,650	12	Ductile Iron	130.0	215.68	356.64	0.25	0.15
P-432	J-431	J-432	1,050	12	Ductile Iron	130.0	45.86	356.88	0.01	0.01
P-440	J-440	J-430	4,050	12	Ductile Iron	130.0	1,115.18	356.64	12.70	3.14
P-500	J-306	J-500	7,380	12	Ductile Iron	130.0	547.93	350.14	6.19	0.84
P-502	J-500	J-352	2,150	12	Ductile Iron	130.0	-219.02	350.47	0.33	0.15
P-900	J-900	R-1	5,000	24	Ductile Iron	130.0	4,170.00	375.00	6.12	1.22

Scenario: Base
Steady State Analysis
Junction Report

Node Label	Elevation (ft)	Demand Type	Demand (gpm)	Demand Pattern	Calculated Demand (gpm)	Calculated Hydraulic Grade (ft)	Pressure (psi)
J-105	330	Demand	0.00	Fixed	0.00	374.24	19.13
J-110	305	Demand	0.00	Fixed	0.00	373.94	29.81
J-120	195	Demand	43.16	Fixed	43.16	361.48	71.99
J-122	195	Demand	12.80	Fixed	12.80	361.49	72.00
J-126	195	Demand	13.23	Fixed	13.23	354.17	68.83
J-130	195	Demand	11.00	Fixed	11.00	356.33	69.76
J-140	190	Demand	26.12	Fixed	26.12	351.31	69.76
J-160	200	Demand	184.10	Fixed	184.10	340.71	60.85
J-170	210	Demand	193.26	Fixed	193.26	337.68	55.21
J-171	190	Demand	164.53	Fixed	164.53	337.62	63.83
J-172	210	Demand	202.03	Fixed	202.03	337.09	54.96
J-180	210	Demand	241.44	Fixed	241.44	335.94	54.46
J-182	205	Demand	115.81	Fixed	115.81	335.95	56.63
J-183	200	Demand	51.81	Fixed	51.81	337.10	59.28
J-184	150	Demand	128.90	Fixed	128.90	340.21	82.25
J-190	205	Demand	61.45	Fixed	61.45	335.64	56.49
J-192	195	Demand	198.37	Fixed	198.37	335.16	60.61
J-193	190	Demand	111.31	Fixed	111.31	335.31	62.84
J-194	190	Demand	253.24	Fixed	253.24	334.88	62.65
J-195	205	Demand	23.98	Fixed	23.98	335.01	56.22
J-196	200	Demand	166.97	Fixed	166.97	335.02	58.39
J-200	275	Demand	17.81	Fixed	17.81	369.51	40.87
J-210	190	Demand	71.24	Fixed	71.24	361.58	74.20
J-220	190	Demand	15.63	Fixed	15.63	356.13	71.84
J-230	195	Demand	25.05	Fixed	25.05	353.00	68.33
J-231	200	Demand	16.14	Fixed	16.14	352.68	66.02
J-234	210	Demand	27.83	Fixed	27.83	352.64	61.68
J-235	225	Demand	84.04	Fixed	84.04	352.17	54.99
J-240	210	Demand	275.48	Fixed	275.48	346.92	59.21
J-250	200	Demand	77.41	Fixed	77.41	337.88	59.62
J-260	192	Demand	48.34	Fixed	48.34	337.44	62.89
J-265	205	Demand	301.07	Fixed	301.07	336.03	56.66
J-280	195	Demand	75.14	Fixed	75.14	335.52	60.77
J-304	285	Demand	0.00	Fixed	0.00	359.68	32.29
J-306	240	Demand	17.81	Fixed	17.81	356.32	50.30
J-308	215	Demand	34.51	Fixed	34.51	356.58	61.22
J-310	215	Demand	30.05	Fixed	30.05	356.81	61.32
J-320	190	Demand	69.57	Fixed	69.57	354.74	71.24
J-330	195	Demand	87.94	Fixed	87.94	353.97	68.74
J-331	210	Demand	11.69	Fixed	11.69	352.24	61.51
J-332	230	Demand	28.38	Fixed	28.38	351.45	52.52
J-334	235	Demand	10.02	Fixed	10.02	351.46	50.36
J-336	255	Demand	66.23	Fixed	66.23	350.66	41.37
J-340	225	Demand	100.27	Fixed	100.27	351.79	54.83
J-342	215	Demand	31.37	Fixed	31.37	351.79	59.15
J-344	220	Demand	36.37	Fixed	36.37	351.90	57.04
J-345	210	Demand	35.62	Fixed	35.62	352.11	61.45
J-346	220	Demand	16.14	Fixed	16.14	351.82	57.00
J-350	245	Demand	64.84	Fixed	64.84	351.51	46.06
J-352	262	Demand	263.81	Fixed	263.81	350.47	38.26

Scenario: Base
Steady State Analysis
Junction Report

Node Label	Elevation (ft)	Demand Type	Demand (gpm)	Demand Pattern	Calculated Demand (gpm)	Calculated Hydraulic Grade (ft)	Pressure (psi)
J-360	270	Demand	87.57	Fixed	87.57	351.52	35.25
J-365	290	Demand	12.24	Fixed	12.24	351.51	26.60
J-370	250	Demand	36.69	Fixed	36.69	351.56	43.92
J-380	245	Demand	67.18	Fixed	67.18	351.63	46.11
J-382	230	Demand	56.73	Fixed	56.73	351.69	52.62
J-384	230	Demand	87.94	Fixed	87.94	351.89	52.71
J-400	210	Demand	38.40	Fixed	38.40	362.73	66.05
J-410	195	Demand	39.43	Fixed	39.43	360.67	71.64
J-420	185	Demand	104.78	Fixed	104.78	358.98	75.24
J-421	245	Demand	30.05	Fixed	30.05	360.35	49.88
J-422	235	Demand	26.16	Fixed	26.16	358.86	53.56
J-428	255	Demand	57.88	Fixed	57.88	356.87	44.05
J-430	160	Demand	112.01	Fixed	112.01	356.64	85.03
J-431	220	Demand	36.73	Fixed	36.73	356.89	59.19
J-432	250	Demand	7.24	Fixed	7.24	356.88	46.22
J-440	185	Demand	483.69	Fixed	483.69	343.94	68.73
J-500	210	Demand	766.95	Fixed	766.95	350.14	60.60
J-900	160	Inflow	4,170.00	Fixed	-4,170.00	381.12	95.62

CITY OF SHERWOOD
MODEL FOUR-PRESSURE ZONE

Scenario Summary Report

BASE

Scenario Summary

Demand Alternative	.96 base
Physical Alternative	Base-Physical
Initial Settings Alternative	Base-Initial Settings
Operational Alternative	Base-Operational
Age Alternative	Base-Age Alternative
Constituent Alternative	Base-Constituent
Trace Alternative	Base-Trace Alternative
Fire Flow Alternative	Base-Fire Flow

Hydraulic Analysis Summary

Analysis	Steady State
Friction Method	Hazen-Williams Formula
Accuracy	0.001000
Trials	40

Calibration

Demand Operation	<none>	Roughness Operation	<none>
Demand	0.00	Roughness	0.00

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CITY OF SHERWOOD-MODEL FOUR
PRESSURE ZONE
FUTURE - URBAN RESERVE (2017)
 * NO FIRE FLOW
 * RESERVOIR @ 375'

P-70 & P-80 @ 16"

ONE LARGE PUMP

HYDRAULIC STATUS:

Hydraulic status for steady-state conditions

Balanced	Trials = 5, Accuracy = 0.000418
Flow Supplied	808.15 gpm
Flow Demanded	808.15 gpm
Flow Stored	0.00 gpm
R-1	Reservoir: Emptying
PMP-1	Pump: On
PRV-1	PRV: Active, Setting = 47.83 psi
PRV-2	PRV: Active, Setting = 52.00 psi
PRV-3	PRV: Active, Setting = 64.99 psi
PRV-4	PRV: Active, Setting = 52.00 psi
PRV-5	PRV: Active, Setting = 59.99 psi

Scenario: BASE
Steady State Analysis
Reservoir Report

Node Label	Reservoir Surface Elevation (ft)	Reservoir Inflow (gpm)	Calculated Hydraulic Grade (ft)
R-1	375	-808.15	375.00

Scenario: BASE
Steady State Analysis
Pump Report

Link Label	Shutoff Head (ft)	Shutoff Discharge (gpm)	Design Head (ft)	Design Discharge (gpm)	Maximum Operating Head (ft)	Maximum Operating Discharge (gpm)	Current Status	Start Calculated Hydraulic Grade (ft)	End Calculated Hydraulic Grade (ft)	Discharge (gpm)	Pump Head (ft)
PMP-1	163.68	0.00	128.59	1,063.07	0.00	2,126.14	On	374.97	519.56	808.15	144.60

Scenario: BASE
Steady State Analysis
Valve Report

Link Label	End Elevation (ft)	Diameter (in)	Minor Loss	Initial Grade Setting (ft)	Initial Status	Current Status	Discharge (gpm)	End Calculated Hydraulic Grade (ft)	Headloss (ft)
PRV-1	315	8	0.00	425.50	Active	Throttling	4.45	425.60	91.68
PRV-2	310	8	0.00	430.15	Active	Throttling	11.14	430.25	87.02
PRV-3	275	8	0.00	425.15	Active	Throttling	62.65	425.28	92.57
PRV-4	305	6	0.00	425.15	Active	Throttling	30.29	425.25	94.23
PRV-5	300	8	0.00	438.60	Active	Throttling	15.02	438.72	80.73

Scenario: BASE
Steady State Analysis
Pipe Report

Link Label	Start Node	End Node	Length (ft)	Diameter (in)	Material	Roughness	Discharge (gpm)	End Calculated Hydraulic Grade (ft)	Headloss (ft)	Friction Slope (ft/1000ft)
P-2	PMP-1-In	R-1	80	16	Ductile Iron	130.0	-808.15	375.00	0.03	0.42
P-5	J-10	PMP-1-Ou	80	16	Ductile Iron	130.0	-808.15	519.56	0.03	0.42
P-10	J-10	J-15	1,020	16	Ductile Iron	130.0	209.78	519.49	0.04	0.03
P-11	J-11	J-10	720	8	Ductile Iron	130.0	-45.88	519.53	0.04	0.06
P-11A	PRV-4-In	J-11	70	6	Ductile Iron	130.0	-30.29	519.48	0.01	0.12
P-11B	J-11A	PRV-4-Ou	210	6	Ductile Iron	130.0	-30.29	425.25	0.02	0.12
P-11C	J-12	J-11A	940	8	Ductile Iron	130.0	-30.29	425.23	0.03	0.03
P-15	J-15	J-20	500	10	Ductile Iron	130.0	206.43	519.33	0.17	0.33
P-20	J-20	J-25	980	10	Ductile Iron	130.0	195.86	519.03	0.30	0.30
P-25	J-30	J-25	550	8	Ductile Iron	130.0	-178.60	519.03	0.42	0.76
P-27	J-25	J-28	570	8	Ductile Iron	130.0	12.80	519.03	0.33e-2	0.01
P-30	J-35	J-30	660	8	Ductile Iron	130.0	-75.17	518.61	0.10	0.15
P-31	J-30	J-31	500	10	Ductile Iron	130.0	383.42	518.08	0.53	1.05
P-32	J-30	J-70	690	10	Ductile Iron	130.0	-310.04	519.10	0.49	0.71
P-33	J-31	J-32	470	12	Ductile Iron	130.0	383.42	517.88	0.20	0.43
P-34	J-32	J-36	750	12	Ductile Iron	130.0	85.47	517.86	0.02	0.03
P-35	J-40	J-35	530	8	Ductile Iron	130.0	-52.91	518.51	0.04	0.08
P-36A	J-36	PRV-3-In	80	8	Ductile Iron	130.0	62.65	517.85	0.01	0.11
P-36B	PRV-3-Ou	J-101	100	8	Ductile Iron	130.0	62.65	425.27	0.01	0.11
P-37	J-32	J-37	710	8	Ductile Iron	130.0	42.30	517.84	0.04	0.05
P-38	J-37	J-38	1,870	8	Ductile Iron	130.0	8.29	517.84	0.47e-2	0.25e-2
P-39	J-38	J-37	820	8	Ductile Iron	130.0	-12.86	517.84	0.47e-2	0.01
P-40	J-45	J-40	260	8	Ductile Iron	130.0	-40.11	518.47	0.01	0.05
P-42	J-48	J-45	470	8	Ductile Iron	130.0	46.53	518.45	0.03	0.06
P-45	J-45	J-50	360	8	Ductile Iron	130.0	79.96	518.39	0.06	0.17
P-50	J-50	J-60	410	8	Ductile Iron	130.0	-108.00	518.52	0.12	0.30
P-52	J-50	J-52	1,570	8	Ductile Iron	130.0	171.83	517.28	1.11	0.71
P-53	J-52	J-53	690	8	Ductile Iron	130.0	4.45	517.28	0.55e-3	0.8e-3
P-54	J-52	J-54	250	8	Ductile Iron	130.0	31.17	517.28	0.01	0.03
P-56	PRV-2-Ou	J-56	140	8	Ductile Iron	130.0	11.14	430.25	0.64e-3	0.46e-2
P-56A	J-54	PRV-2-In	100	8	Ductile Iron	130.0	11.14	517.28	0.43e-3	0.43e-2
P-57	PRV-1-Ou	J-57	600	8	Ductile Iron	130.0	4.45	425.60	0.49e-3	0.81e-3
P-57A	J-54	PRV-1-In	40	8	Ductile Iron	130.0	4.45	517.28	0.61e-4	0.15e-2
P-58	J-56	J-58	630	8	Ductile Iron	130.0	5.57	430.25	0.76e-3	0.12e-2
P-60	J-70	J-60	790	8	Ductile Iron	130.0	176.24	518.52	0.59	0.74
P-65	J-60	J-48	380	8	Ductile Iron	130.0	53.77	518.48	0.03	0.08
P-70	J-80	J-70	730	16	Ductile Iron	130.0	507.43	519.10	0.13	0.18
P-80	J-90	J-80	1,310	16	Ductile Iron	130.0	519.67	519.23	0.25	0.19
P-90	J-10	J-90	250	16	Ductile Iron	130.0	549.72	519.48	0.05	0.21
P-92	J-90	J-92	990	8	Ductile Iron	130.0	30.05	519.45	0.03	0.03
P-93	J-92	PRV-5-In	110	8	Ductile Iron	130.0	15.02	519.45	0.85e-3	0.01
P-94	PRV-5-Ou	J-94	950	8	Ductile Iron	130.0	15.02	438.71	0.01	0.01
P-100	J-101	J-100	549	8	Ductile Iron	130.0	0.00	425.27	0.00	0.00
P-101	J-102	J-101	350	8	Ductile Iron	130.0	-62.65	425.27	0.04	0.11
P-103	J-103	J-102	520	8	Ductile Iron	130.0	-38.16	425.23	0.02	0.04
P-104	J-104	J-103	840	8	Ductile Iron	130.0	-14.23	425.21	0.01	0.01
P-105	J-104	J-105	170	8	Ductile Iron	130.0	14.23	425.20	0.12e-2	0.01
P-107	J-105	J-107	390	8	Ductile Iron	130.0	14.23	425.20	0.27e-2	0.01
P-108	J-12	J-107	310	8	Ductile Iron	130.0	4.13	425.20	0.21e-3	0.69e-3
P-610	J-52	J-610	700	12	Ductile Iron	130.0	127.85	517.24	0.04	0.06

Scenario: BASE
Steady State Analysis
Pipe Report

Link Label	Start Node	End Node	Length (ft)	Diameter (in)	Material	Roughness	Discharge (gpm)	End Calculated Hydraulic Grade (ft)	Headloss (ft)	Friction Slope (ft/1000ft)
P-612	J-612	J-610	1,570	12	Ductile Iron	130.0	99.80	517.24	0.06	0.04
P-614	J-614	J-612	1,840	12	Ductile Iron	130.0	177.73	517.30	0.19	0.10
P-616	J-32	J-614	1,900	12	Ductile Iron	130.0	255.65	517.49	0.39	0.20

Scenario: BASE
Steady State Analysis
Junction Report

Node Label	Elevation (ft)	Demand Type	Demand (gpm)	Demand Pattern	Calculated Demand (gpm)	Calculated Hydraulic Grade (ft)	Pressure (psi)
J-10	335	Demand	2.78	Fixed	2.78	519.53	79.80
J-11	315	Demand	15.58	Fixed	15.58	519.48	88.43
J-11A	280	Demand	0.00	Fixed	0.00	425.23	62.80
J-12	295	Demand	26.16	Fixed	26.16	425.20	56.30
J-15	385	Demand	3.34	Fixed	3.34	519.49	58.16
J-20	310	Demand	10.58	Fixed	10.58	519.33	90.52
J-25	365	Demand	4.45	Fixed	4.45	519.03	66.61
J-28	375	Demand	12.80	Fixed	12.80	519.03	62.28
J-30	325	Demand	30.06	Fixed	30.06	518.61	83.72
J-31	330	Demand	0.00	Fixed	0.00	518.08	81.33
J-32	280	Demand	0.00	Fixed	0.00	517.88	102.87
J-35	380	Demand	22.26	Fixed	22.26	518.51	59.90
J-36	275	Demand	22.82	Fixed	22.82	517.86	105.02
J-37	335	Demand	21.15	Fixed	21.15	517.84	79.07
J-38	320	Demand	21.15	Fixed	21.15	517.84	85.55
J-40	375	Demand	12.80	Fixed	12.80	518.47	62.04
J-45	395	Demand	6.68	Fixed	6.68	518.45	53.39
J-48	405	Demand	7.24	Fixed	7.24	518.48	49.07
J-50	410	Demand	16.14	Fixed	16.14	518.39	46.87
J-52	360	Demand	8.35	Fixed	8.35	517.28	68.01
J-53	285	Demand	4.45	Fixed	4.45	517.28	100.45
J-54	315	Demand	15.58	Fixed	15.58	517.28	87.47
J-56	285	Demand	5.57	Fixed	5.57	430.25	62.81
J-57	250	Demand	4.45	Fixed	4.45	425.60	75.93
J-58	290	Demand	5.57	Fixed	5.57	430.25	60.65
J-60	420	Demand	14.47	Fixed	14.47	518.52	42.60
J-70	350	Demand	21.15	Fixed	21.15	519.10	73.12
J-80	300	Demand	12.24	Fixed	12.24	519.23	94.80
J-90	310	Demand	0.00	Fixed	0.00	519.48	90.58
J-92	310	Demand	15.02	Fixed	15.02	519.45	90.57
J-94	285	Demand	15.02	Fixed	15.02	438.71	66.47
J-100	255	Demand	0.00	Fixed	0.00	425.27	73.63
J-101	275	Demand	0.00	Fixed	0.00	425.27	64.98
J-102	280	Demand	24.49	Fixed	24.49	425.23	62.80
J-103	285	Demand	23.93	Fixed	23.93	425.21	60.63
J-104	285	Demand	0.00	Fixed	0.00	425.20	60.63
J-105	268	Demand	0.00	Fixed	0.00	425.20	67.98
J-107	302	Demand	18.36	Fixed	18.36	425.20	53.28
J-610	400	Demand	227.65	Fixed	227.65	517.24	50.70
J-612	345	Demand	77.92	Fixed	77.92	517.30	74.51
J-614	340	Demand	77.92	Fixed	77.92	517.49	76.75

CITY OF SHERWOOD
WATER SYSTEM MASTER PLAN UPDATE

APPENDIX D

SUMMARY OF ESTIMATED PROBABLE PROJECT COSTS

Project Description	Probable Project Cost ($)
1. Treated Water Reservoirs	
■ Initial Phase: 6.0-MG concrete tank	$ 3,800,000
■ Future Phase: 3.0-MG concrete tank**	1,925,000
2. Booster Station	
■ Southwest Booster Pumping Station (Fig. 7-1, Item 9)	700,000
■ 8-inch intertie across Hwy. 99W	150,000
3. Distribution System	
■ 12-inch pressure zone main under park site (Fig. 7-1, Item 1)	125,000
■ 12-inch pipe under Pine, Columbia and Washington (Fig. 7-1, Item 2)	140,000
■ 8- and 12-inch pipes under Gleneagle and Twelfth (Fig. 7-1, Item 3)	195,000
■ 12-inch pipe under Lincoln and Oregon (Fig. 7-1, Item 4)	220,000
■ 12-inch pipe under Tualatin-Sherwood and Tualatin-Scholls Roads w/ Hwy. 99W crossing (Fig. 7-1, Item 5)	385,000
■ 12-inch pipe between Roellich and Edy (Fig. 7-1, Item 6)	130,000
■ 24-inch main from the Bull Run Connection to reservoir site (Fig. 7-1, Item 7)	790,000
■ 24-inch replacement pipe reservoir site and intersection of Lincoln and Division (Fig. 7-1, Item 8)	50,000
■ 12-inch pipe NW from Hwy. 99W to connect to Edy PUD (Fig. 7-1, Item 10)	40,000
■ 12-inch pipe under Galbreath and Cipole Roads (Fig. 7-1, Item 11)	270,000
■ Replace 2-, 4- and 6-inch pipe lines with 8-inch pipe throughout City	690,000
TOTAL	$ 9,610,000

** Present worth cost, assuming the tank is constructed in 2005 (3% discount rate).

City of Sherwood Oregon

1999 Water System Master Plan Summary

Revised September 14, 1999

City of Sherwood, Oregon

1999 Water System Master Plan Summary

September 1999

Walt Hitchcock, Mayor

<u>Council Members</u>

Mark Cottle
Tom Krause
Scott Franklin
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Lee D. Weislogel, City Manager Pro Tem

Prepared by:
Bookman Edmonston Engineering
Squier & Associates
Murray, Smith & Associates
Robert E. Meyer, P.E., PLS, Consulting Engineer
Nicki Colliander, Engineering Coordinator

REGISTERED PROFESSIONAL ENGINEER
OREGON
MAY 6, 1955
ROBERT E. MEYER

Expires 12/31/00

Copy # 14

City of Sherwood, Oregon
Water System Master Plan Summary
September 1999

In addition to this summary, the City of Sherwood 1999 Water Master Plan consists of the following documents:

1. City of Sherwood, Oregon Water System Master Plan Update, dated April 1999, authored by Bookman-Edmonston Engineering, Inc, authorized by Sherwood, October 1997.

2. City of Sherwood Municipal Well Field Hydrogeological Evaluation, dated June 11, 1999, authored by Squier Associates, authorized by Sherwood, January 1999.

3. Analysis of Southwest Sherwood Service Zone, dated September 13, 1999, authored by Murray Smith Associates, authorized by Sherwood, February 1999.

4. In addition to the above three documents, Sherwood authorized the preparation of a Water Management and Conservation Plan. This Plan, mandated by Oregon State Water Resources Department (Division) is underway and will be presented to Sherwood for review and adoption shortly. When adopted, it will be an important element of Sherwood's 1999 Water Master Plan.

The above listed documents and this summary titled "City of Sherwood 1999 Water System Master Plan Project Summary", and system map, titled "City of Sherwood 1999 Water System Master Plan", is intended to constitute Sherwood's 1999 Water System Master Plan. The last Sherwood Water Master Plan was adopted in June 1991.

The City of Sherwood 1999 Water Master Plan, to be a useful tool, must be thoroughly reviewed annually.

Project Summary:

The projects listed in the attached Project Summary have been identified as necessary to provide Sherwood's water customer with a reliable, safe and economical product. The guidelines and standards used in identifying the projects were Oregon Administrative Rules, Chapter 333, including ORS 448, Drinking Water Program of the Oregon State Health Division and the American Water Works Association.

Several of the projects, through necessity, have been started, such as the search for additional water sources. Other projects have been completed, such as the Municipal Well Field Hydrogeological Evaluation.

Priorities:

Projects in process:

1. Securing additional source

2. Planning is underway to find a way to increase the pressure and flow to the West Sherwood Pressure Zone, an area above elevation 250 and containing 180 acres. The area includes a proposed elementary school scheduled to open in September 2000 and the YMCA.

3. Replacement and upsizing to 12 inches, the Lincoln and Oregon Street water lines.

The projects in the spread sheet, pages 3 through 6 are prioritized. Prioritization is dependent upon available funding, time and location of development.

Project Probable Cost:

The probable project cost has been developed for several projects. The cost contains a reasonable amount for construction, plus 30-50 percent for contingencies, overhead, and engineering.

In all cases, the probable cost is based on 1998 and 1999 construction costs.

Prepared by Consulting Engineer, Robert Meyer, P.E., PLS
September 14, 1999

City of Sherwood, Oregon

1999 Water System Master Plan

Project Summary

#	Description	Status	Costs Est.	Comments
1	**Addition Source of Supply** to meet year 2017 demands	Planning	To be determined	To supplement its water source, Sherwood is in the evaluating two sources, (1) joining with Tigard, Wilsonville, Tualatin, and Tualatin Valley Water district, in the development of the Willamette River Water Treatment Facility and (2) purchase of water from City of Portland combined system.
2	**West Sherwood Service Zone** - To provide that area of West Sherwood above elevation 250 feet with a residual pressure of approximately 45 pounds per square inch during instantaneous demand and fire flow demand at either the school site or YMCA.	Planning	$3.3 Million	The recommended solution is: (a) Install a ground level reservoir at about elevation 440 feet, along Kruger Road, approximately 2200 feet west of Hwy 99W. (b) The reservoir is to be filled from the Wyndham Ridge Booster Station via new piping along Elwert Road. (c) New piping from the reservoir to the distribution system, along with reinforcement piping, will provide adequate flow and pressure to the West Sherwood Service Zone as well as fire protection to the YMCA and proposed elementary school on Old Hwy 99W.
3	**Well Field Evaluation** - A study "City of Sherwood, Municipal Well Field Hydrogeological Evaluation dated June 1999, prepared by Squier & Associates, predicts a sustained yield of 1.0 million gallon per day in year 2000 through 2006 and reduces to 840,000 gallons per day through year 2010. On an average day the wells will produce water for 8,000 residents through year 2006 and nearly 6900 residents through 2010. The above predictions of well yield assume improvements to Well 3 and Well 5.	Completed	N/A	Sherwood is in a Groundwater Limited Area which places restrictions on development of new wells. However, the wells do not produce their design capacity, or allowable yield. Sherwood should investigate means to increase yield to design capacity.
3A	Deepen Well #5 - Increase yield	Requires Immediate Action	To be determined	Well #5 depth was terminated 20 feet above a primary water bearing basalt pillow. If successful, deepening will allow the closing off of the water zone which cascades into the water causing milky (aerated) water.
3B	Lower Well #3 Pump Bowls	Scheduled for completion winter 1999-2000	To be determined	Bowls are at 130 feet depth and well depth is at 319 feet. Lowering bowls will help insure a reliable yield.
3C	Spada Farm Well Analysis - The Spada Farm Well is located outside of the Urban Growth Boundary and east of the proposed Home Depot site. The eight inch well drilled 1983 to depth of 500 feet was tested at 400 gallons per minute. The owner has approached Sherwood to investigate the well as a possible source for municipal use.	Testing to be completed in October.	To be determined	This well is to be investigated and tested as a possible potable source for Sherwood. This well may be a consideration of a municipal irrigation source if it is not economically feasible to improve the Spada Well for use as a potable source.

City of Sherwood, Oregon

1999 Water System Master Plan

Project Summary

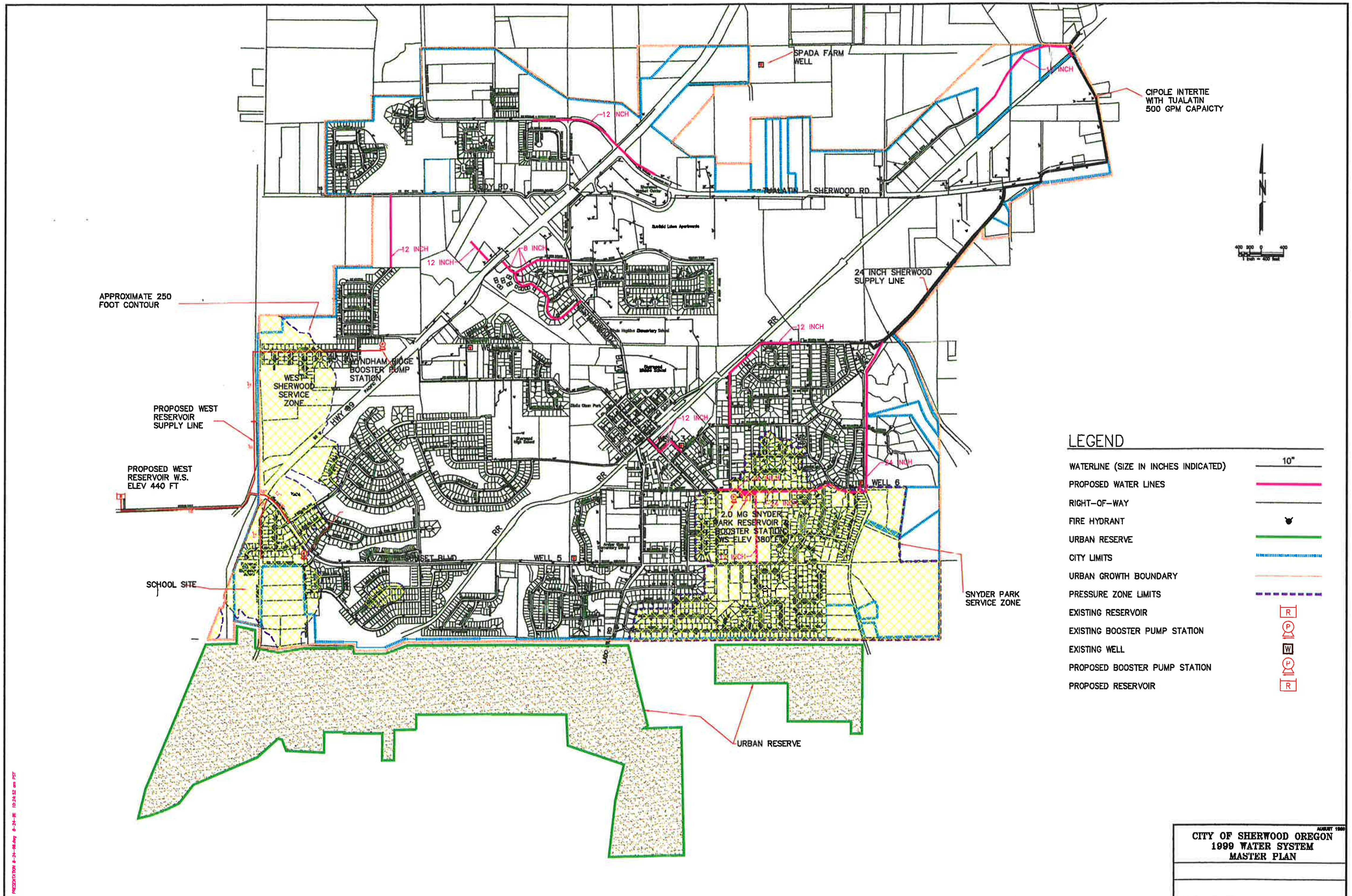
#	Description	Status	Costs Est.	Comments
4	**Lincoln and Oregon Streets Water Line** - This project replaces a leak prone six inch water line requiring frequent repair. The new 12 inch replacement line completes a 12 inch loop by connecting to a 12 inch near Oregon and Roy and to a 12 inch at Lincoln and Willamette.	Construction	$ 327,300	Construction is underway and scheduled for completion October 2, 1999.
5	**Treated Water Storage** - Water storage requirements for Sherwood in Year 2017 is 9.0 million gallons (MG). The majority of the storage 5 to 7 MG is to be placed at Snyder Park site near the 2.0 MG reservoir. The remainder of the storage is planned for the west and south part of Sherwood.	Planning	$3,800,000 for 6.0 MG Storage	Construction of new storage to serve the West Sherwood Service Zone may delay the need for additional storage at Snyder Park.
6	**Water management and Conservation Plan** - An implemented Water Management and Conservation Plan is designed to conserve water which may reduce and/or delay size of some system components.	Planning	Implementation costs to be determined	This plan is mandated by the Oregon Department of Water resources as a requirement of the permit for Well #6.
7	**Distribution System Improvements**	Planning Design		Size and location of the water distribution system improvements were finalized after completing several computer network models under different demand. The priority of the distribution system improvements is dependent upon location, size, and type of growth.
7A	Install 5,000 feet of 24 inch pipe along Murdock Road and Division Street from the Regional Supply line to the Snyder reservoir site		$ 790,000	
7B	Install 300 feet of 24 inch pipe to replace the 16 inch Gravity Zone pipe between the Snyder Park reservoir site and the intersection of Lincoln and Division Streets.		$ 50,000	
7C	Install 1,600 feet 12 inch pipe from the Pressure Zone booster station through Snyder Park to the intersection of Sunset Boulevard and Aldergrove Avenue. The upgrade is needed to deliver fire flows to the southerly portion of Aldergrove Avenue and the area of Highpoint Drive and Cascara Terrace.		$ 125,000	
7D	Install 3000 feet of 12 inch pipe across Highway 99W under Tualatin-Sherwood and Sherwood-Scholls Roads to complete system loop.		$ 385,000	
7E	Install 1,500 feet of 12 inch pipe from the north end of a 10 inch pipe in Roellich Avenue to Edy Road to complete system loop.		$ 130,000	

City of Sherwood, Oregon

1999 Water System Master Plan

Project Summary

#	Description	Status	Costs Est.	Comments
7F	Install 500 feet of 12 inch pipe northwest from Highway 99W near Cedar Creek to connect to a 10 inch in Edy Village (Development plans submitted)		$ 40,000	
7G	Install 2,400 feet of 12 inch pipe along Galbreath and Cipole Roads to connect to system at north edge of BMC West.		$ 270,000	
7H	Install 2,950 feet of 8 inch piping and 300 feet of 12 inch piping to replace the 6 inch water lines under Gleneagle Drive and Twelfth Street.		$ 195,000	
7I	Install 300 feet of 12 inch piping in Washington Street to increase capacity and replace leaking 8 inch water mains.		$ 140,000	
7J	Replace 11,300 feet of 2-, 4- and 6 inch pipe lines at various locations with 8 inch pipe.		$ 690,000	
8	**Snyder Park Reservoir Structural Analysis**	Scheduled for completion by October 1999.		Snyder Park Reservoir Structural Analysis is required on this 27 year old facility. Since design in 1972, the Uniform Building Code has changed the class of earthquake to a more severe quake. The analysis will uncover any structural defects that may require attention.
9	**Water meter inspection and replacement program**			Implement a systematic water meter inspection and replacement program to remove meters that no longer function properly.
10	**Hydrant flushing program**			Develop a schedule for periodically flushing fire hydrants throughout the system.
11	**Snyder Park Pressure Zone Booster Station** - This station, constructed in 1996, services the southeastern area of the City.	Draft analysis complete	$ 160,000	The installed pumping capacity was designed for peak and fire flow demand at complete build out of the service area. However, at average demand and night time demand, it is necessary to continuously run a 50 horsepower pump to sustain pressure. This situation as well as other deficiencies requires correction.



CITY OF SHERWOOD OREGON
1999 WATER SYSTEM
MASTER PLAN

City of Sherwood

MUNICIPAL WELL FIELD
Hydrogeological Evaluation

August 23, 1999

99416

SQUIER ASSOCIATES

City of Sherwood
20 N.W. Washington Street
Sherwood, Oregon

CITY OF SHERWOOD
MUNICIPAL WELL FIELD
HYDROGEOLOGICAL EVALUATION

August 23, 1999

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CITY OF SHERWOOD
MUNICIPAL WELL FIELD
HYDROGEOLOGIC EVALUATION

1.0 INTRODUCTION

This report presents the findings of a hydrogeologic evaluation conducted for the City of Sherwood, Oregon's municipal ground water well field. The study, as completed by Squier Associates, Inc., includes a review of available geologic and hydrogeologic reports, well field-specific hydrogeologic data, and the evaluation of well-specific hydrologic data. This hydrogeologic evaluation has been undertaken for the purpose of providing a tool that will assist the City of Sherwood in its understanding of this ground water resource. By increasing their understanding, the City of Sherwood will be able to make informed decisions on their future water supply plans.

1.1 Project Background

The City of Sherwood is located southwest of the Portland Metropolitan area (refer, Figures 1 and 2). The City has a current estimated population of about 9,800 people and anticipates increased growth. Historically, Sherwood has obtained all of its municipal drinking water supply from ground water. Most of this water is produced by pumping four ground water supply wells, designated Well number (No.) 3 through Well No. 6. These four ground water supply wells have been installed in various part of the city as shown on Figure 3. Well No. 1 (drilled in 1890) and Well No. 2 (drilled in 1923) were shut down in 1984 due to the presence of microbiological contamination. Well No. 1 and Well No. 2 were located for this study.

In 1998, Sherwood began updating its Water System Master Plan. As part of the planning process, Sherwood is evaluating several alternatives to help meet forecasted future water demand, including purchasing water from either the City of Portland's Bull Run Water Supply System, or joining other water providers in developing the proposed Willamette River Water Supply System. Critical to the City's commitment to one or both of the surface water sources, is an understanding of the long term reliability of the existing well system. This hydrogeologic evaluation is a direct outgrowth of this planning process, whereupon Sherwood has made the decision to increase its understanding of the reliability of the ground water supply.

1.2 Project Objectives

The objectives of the hydrogeologic evaluation for the City of Sherwood's Municipal Well Field are:

- Determine the source(s) of water for each of the production wells (i.e. aquifer identification) and develop a conceptual understanding of the basalt ground water system.
- Evaluate seasonal and long term trends in well water levels, using data provided by the City.
- Characterize groundwater quality in general, and specific water quality of each city well using available data.
- Identify ways in which Sherwood can improve the long term reliability of its ground water system in terms of water quantity and quality.
- Evaluate the feasibility of options identified by the City to increase system yield, such as adding new wells, and water rights transfers.
- Discuss an approach to conducting a preliminary evaluation of Aquifer Storage Recovery (ASR) feasibility.

1.3 Scope of Work

The scope of work for this municipal well field hydrogeological evaluation has been tailored to meet the City of Sherwood's objectives. In order to facilitate meeting these objectives, this work has been subdivided into 5 tasks, as follows:

Task a. Project Database. This subtask includes the collection, review, and compilation of the available geologic, ground water, and well information pertinent to the Sherwood Municipal Well Field.

Task b. Develop Conceptual Hydrogeologic Model. Using well logs, information from published reports, input from local experts, and our experience working with aquifers of the Columbia River Basalt Group (CRBG), a Conceptual Hydrogeological Model of the Sherwood ground water system has been developed.

Task c. Develop Hydrographs for Selected Wells. An understanding of seasonal and long term water level trends is considered critical to developing an approach to ground water management.

Task d. Conduct Well Hydraulics Evaluation. The increased understanding of the factors that appear to control individual well productivity is an important part of this evaluation. Included in this understanding are a detailed analysis of each production well in terms of its design, casing depth, diameter, pump size and depth setting, specific capacity, and historical water levels.

Task e. Ground Water Quality Evaluation. Of equal importance is water quality. Using data provided by Sherwood, ground water quality has been reviewed and evaluated.

Task f. Prepare Report with Recommendations. This report includes maps, figures and tables to support the text as appropriate. Included in the report is a summary of our interpretations about the hydrogeology of the Sherwood area, identification of basalt aquifers, a survey of ground water use in the study area, and recommendations for management of the ground water resource, from both a water quantity and quality perspective.

2.0 EXISTING WATER SUPPLY WELL SYSTEM

The City of Sherwood maintains four water supply wells for the purpose of municipal ground water production (refer, Figure 2). Information on these four wells is provided below and summarized in Table 1. In addition, the City of Sherwood maintains a reservoir located at 275 Division Street. The water supply wells are located according to street addresses, tax lots, and according to the U.S. Geological Survey (USGS) coordinate system. The USGS coordinate system is based on a series of parallel lines extending 6-miles apart. The north / south tiered lines are numbered consecutively from Township 1 North (T1N) and Township 1 South (T1S). The east / west extending lines are numbered consecutively from Range 1 East (R1E) and Range 1 West (R1W). The 6-mile square blocks formed by the Township and Range lines are termed Sections. There are 36 sections within a Township / Range Block. The Sections are further divided into quarter-quarters using the letters a (northeast), b (northwest), c (southwest), and d (southeast).

2.1 Well No. 3

Location Details. 300 Pine Street (corner of Pine and Willamette Streets)
County Map 2S132DB, Tax Lot 400
T2S / R1W / Section 32bd.

Well Description. According to Oregon Water Resources Department (OWRD) records, Well No. 3 is the oldest of the operating ground water production wells in the Sherwood Municipal Well Field System. Copies of the information on file with the OWRD are included in Appendix A. Well No. 3 was drilled in 1946 and is reported by OWRD to be 339 feet deep, however the City of Sherwood's records indicate that the well is 319 feet deep. OWRD records indicate that Well No. 3 was constructed with 16-inch casing set from surface (0 feet) to depth 36 feet and 12-inch casing set from depth 35 feet to depth 122 feet. The City's records indicate that the 12-inch casing is set from surface to depth 77 feet. The City's information further indicates that Well No. 3 has an 8-inch pump with piping connected to a 75 horse power Johnston pump and an intake setting of 110 feet.

In April 1999 Schneider Equipment, Inc. and Drilling Company of Saint Paul, Oregon (Schneider) began rehabilitation of Well No. 3. Accordingly the well's pump intake was lowered to depth 130 feet. The pump intake depth of 130 feet is used in Table 1.

According to the 1946 well log for Well No. 3, basalt (described as lava rock) was encountered at depth 137 feet. A "sand rock" is described as being encountered from depth 38 feet to 137 feet.

Since the casing only extends to depth 122 feet, this indicates that 15 feet of detrital sediments overlying the basalts are uncased. No specific information on water bearing zones is provided. Also according to the 1946 well log, the well had an initial specific capacity of about 12 gallons per minute (gpm) per foot. The specific capacity is defined as the ratio of the pumping rate to the drawdown in the well and is typically considered an indicator of well productivity . According an evaluation completed by the City of Sherwood in 1984, this well is capable of producing 920 gpm and is a good producing well. Using water level measurements from December 17, 1998, it appears that the well has a drawdown of 21 feet after 1 hour of pumping, suggesting a current specific capacity of 44 gpm per foot.

Historical Well Use. Data provided by the City of Sherwood concerning historical pumping efforts for Well No. 3 have been tabulated. The available data only covers the period between 1993 to March 1999. During this period the well has typically been used to produce between 9 to 13 million gallons per month. The well has been used to produce as much as 26.4 million gallons in one month's period.

Historical Water Quality. Representative water quality samples were collected and analyzed by a contracted analytical laboratory from Well No. 3 in June 1996. The samples were tested for volatile organic compounds (VOCs), pesticides (including herbicides), and inorganics. No concerns were detected during this evaluation. The inorganics evaluated included antimony, arsenic, barium, beryllium, cadmium, chromium, cyanide, fluoride, lead, mercury, nickel, nitrate, nitrite, selenium, sulfate, and thallium. These elements and compounds were not detected in the water quality sample.

The historical water quality data indicates that the ground water produced by Well No. 3 is moderately hard, with a measured hardness of 88. Hardness is typically reported as parts per million (ppm) of Calcium carbonate ($CaCO_3$) equivalents and is calculated from the concentrations of calcium and magnesium. Hardness values ranging from 51 to 120 are considered moderately hard.

2.2 Well No. 4

Location Details. 1530 Meinecke Road,
County Map 2S131A, Tax Lot 701
T2S / R1W / Section 31ab.

Well Description. Well No. 4 was drilled in 1969 and according to OWRD records is 455 feet deep (refer, Appendix B). The City records indicate a completion depth of 458 feet deep. Casing was installed from about 1-foot above ground (+1 feet) to depth 99 feet. The ground water intake was set at 400 feet with an 8-inch pump piping connected to a 60 horse power Cornell pump.

The 1969 well log for Well No. 4 indicates that basalt was encountered at depth 78 feet. Red basalt is identified at depth 98 feet suggesting weathering and the presence of water. The casing was set at depth 99 feet, probably based on the presence of a water bearing zone at that depth. The overlying sediments are effectively sealed off by the casing in this well.

Historical Well Use. The initial specific capacity for this well, as measured in 1969, was approximately 2 gpm per foot. In 1984 the total well capacity was reported as 350 gpm and the well was reported as a low to moderate producing well. Since 1993, the production range for this well has been from about 2 to 4 million gallons per month. Using January 1997 water level data and a pumping rate of 350 gpm, the well's specific capacity remains about 2 gpm per foot.

Historical Water Quality. Historical water quality reports for Well No. 4 include a series of sampling and analysis events conducted between August 1986 and November 1986 for mercury content. Apparently 0.0021 milligrams per liter (mg/L) of mercury was detected in August, an average of 0.0016 mg/L of mercury for four samples was detected in September, and less than 0.0005 mg/L was detected in November 1986. Water quality samples were also collected in 1996 and tested for the same parameters as Well No. 3 at that time (VOCs, pesticides, and inorganics). No concerns were noted in the evaluation. A water sample was also tested for gross alpha radiation in 1997. This sample displayed 0.19 picocuries per liter (pCi/L). This is not a concern since it is less than 15 pCi/L, the standard developed by the Federal government for radionuclides. Testing for inorganics was again completed in 1997, with no concerns noted. Historical hardness evaluations of Well No. 4's ground water indicate a range from 26.7 to 85.5. This suggests that the well water is soft to moderately hard.

2.3 Well No. 5

Location Details. 16491 Sunset Boulevard
County Map 2S132 CB, Tax Lot 6600
T2S / R1W / Section 32cb

Well Details. Well No. 5 was drilled in 1984 and is 800 feet deep. OWRD records indicate that 16-inch diameter casing set from 1.5 feet above surface (+1.5 feet) to depth 200 feet (refer, Appendix C). The City's records indicate that the 16-inch diameter casing is set from surface (0 feet) to depth 252 feet. The well is completed with 8-inch diameter piping connected to a 150 horse power Worthington pump whose intake setting is at depth 430 feet.

According to the OWRD well Log, broken porous basalt was encountered at depth 83 feet. The first water bearing basalt zone is listed at depth 252 feet and extends 14 feet to depth 266 feet. The depth 252 feet is the depth that the City reports that the open hole begins. An additional 5-foot thick water bearing zone is also listed at depth 283 feet. Well No. 5 reportedly has a zone of cascading water at approximate depth 253 feet. In 1984 a down-hole video camera well log of Well No. 5 was completed. The interpretation of the video indicates a very rough, broken zone with caving at depth 253 feet. The video interpretation narrative also provides additional information on other zones including pillow basalts, broken zones, and fracture patterns (refer, Appendix C). In 1999 Dr. Marvin Beeson reviewed historical drill cuttings archived by the City. Dr. Beeson has developed a well log identifying the CRBG Formations, Members, and Units, as well as zones of interest. A copy of Dr. Beeson's well log for Well No. 5 is included in Appendix C.

Historical Well Use. Information on the initial specific capacity for this well was not recorded at the time the well was installed. The historical production from this well averaged about 7 million gallons per month, ranging from about 4 to about 15 million gallons per month. In March 1997, the well was used to produce 103 million gallons. Using the City of Sherwood's recorded measurements for August 27, 1996, a specific capacity of 2.4 gpm per foot has been calculated for Well No. 5.

Historical Water Quality. The water quality testing for Well No. 5 was according to the same schedule as Well No. 4. No VOCs, pesticides, nor gross alpha particles were reported. Low levels, below levels of regulatory concern of lead (0.011 mg/L) and mercury (0.0013 mg/L) were reported in 1986. A low concentration of arsenic (0.007 mg./L) was detected in 1989 and lead was again detected at 0.006 mg/L in 1992. Hardness testing on the Well No. 5 ground water indicates a hardness of 811, suggesting moderately hard water.

2.4 Well No. 6

Location Details. 1830 Roy Street
County Map 2S132, Tax Lot 12100
T2S / R1W / Section 32ad

Well Details. Well No. 6 is the newest of Sherwood's water supply wells, having been completed in 1997 (refer, Appendix D). Records indicate that the 16-inch diameter steel casing is set from 1-foot above surface (+1 feet) to depth 301 feet. A 12-inch welded steel liner has been set from depth 294 feet to 889 feet. Factory mill-cut perforations are in the liner at depths 430 feet to 450 feet, 640 feet to 680 feet, and 769 feet to 889 feet. Currently the well, which is 889 feet deep, has an 8-inch diameter production piping connected to a 75 horse power American Turbine pump with bowl set at 300 feet.

A very complete well log package has been developed by Schneider Drilling Company for the installation of Well No. 6. Apparently the drillhole was advanced to depth 1,030 feet and cemented back to depth 889 feet. A copy of the well log package on file with OWRD is included in Appendix D. In addition, Dr. Beeson also developed a well log based on the drill cuttings provided by Schneider Drilling and the City. Dr. Beeson's well log for Well No. 6 is also included in Appendix D. According to these well logs, basalt was encountered at depth 30 feet and extended to depth 1,014 feet. Predominantly clay was encountered after depth 1,014 feet.

Historical Well Use. Well No. 6 is the newest and most productive of Sherwood's water supply wells. Well No. 6 was completed in 1997 and test pumped as high as 2,000 gpm. It has a relatively high specific capacity (48 gpm per foot), and is capable of producing in excess of 1,000 gpm. However, due to OWRD permit restrictions this well is pumped at about 600 gpm. The well produced almost 128 million gallons in 1998, averaging about 10 gallons per month. Peak use of the well was in August 1998, when it produced about 21 million gallons.

Historical Water Quality. The water from Well No. 6 was tested for inorganics in 1996 and 1997 and VOCs in 1997. No VOCs were detected. The inorganics detected include sodium (which was detected in all the wells), manganese (0.04 mg/L), and iron (0.08 mg/L). Manganese and iron are typically found in basalt water wells. Sodium is not a regulated compound. Manganese and iron are regulated as Secondary Contaminants, with unenforceable regulatory levels. Arsenic, lead, and the other inorganics of concern were not detected. A hardness of 204, indicating very hard water, was reported for ground water from Well No. 6.

2.5 Possible Additions to the City's Wellfield (Spada Well)

The Spada Well is located north of Sherwood, in a former agricultural area zoned for future inclusion in the Sherwood Urban Growth Boundary. It is an 8-inch well that was drilled 500 feet deep in 1983, and when drilled produced 400 gpm. Water quality is not known; however it is a basalt well and quality should be comparable to Sherwood's other wells. A copy of the well log onfile with OWRD for the Spada well is included in Appendix E.

3.0 GEOLOGY

The nature and occurrence of geologic units and geologic structures governs to a large degree the behavior of ground water, and consequently, the availability of water in volumes sufficient to support municipal systems. An understanding of the vertical and lateral distribution of geologic units, and tectonic structures which effect these units is considered fundamental to managing the ground water resource.

3.1 Regional Setting

Sherwood is located adjacent to the Willamette Valley physiographic province, between the Coast Range to the west, and the Willamette Valley to the east, within a physiographic subdivision known as the Tualatin Valley. The Tualatin Valley area is characterized by a broad, generally low relief flood plain mantled with alluvium, and surrounded by low to moderate elevation hills composed largely of basalt bedrock. An example of these hills is Parrett Mountain, immediately south of Sherwood, which reaches an elevation of about 1,200 feet.

The dominant feature of the Tualatin Valley is the Tualatin River. This river meanders from its headwaters in the Coast Range to the Willamette River, east of Sherwood. The river passes approximately 2 miles to the north of the Sherwood area. Cedar Creek, a secondary tributary flows northeastward from Parrett Mountain to the City Limits and then flows northward to its confluence with Chicken Creek, which ultimately empties into the Tualatin River north of the City. Another tributary to the Tualatin River, Rock Creek flows northwestward, east of the City.

3.2 Regional Geologic Stratigraphy

A regional geologic map is included in the Ground Water Report No. 40, *Groundwater Conditions of Basalt Aquifers, Parrett Mountain, Northern Willamette Valley, Oregon* (Miller, et al., 1994). The pertinent portion of that map is included as Figure 3 with this report. Regional geologic stratigraphy is typically subdivided into five major units. The five geologic units are listed below, from youngest to oldest:

- <u>Recent Alluvium and Willamette Silt</u>. Recent alluvium is a general term for the deposits of local rivers, streams, and lakes that have been laid down due to the mechanics of the particular water body. In the Sherwood area the primary source of the most recent alluvium is the Tualatin River. The Willamette Silt is typically a massive, mica-bearing clayey silt, often with very fine sand and minor organic woody material (Wilson, 1998). The unit may exhibit crude lamination, and includes an upper oxidized yellow-brown section and grades to an oxygen reduced blue-gray section. The Willamette Silt is a term originally used by Allison (1953) and later by Trimble (1963) for the predominant surficial geologic unit in the Willamette Valley area. This unit includes the Catastrophic Flood Deposits, which consist of crudely to complex layered, poorly consolidated fine to medium sand and silt as well as gravel. The Catastrophic Flood Deposits were formed by repetitive catastrophic glacial outburst floods from ancient Lake Missoula, located in the western Montana area, that occurred between 13,000 and 18,000 years ago. During these repetitive flood events, sediment rich flood waters entered the Tualatin basin from the Columbia River, forming a temporary lake with a surface elevation near 400 feet mean sea level (msl). Soil development has introduced significant clay in the upper 6 to 15 feet of the deposited sediments.

- Hillsboro Formation. In his doctoral dissertation, Wilson (1998) proposes the Hillsboro Formation as a new name for the Pliocene-Pleistocene sediments found above the CRBG and below the Willamette Silt. Wilson's stratigraphic organization places all material found in the Tualatin Valley and formerly assigned to the Troutdale Formation and Sandy River Mudstone into the Hillsboro Formation. Accordingly, the fluvial and lacustrine silt, sand, gravel and conglomerate of Miocene to Pliocene age and formerly considered Troutdale Formation, are considered as Hillsboro Formation. The Troutdale Formation is now used to distinguish deposits of Columbia River origin, generally poorly to moderately consolidated gray brown silt, with lenses of sand and gravel.

  In addition, the Helvetia Formation sediments, which reach thicknesses over 800 feet in the Hillsboro area, are included as part of the Hillsboro Formation. Schlicker and Deacon (1967) describe the Helvetia Formation as a red-brown, poorly consolidated sand, sandy silt and silty clay deposit. It is found to unconformably overlie the CRBG. The Helvetia Formation has been mapped to occur throughout the Tualatin Valley region, and is believed to be related to the earliest Miocene sediments. In places it may represent a weathering horizon in the basalt, rather than a distinct alluvial deposit.

- Columbia River Basalt Group. The Columbia River Basalt Group (CRBG) is comprised of a series of Miocene to Pliocene basalt flows that erupted from fissures in eastern Oregon and Washington, and western Idaho. Thicknesses of individual basalt flows range from a few feet to several hundred feet, but are commonly less than 100 feet. The thickness of the CRBG in western Oregon ranges from a few hundred to as much as 1,500 feet. Knowledge of CRBG stratigraphy has evolved through the application of geochemistry, magnetic stratigraphy, and lithologic characteristics. The following five units of the CRBG (and one paleosol horizon) comprise the basalt stratigraphic section in the Sherwood area:

- Ginkgo Unit: The Gingko unit is the lower of two units of the Frenchman Springs member of Wanapum Basalt, an intercanyon basalt flow that made it all the way to Cape Foulweather (near Newport, Oregon). The younger unit of the Frenchman Springs member (Sand Hollow unit) represents the youngest basalt flow in the Portland Basin. The Sand Hollow unit is the uppermost unit of the CRBG in the Portland Basin and, hence, was exposed at the ground surface for an extensive time period, promoting deep weathering and erosion.

  The Ginkgo unit flowed over a somewhat variable topographic surface due to the initial tectonic folding, faulting, and uplifting in the region, as tectonic deformation was underway prior to that time. This tectonic deformation contributed to the erratically distributed two units of the Frenchman Springs member. Erosional remnants of these basalt flows are severely weathered and may only be found 10 to 30 feet thick, however, a maximum unit thickness up to 60 feet has been encountered.

  Where thick and not completely weathered, the Ginkgo basalt displays abundant plagioclase feldspar phenocrysts in a fine-grained, glassy groundmass. Phenocrysts (large crystals within a fine groundmass) and glomerocrysts (clots of phenocrysts), ranging from 0.3 centimeters (cm) to 2 cm in size, provide a reliable visual stratigraphic indicator identifiable in Gingko core specimens.

- Vantage Horizon: A prominent interbed and stratigraphic reference called the Vantage Horizon underlies the Ginkgo basalt and represents an erosional unconformity and significant geologic time break between placement of the Wanapum Basalt Formation's Frenchman Springs member and the underlying Grande Ronde Basalt. In the Portland area, the Vantage Horizon is a relatively thin, 1-foot to 4-foot thick paleosol, or ancient soil horizon, that locally overlies a weathered upper Grande Ronde Basalt surface.

The material in the Vantage Horizon typically contains well consolidated sedimentary materials including volcanic ash (tuff), sand, and other fluvial deposits, as well as carbon and wood remnants of a Miocene forest.

- Sentinel Bluffs Unit: The Sentinel Bluffs unit is the youngest of the Grande Ronde Basalt. The Grande Ronde Basalt typically consists of dense to slightly vesicular basalt flows with localized variably lithified flow breccia (a rock composed of angular rock fragments). Vesiculation occurs most commonly at flow tops and is formed by the post-emplacement migration and entrapment of aqueous vapor bubbles. Vesicular zones are laterally continuous for hundreds of feet to many miles. Regionally the Grande Ronde Basalt displays blocky to columnar jointing with poorly developed or absent entablature zones. Figure 4 has been provided to display the characteristic structure types of the CRBG.

  The thickness of the Sentinel Bluffs unit has been found to be generally uniform, ranging between 170 feet and 185 feet in the Portland Region. The Sentinel Bluffs unit consists of two identifiable primary flows, an upper and a lower flow. The upper and lower subunits typically are both about 85 feet to 90 feet in thickness and also are relatively uniform in distribution. The two flows are similar in characteristics, such as geochemical composition, strength, joint spacing, and weathering. The upper and lower flow boundary is typically identified at an interflow contact zone with minor flow breccia.

  Flow structure of the Sentinel Bluffs unit is generally of the blocky columnar to entablature colonnade form (refer, Figure 4). The upper flow usually has no or one internal intraflow contact; the lower flow commonly has two to three intraflow contacts, and occasionally up to four apparent intraflow contacts. Intraflow contacts, and the upper/lower interflow contact, generally include a thin (usually 1 foot or less) basal flow breccia or broken zone belonging to the overriding upper flow, overlying a thin contact zone a few inches thick, which, in turn, overlies an upper flow breccia and vesicular zone of the underlying flow. Development and thickness of the underlying flow breccia is highly variable, ranging from 1 foot to 20 feet or more. However, the relatively thin intraflow breccia zones have been most commonly observed.

Where relatively unweathered, the Sentinel Bluffs Unit is light to dark gray in color, slightly to very slightly weathered, and generally ranges in hardness from high to very high strength. Frequently the very high strengths are noted near the flow base. Extensive cooling related fractures are characteristic for the Sentinel Bluffs Unit. Fracture patterns suggest that much of the Sentinel Bluffs Unit consists of poorly developed entablature jointing with random joint orientations of short extent. Longer undulating colonnade type joints are only anticipated near flow bases, or locally as relatively small diameter columns that may fan radially at various attitudes. Cooling joints are generally moderately rough, of short extent, and, in the poorly developed entablature flow sections, non-planar and random in orientation. Colonnade flow portions have more preferentially oriented sets that tend to be high angle to vertical, relatively smooth, undulating, and of longer vertical extent than horizontal.

Each flow is separated from overlying and underlying flows by an interflow zone that may or may not contain a thin soil horizon. Where present, soil horizons are generally well-lithified and less than 1 foot in thickness. Several of the identified flow boundaries are characterized by flow breccia zones, varying in thickness up to 20 feet. Rock clasts comprising the breccia are generally subangular, gravel- to cobble-sized and vary in appearance from vesicular to dense, and in color. The character of the flow breccia zones varies substantially, from well-indurated, relatively competent and unweathered material to unlithified rubble zones with relatively high hydraulic conductivity.

- Winter Water Unit: The Winter Water unit is a part of the Grande Ronde Basalt. In hand specimen a basalt of the Winter Water unit is very similar in appearance and characteristics to the overlying flows. Differentiation from the Sentinel Bluffs unit and Winter Water unit is made with certainty only on the basis of geochemical composition and defining major oxide and trace element composition. Generally, the Winter Water unit basalt contains sparse, small blocky plagioclase glomerocrysts, is slightly darker gray in color than the overlying Sentinel Bluffs Unit and glassy to fine-grained (mineral grains have no form or are not visible without magnification), but this distinction is often masked by intraflow variability.

Extensive cooling related fractures are characteristic for the Winter Water unit, as they are in the Sentinel Bluffs unit. The Winter Water unit has typically two flow subunits. Flow structure, like the overlying Sentinel Bluffs unit, is characterized as columnar to entablature / colonnade, but is recognized to vary widely. A persistent flow breccia or pair of closely spaced flow breccias commonly defines the upper contact of the Winter Water unit. Basalt flow thicknesses between the two brecciated flows ranged from about 10 feet to 30 feet. The primary flow unit underlying the second breccia, however, displays substantial thickness up to 110 feet.

- Ortley Unit: The Ortley unit interfingers with the Umtamun unit and therefore may be observed as lithologically interchanged. This requires additional investigatory work to separate the units. This unit displays a uniform physical appearance with typically a glassy to fine grained and aphyric groundmass. The Ortley unit is noted to lack the tiny laths of plagioclase feldspar found in basalts of the Umtamum unit (refer, below) and the overlying Winter Water unit (refer, above). The Ortley unit flows typically display entablature or colonnade jointing patterns and less frequently change to a blocky - columnar jointing pattern.

- Umtanum Unit: As noted above, the Umtanum unit interfingers with the Ortley unit. The Umtanum unit is typically aphyric with a distinctive entabulature or colonnade jointing pattern. Of the two flows of the Umtanum unit, the upper flow has very low paleomagnetic inclination and can be distinguished from the Winter Water unit by the lack of phenocrysts.

- Grouse Creek Unit: A member of the Grande Ronde Basalt, the Grouse Creek unit has at least 10 distinct flows, although not all 10 have ever been identified in one section. Although, this unit is identical in appearance to the younger Ortley unit, the Grouse Creek unit flows lack microphenocrysts and rarely contain plagioclase phenocrysts in addition to being associated with a wide range of joint patterns. Geochemically, this unit is characterized by low titanium oxide (TiO_2) content and intermediate to low magnesium oxide (MgO) content. Also noted and useful in differentiating the unit is that basalts of the Grouse Creek Unit display a reverse magnetic polarity compared to current conditions and the Winter Water and Ortley Units.

- Wapshilla Ridge Unit: The basalts of the Wapshilla Ridge Unit within the Grande Ronde Basalt Formation are also characterized by a reversed polarity. This unit typically displays entablature to colonnade structure in the western regions of the flow. Hand samples are commonly glassy to fine grained with abundant microphyric plagioclase and rare plagioclase phenocrysts. Geochemically, low MgO and high to very high TiO_2 contents are associated with the Wapshilla Ridge Unit, providing another useful tool for unit differentiation.

- Mount Horrible / Downy Gulch Units: The Mount Horrible unit is physically and compositionally similar to the Grouse Creek unit, but the two units are always divided by the Wapshilla Ridge unit. The Downey Gulch unit has basalts that are associated with high TiO_2 and intermediate to low MgO flows. This unit is fine grained with microphyric and aphyric plagioclase crystals.

- Oligocene-Miocene Sedimentary Rocks (Marine Sediments). These rocks include tuffaceous, quartzitic, and silt and clay deposits, including siltstones and claystones (often appearing as "shale" on driller's logs). The marine sediments are found below the CRBG and are considered the bottom of the ground water system. Ground water within the marine sediments is often brackish, and exhibits elevated concentrations of total dissolved solids and may be only marginally potable, or non-potable.

3.3 Sherwood Area Geology

Based upon a review of logs for wells in the immediate vicinity of Sherwood and geologic maps from Miller, et al (1994) and Schlicker, et al (1979), the near surface geology includes a cover of quaternary alluvium, overlying Willamette Silt and Hillsboro Formation in the downtown area. An approximate 100-foot section of Hillsboro Formation is encountered in Well No. 3. South of downtown, as the land surface rises toward Parrett Mountain, CRBG is exposed at the surface, including several identified units of the Grande Ronde Basalt (refer, Figure 3). The CRBG flows are tilted, dipping towards the south and east.

Parrett Mountain is a highland area that separates the Tualatin Valley from the Willamette Valley. The area typically gently slopes from the valley lowlands to the Parrett Mountain peak at 1,250 feet. Some steep slopes are found in stream canyons and along the scarp of the Sherwood Fault. The highlands are formed by compressional forces creating the uplift of the basalt units, which dip to the southeast.

Geologic structures of the Sherwood area include the inferred southwest extension of the Columbia Transarc Lowland, known as the Sherwood Trough. The Columbia Transarc Lowland is a significantly major geologic structure that has allowed the Columbia River access through the Cascade Mountain Range. The primary structure of the Sherwood Trough is the Parrett Mountain and Lake Oswego Structure which is approximately 5 to 15 miles in length.

Based upon geologic unit interpretations and geologic mapping by Miller (1994), an approximate North 60 degrees East (N60°E)-trending dip-slip fault, referred to as the Sherwood Fault, is considered to define the south-east end of Tualatin Basin in the Sherwood area (refer, Figure 3). A dip-slip fault is a fault whose net slip is line with the dip of the fault trace. The fault is inferred to be near vertical with the northern fault block being downdropped. This downdropped block assists in forming the Sherwood Trough. The City of Sherwood's downtown area, water supply Well No. 3, Well No. 4, and Well No. 5 are located on the northern, downdropped block. Divergent, or splay faults, fan out from the southern fault extremity on the southern block. The vertical offset ranges from 50 feet 100 feet on the primary fault and 10 feet to about 35 feet on the southern fault splays.

4.0 HYDROGEOLOGY

All rocks have some capacity to hold an amount of water. The inherent physical characteristics of the different types of rocks (including unconsolidated materials) are relevant to the rock's ability to hold water. The rock's ability to store, transmit, and yield water is what is used to differentiate good aquifers from poor aquifers, in addition to aquifers from aquitards or confining beds. The hydrogeological characteristics that will be addressed in this report are as follows:

- Effective Porosity
- Hydraulic Conductivity
- Transmissivity
- Storativity and Specific Yield
- Ground Water Flow Direction
- Ground Water Flow Velocity
- Ground Water Recharge and Discharge

The source of ground water for the City of Sherwood's well field is water-bearing zones of the CRBG basalts. Relative to the CRBG, the water bearing zones are the interflow zones associated flow tops or bottoms which tend to be vesicular (containing many small cavities) with flow breccia zones and highly fractured. These interflow zones are laterally extensive and somewhat predictable. At depth, the interflow zones typically exhibit the characteristics of confined aquifers, however under special conditions they may behave as unconfined aquifers. How these hydrogeologic characteristics relate to CRBG water bearing zones and the City of Sherwood's well field is provided below.

4.1 Effective Porosity

The porosity of a hydrogeologic unit is the percentage of the aquifer that consists of open spaces. These open spaces are the locations where ground water may reside. The interconnectedness of the open spaces is a relationship referred to as the material's permeability. The effective porosity is the percentage of that aquifer from which water can be retrieved (not all of the water could be withdrawn). Porosities that are developed during the formation of the rock are referred to as primary porosities and porosities that are formed by some activity after the rock mass has formed, such as faults, are referred to as secondary porosities.

Vesicles, cooling fractures, and flow breccia within the basalt units provide the primary porosity. Subsequent faults and tectonic deformation may locally provide higher secondary porosities. Joint fractures, perpendicular to the flow boundaries do not tend to increase the effective porosity significantly. The vesicular interflow zones tend to be laterally extensive but may be truncated faults that clayey gouge or have been subject to clay mineralization. The massive entablature/ colonnade structure that forms the bulk of the basalt units tends to have extremely low porosity with even less effective porosity, due to a lack of open interconnected fractures in the blocks.

According to current hydrogeologic literature, the effective porosity of fractured basalt ranges from 5 percent to 35 percent. Massive dense basalt typically have an effective porosity of 0.5 percent or less, although it may contain abundant cooling fractures. The effective porosities for productive interflow zones are generally considered the equivalent to unconsolidated gravels and range from 20 percent to about 30 percent A CRBG unit can have effective porosities spanning this whole range within the same unit.

4.2 Hydraulic Conductivity and Transmissivity

The term hydraulic conductivity refers to the water transmitting characteristic of an aquifer in quantitative terms. A measurement of hydraulic conductivity is a coefficient of proportionality that describes the rate at which water can move through a permeable medium under a given head. Hydraulic conductivities are typically measured in a unit of lateral movement during a given time period, such as feet per day.

The transmissivity of an aquifer is a measure of the capacity of an aquifer to yield water and hence, is directly related to an aquifer's hydraulic conductivity. In order to estimate the transmissivity of an aquifer, the hydraulic conductivity is simply multiplied by the aquifer's thickness. Transmissivity can be also derived from the analysis of an aquifer pump test. The transmissivity of an aquifer is the rate at which water is transmitted through a unit width of an aquifer under a unit hydraulic gradient and transmissivity values are typically presented in gallons per day (gpd) per foot.

Squier Associates has analyzed the semi-logarithmic plot of time against drawdown for the 1,100 gpm aquifer pumping test conducted on Well No. 6 in 1997 by Schneider. Using the Cooper-Jacob formula approximating the Theis non-equilibrium equation, we derived a transmissivity estimate of 65,000 gpd per foot. This value is within the range of other CRBG aquifers developed for municipal ground water supply in western Oregon.

Aquifer test data sufficient to estimate aquifer transmissivity at the other Sherwood well locations do not exist. Empirical relationships between specific capacity and transmissivity (Driscoll, 1986) can be used to estimate transmissivity using the following equation:

$$Q/2 = T/2000 \text{ (for confined aquifers)}$$

Using this method of estimating, it appears that the transmissivity of the basalt aquifer at Well No. 3 is greater than 100,000 gpd per foot. This estimated transmissivity value is greater than the calculated transmisssivity value for Well No. 6. Using the same estimation techniques, transmissivity values on the order of 4,000 gpd per foot to 5,000 gpd per foot were derived for Well No. 4 and Well No. 5.

4.3 Storativity and Specific Yield

The storativity (or storage coefficient) of an aquifer is that volume of ground water released from or added to a unit cross-sectional area due to the measured decline or increase in average hydraulic head. The specific storage capacity of an aquifer is dimensionless. Storativity considers the compressibility and elasticity of the aquifer and water and the release or addition of water from draining or refilling the open pore volume of a confined aquifer. The storage term used for unconfined aquifers is specific yield. Effective porosity is roughly equivalent to specific yield for unconfined aquifers.

Storativity is a coefficient that is equal to the volume of water an aquifer releases from storage per unit surface area of the aquifer, per unit change in head. Confined aquifers, such as the CRBG aquifers, typically have a low storage coefficient, ranging from 1 $\times 10^{-3}$ to 1 $\times$ 10^{-5} (dimensionless). Drawdown data from an observation well during a controlled aquifer test are required to calculate storativity, and to our knowledge no such data exist for the Sherwood wells.

4.4 Ground Water Flow

Ground water flow direction is simply estimated based upon an evaluation of water head (surface) elevations. Flow is always from high head (elevations) to low head. More exact predictions of ground water flow direction requires measured elevations of water level or potentiometric head from three or more points at reasonably contemporaneous times. Regional ground water flow can be determined by evaluation of a flownet consisting of ground water equipotential and flow lines. The equipotential lines represent contours of similar ground water elevations. Flow direction is perpendicular to these equipotential lines.

Variable controls of the ground water flow include the physical character of the medium, such as heterogeneities, anisotrophies and impermeable boundaries. Furthermore, seasonal fluctuations in the water table introduce transient effects in the flow system. The regional ground water flow may be evaluated while taking into consideration the controlling variables.

In the Sherwood area, the ground water flow direction could be assumed to be from the Parrett Mountain highlands (to the south) northwards towards the Tualatin River. Ground water would also flow from the Chehalem Mountains (to the north), southward and eastward also towards the Tualatin River. Local variations to this regional pattern will be expected based on the locations of various surface features such as creeks, streams, gullies, and lakes and subsurface features such as more permeable beds and faults.

In order to develop a better understanding of the ground water flow direction, the estimated elevations for the top of casing of the water supply wells was estimated from a USGS topographical map of the Sherwood area. Estimations of the water table elevations were then made based on water level information supplied by the City for the summer of 1998. Based on this information it appears that the ground water located east of the City, and east of Chicken Creek, flows towards the north northwest (approximately N20°W) and the ground water west of the City, and west of Chicken Creek, flows to the northeast (approximately N60°E). These flow directions closely approximate the surface topography for the respective areas. These rough estimations do not factor in the complexity required by the presence of the Sherwood Fault and should be considered subordinate to more site specific data.

The hydraulic gradient is the measurement of vertical change over a unit horizontal distance, usually presented as feet per foot. Using the same data and assumptions (with the same limitations) that were used to estimate the flow directions, an hydraulic gradient has been calculated. The ground water in Sherwood's wellfield area is estimated to be moving at a gradient of about 0.01 feet per foot.

4.5 Ground Water Velocity

Specific discharge, also known as Darcy's velocity, is the product of the hydraulic conductivity of a particular geological medium and the hydraulic gradient. The velocity of flow is proportional to the hydraulic gradient. The specific discharge is a macroscopic conceptualization of the ground water flow rate. It should be noted that Darcy's Law is valid for most granular materials and that other principles apply in a fractured media. However, as stated previously, the aquifer characteristics of the laterally extending interflow zones are more similar to the aquifer characteristics of gravel zones than massive fractured geologic media.

Schneider's 1997 aquifer test data for Well No. 6, our porosity assumptions, and the estimated ground water flow gradient can be used as variables for the ground water flux equation for Darcy velocity. By placing these variables into the equation, the Darcy velocity for the ground water is calculated to be about 11.5 feet per day if 20 percent porosity is assumed. If 30 percent porosity is assumed the ground water velocity is about 7.7 feet per day. The calculated Darcy velocity for gravels, using published data on gravels, also equals about 11.5 feet per day. Calculations made using published data on massive and fractured basalt produces a Darcy velocity ranging from 0.0004 feet per day to 0.3 feet per day, revealing that water velocity through the interflow zones is significantly more rapid than through the main rock mass.

4.6 Ground Water Recharge and Discharge

Ground water systems tend to be in a dynamic state of equilibrium, or balance, in the natural state. The water balance is based on the principle that the water into the system (recharge) minus the water out of the system (discharge) is equal to variations in storage or normal water level fluctuations. The normal water level fluctuations for the City of Sherwood well field have been observed and recorded intermittently since 1979. The ground water levels, as measured by the City of Sherwood, generally indicate seasonal trends. There are three main influences on the water level of the City of Sherwood well field:

- pumping rate of the well field,
- rate of recharge, and
- seasonal precipitation.

Recharge to the basalt aquifers occurs through exposed flowtops, fault zones, along alluvium/basalt or colluvium/basalt boundaries, and by vertical migration through the surficial soils and into the underlying basalt flows. It should be noted that a fractured basalt aquifer possesses an anisotropic character and therefore local characteristics should not be assumed to be regional in extent. Discharge from the basalt aquifer system occurs through ground water pumped for municipal and domestic use, in addition to the natural out-flow from the system, such as in springs, and into stream channels.

Faults can act as zones of enhanced recharge or discharge or barriers. The presence or absence of gouge (crushed and ground up rock), hydrothermal alteration, brecciation of adjacent rock and type of fault (compressional or tensional) all have a significant influence on the effect on migrating water. A fault will be a conduit and allow the migration of recharging water if the fault is relatively young and does not contain significant fine gouge. As water migrates through the structure, clay infilling and mineralization occurs and the faults ability to transmit water becomes impaired. Accordingly older faults become barriers to ground water flow.

Wells that produce water from the alluvium and Hillsboro Formation would be primarily recharged from precipitation and surface water bodies. This is also the primary recharge mechanism for wells that produce from the shallow unconfined water-bearing zones of the CRBG. It is our understanding that Well No. 3 would fall into this category. The other water wells that produce from deeper CRBG water bearing zones are primarily recharged from upland areas where the basalt units outcrop or a near the surface at unconformable boundaries with alluvium. The recharge by downward vertical migration is slow due to the confining nature of the colonnade/entablature zones of the CRBG. Locally, the faulting present throughout the area may control recharge of the deeper units.

5.0 CONCEPTUAL HYDROGEOLOGIC MODEL

5.1 Key Elements

The Conceptual Hydrogeologic Model is a verbal and graphical representation of the known and assumed characteristics of the hydrogeologic system. The Conceptual Hydrogeologic Model will assist in the understanding of current processes and the prediction of others. The following sections present in detail the data sources and interpretations that formulated the basis of our working model of the City of Sherwood's well field hydrogeologic system.

The Conceptual Hydrogeological Model is based on the following key elements:

- The shallow subsurface consists of detrital sedimentary deposits, predominantly silt and clay, with some sand. This unit, considered overburden in this report, occasionally is lithified to form siltstone, claystone, or possibly even sandstone. The overburden sediments range in thickness from a few tens of feet to a few hundreds of feet.

- The overburden sediments are underlain by basalt of the CRBG. The CRBG represents a series of basaltic lava flows that entered the vicinity from the east. These rocks are predominantly massive, low permeability, crystalline rocks with relatively thin, yet laterally extensive, permeable interflow zones of vesicular and/or brecciated rock. Local geologic structures can significantly influence the hydrogeologic properties of these materials.

- Certain vesicular and/or brechiated interflow zones are capable of being highly productive aquifers. Sherwood's four water supply wells extend into and produce ground water from these zones.

5.2 Cross Sections

The understanding of the subsurface is facilitated by the use of geologic cross-sectional diagrams developed for the subsurface of the municipal well field area (refer, Figures 5 and 6). The location and orientations of each of the subsequent cross-sections is presented on Figure 3. Cross-section A-A' traverses the site in a general north-westerly direction (refer, Figure 5). Cross-section B-B' traverses the site in a general north-easterly direction (refer, Figure 6).

The geologic cross-sections were developed using well logs obtained from OWRD's files, published geologic maps, and a study completed by Dr. Marvin Beeson on the drill chips collected from Well No. 5 and Well No. 6 (refer, Appendix E). Initially, a well log inventory was completed for the area around Sherwood, Oregon. From this well log inventory, selected well logs were obtained that provided additional geologic information necessary to develop the cross sections. The selected well logs included the La Bahn well, the Seeley well, and the Crawford well.

All of Sherwood's four water supply wells initially penetrate recent alluvium. Thickness of the alluvium varies, generally being thickest towards the west (Well No.4). Well No. 3 (located just west of the Sherwood Fault) encounters a section of Hillsboro Formation below the recent alluvium. This unit is described as "sand rock" in the historical well log. The complete section of Portland Area CRBG units (including the Gingko unit and Sentinel Bluffs unit) underlie the Hillsboro Formation in Well No. 3. A very thin, weathered section of the Sentinel Bluffs unit is probably encountered below the recent alluvium in Well No. 6. Well No. 4 and Well No. 5 both encounter the Winter Water unit as the uppermost CRBG unit. The primary water bearing zone for Well No. 6 appears to be a pillow basalt zone of the lower Wapshilla Ridge unit. This unit is apparently also just barely tapped by the La Bahn well. Well No. 5 also apparently produces water from the Wapshilla Ridge unit, as well as the Ortley, Umtanum, and Grouse Creek units. However, Well No. 5 was not extended to a depth sufficient to encounter the major water bearing pillow basalt zone of the Wapshilla Ridge Unit. Based on our understanding, Well No. 4 produces from the Ortley, Umtanum, and Grouse Creek units, but does not extend to the Wapshilla Ridge unit. Well No. 3 produces from the Gingko and Sentinel Bluffs units, as well as the Hillsboro Formation.

5.3 Water Level Trends

As part of the hydrogeologic study, Squier Associates updated and modified hydrographs depicting water level trends in Sherwood's production wells. The hydrographs were originally generated by OWRD as part of Ground Water Report No. 40. An electronic version of the spreadsheets was used to initially generate the hydrographs. The hydrographs were then updated with water levels, pumping volumes (monthly), pump intake depths as provided by Sherwood. Monthly precipitation data from a nearby weather station (Rex 1S) was also collected and summarized. Figure 7 presents the monthly precipitation totals for each month from January 1979 to December 1998. Figure 7 also presents a summation of each years total annual precipitation. It should be noted that the period from about 1986 to 1993 was drier than normal, about 35.5 inches per year, and the period from 1994 to present (1999) has been wetter than normal, about 55.5 inches per year (refer, Figure 7). Figure 8 illustrates the relationship between precipitation and water levels.

Although we tabulated data going back to earlier than the original well construction, due to the relatively large data set, we elected to limit Figure 8 to 20 years (1979-1999). Figures 9 through 12 depict hydrographs for each production well, plotting monthly water levels against monthly pumping volumes.

Well No. 3 Trend Analysis. Figure 9 compares measured water levels in Well No. 3 to monthly pumping volumes in the same well for the recent period from October 1995 to April 1998. Included in Figure 9 is a comparison of static water level, measured as daily maximum water level over the sensor, to pumping level (as measured as daily minimum water level over sensor). Based on this figure, there is an approximate drawdown of 30 feet during pumping periods. During periods of high production, the pumping level has dropped below the sensor level (refer, Figure 9). The pumping volume information presented in Tables 2 and 3 suggest that the pumping rate increased about 25 percent from 1993 to 1997, however the 1998 pumping rates are similar to the 1993 pumping rates.

A review of a trend line associated with the static water levels for Well No. 3 suggests a decline in ground water levels of about 0.85 feet per year since 1979. However, a review of a trendline constructed in the water levels depicted in Figure 9, for the recent years, suggest that the recent increased pumping efforts have produced a water level decline of about 6 feet per year.

Well No. 4 Trend Analysis. Figure 10 displays the maximum daily water level (static level) and minimum daily level (pumping level) for Well No. 4 since October 1995. The relationship of the static water level and pumping level indicates that there is an approximate 165-foot drawdown for this well, suggesting a specific capacity of 2 gpm per foot. This agrees with the initial measurements of pumping and measured drawdown, as stated earlier.

The trendline for these two data sets suggests that no appreciable decrease in water levels is occurring. Accordingly it appears that declining static water levels in Well No. 4 are not sustained after pumping activities and have mostly rebounded to closely approximate prepumping levels. However, if the sparse data set from 69 to 1999 is evaluated, a decreasing trend of about 1.7 feet per year can be constructed. Further evaluation by selecting only the dates from March 1997 to March 1999 indicate a decrease of about 5 feet per year.

Well No. 5 Trend Analysis. Figure 11 shows the historical water level data versus monthly pumping rates. The distance between the static water level and pumping level is considerable in Well No. 5 (about 300 feet), suggesting a specific capacity of also about 2 gpm per foot. Well No. 5 water levels have declined about 2 feet per year since 1995, whereas pumping volume has increased by only approximately 8 percent.

Well No. 6 Trend Analysis. Well No. 6 has been online producing water for an insufficient time for the necessary level of certainty to be applied to the trend analysis. However, a relationship between rapidly decreasing water levels and elevated pumping rates can be observed in September 1998 (refer, Figure 12). During the period around August 1998 and October 1998 the pumping rate was raised as high as 0.9 mgd and the associated water levels dropped about 25 feet. Water levels returned to prepumping levels as pumping rates were lowered to about 0.2 mgd (refer, Figure 12).

Overall Trend in Water Levels Compared to Pumping Volume and Precipitation. In order to evaluate the overall trends, the precipitation data was compared to the water levels for each of the Sherwood water supply wells (refer, Figure 8). Further evaluation of the same data was undertaken by looking at the trendline developed using the mean water level from each measuring point for the water supply wells (refer, Figure 13). This figure suggests a decline of approximately 1.2 feet per year since 1961. Since the bulk of this data does not include periods when all four wells were in production, Figure 14 was prepared to display the mean water levels since 1992. The trendline for this recent data suggests a decline of 17 feet per year.

6.0 RELIABLE GROUND WATER YIELD ESTIMATES

This section discusses preliminary reliable ground water yield estimates developed from existing information. This reliability is different from what can be termed *system reliability*, which would take into account the potential for down time due to well or pump failure, and other components of the water system. Most water system operators report a 90 percent reliability factor, but this figure applies to ground water systems with built-in redundancies, i.e. backup wells, a system component that Sherwood currently does not have. To increase the overall reliability of the system (which is different than increasing yield or capacity) redundancy would need to be developed. This redundancy would allow for the institution of regularly scheduled well maintenance activities.

In order to evaluate the reliability of the ground water source, in terms of its sustaining the needs of the City of Sherwood we evaluated the water level trends. A number of these trends, based on different criteria, have been discussed in earlier sections of this report. Table 4 has been developed to summarize the trend analysis. Included in Table 4 are five trendlines, which are discussed in greater detail below. We have used the depth below surface, above the pumping level, that corresponds to the drawdown that the wells typically experience immediately at the start of pumping activities. This level has been termed the "Critical Level".

6.1 Declining Trend Determined From Water Levels From Period 1962 to 1999

In order to evaluate the overall trend of water levels in the vicinity of Sherwood municipal ground Water Well Field, we plotted out the mean water level, based on the available water levels for each month since 1962 (refer, Figure 13). The declining trend determined from this plot was then used to calculate the estimated depth to water in the year 2010. An additional estimate was then made as to what year the water level would reach the Critical Level (as defined above).

If we presume the declining trend observed since 1962 is constant and will continue for the next ten years, then it appears that all four municipal ground water supply wells should be capable of producing a reliable yield until approximately the year 2030 and beyond (refer, Table 4). This prediction presumes no other significant increase in production from the basalt aquifer in the study area beyond current levels.

6.2 Declining Trend From Mean Water Levels From Period 1992 to 1999

The reported water level trends suggest that the decline is not uniform over time, with an apparent increased rate of decline during the recent years. This may be due to the fact that more wells are producing more water from the aquifer's limited supply. In order to evaluate the decline based on declining trends determined from recent water level data, the mean water levels for the wells since 1992 was plotted (refer, Figure 14). The declining trend for this set of data is about 17 feet per year.

This declining trend factor was then used to calculate the estimated water level for the year 2010 and the estimated year the Critical Level would be reached (refer, Table 4). Since this approach weighted data from wells showing significant decline equally with wells showing insignificant declines, the decline rate is considered very conservative and probably higher than should be expected. The estimated years that the Critical Levels are breached are "worst-case" scenarios for this evaluation.

6.3 Declining Trend From Full Set of Well-Specific Data

Since Well No. 3 produces from a different water bearing zone than the other three wells and Well No. 6 also produces from a water bearing zone untouched by the other three wells it is important to look the well-specific water level trends. The declining water level trend for each well was calculated using all of the available data for each well, however, large data gaps exist. Based on this analysis, the following declining trends were determined:

- 0.85 feet per year for Well No. 3,
- 1.7 feet per year for Well No. 4, and
- 8.5 feet per year for Well No. 5.

As before, these declining trends were used to predict what the water levels will be in 2010 and at what year the Critical Level will be reached (refer, Table 4). Since Well No. 6 was not placed until 1997, there was no reason to include the well with this evaluation. The predicted water levels using this method are generally similar to the levels predicted using the mean water levels from 1962 to 1999. A key difference is the extension of time for Well No. 3, whereas Well No. 4 and Well No. 5 reached Critical Level's the soonest.

6.4 Declining Trends Based on 1993 to 1999 Well-Specific Water Levels

The City of Sherwood has a Supervisory Control and Data Acquisition (SCADA) system placed online in 1993 for its wells. The data from this SCADA system was downloaded, placed into spreadsheets, and subsequently graphed (refer, Figures 9 through 12). Well specific trends were determined from each of the SCADA water levels and used to predict the water levels in the year 2010 and the year when the Critical Level would be reached (refer, Table 4). The following conservative decline rates were used for this determination:

- 6 feet per year for Well No. 3,
- 2 feet per year for Well No. 4, and
- 2 feet per year for Well No. 5.

6.5 Declining Trends Based on 1997 to 1999 Well-Specific Water Levels

Since there is an apparent increase in the rate of decline in the recent years only the SCADA data from 1997 to 1999 were used to evaluate the rate of decline trends for the four water wells. This data is also tabulated in Table 4. In summary, the following rates of decline were determined:

- 16 feet per year for Well No. 3,
- 5 feet per year for Well No. 4, and
- 10 feet per year for Well No. 5.

In addition, the SCADA data was evaluated for Well No. 6. This evaluation showed no appreciable decline trend in the Well No. 6 water levels.

6.6 Time - Drawdown Calculation for Well No. 6

Since an insufficient amount of time exists for the development of trends from the Well No. 6 data, an alternative approach was undertaken. As discussed previously, Well No. 6 has good specific capacity and produces from a water bearing zone that has sufficient transmissivity to support well pumping rates in the range of 500 to 1,000 gpm. Using the estimated transmissivity value of 65,000 gpd per foot, time-drawdown calculations may be used to project long-term drawdowns produced by pumping. At present pumping rates (about 600 gpm), it appears that the well could be pumped continuously for up to 90 days with less than 40 feet of drawdown in the well.

Increasing the pumping rate to 1,000 gpm would increase the 90 day drawdown to approximately 55 feet.

6.7 Well No. 6 Interference

Another concern from pumping Well No. 6 is its affect on water levels in the other wells. Well No. 6 is not 100 percent efficient, meaning that drawdown in the aquifer nearby to the well is not equal to the well drawdown during pumping. Specific capacity data indicate the well is moderately efficient, on the order of 65 percent. Therefore, 40 feet of drawdown in the well means aquifer drawdown is approximately 32 feet just a few feet from the well bore.

Assuming that well drawdown is equal to 40 feet in a 65 percent efficient well and the drawdown is 26 feet in a uniform, homogeneous aquifer (which is unlikely) by projecting a semi-logarithmic graph of drawdown versus the log of the distance from the well, aquifer drawdowns at selected distances can be estimated. Also by using the Jacob equation for distance-drawdown, and the same assumptions, we have calculated the following:

- Aquifer drawdown 1,000 feet from Well No. 6 = 16 feet
- Aquifer drawdown 10,000 feet from Well No. 6 = 11 feet

The above estimates do not take into account variations in aquifer transmissivity (either higher or lower values) which will have an effect on the drawdown response. The estimates also do not include an analysis of well interference. The above analysis is considered quite conservative as it is unlikely that Well No. 6 would be pumped continuously for 90 days. What the above estimates indicate is that drawdown responses may propagate considerable distances from pumping wells.

An initial concern at the start of this study was a perceived well interference from Well No. 6. There was a noted rapid decrease in water levels as observed in Well No. 3 beginning in 1997, the year Well No. 6 was brought online. By comparing the graph of SCADA pumping rates and SCADA water levels for the two wells it becomes apparent that the decrease in water levels in Well No. 3 occurred during a period when it was being pumped at an elevated rate (refer, Figure 15).

6.8 Predicted City Water Supply Requirements

The declining trend lines discussed above are based on the City of Sherwood not significantly increasing their pumping requirements. If the City of Sherwood allows growth at the same rate as growth since 1993, the City's water needs will also increase. Figure 16 has been prepared to present a simplistic prediction of what level of water production Sherwood would need to meet the increased growth. According to this straight line method, the City must increase its water production 82 percent by the year 2010 to meet the projected water need. An increase of 82 percent would greatly stress the existing water system and shortages would probably occur.

We also looked at the available supply for the City's water needs, using the 1998 well-specific water production volumes listed in Table 2 and predicted water levels listed in Table 4 (refer, Table 5). Table 5 presents preliminary estimates of water supply production totals for the years 1998 to 2010, based on assumptions made using the information presented in this report. The estimates are presented according to five scenarios (refer, Table 5).

Scenario 1 presents the "status quo", no changes are made to the system and water production rates for each well Is static. If this condition continues, and the predictions listed in Table 4 are valid, in the year 2001 the City's water production will be curtailed by 31 percent. An additional curtailment will occur in the year 2004 when Well No. 5 reaches a critical level and the City's water production ability becomes about 44 percent of its 1998 production.

Scenario 2 considers mitigation efforts in Well No. 3 involving lowering the pump in the well. If the pump is lowered as the water is lessened due to the declining water levels, the well's production capacity will also be reduced. Without empirical well-specific data, the amount of reduction in the well's production must be assumed. Our review suggests that if the pump is lowered about 100 feet, the production will be roughly halved. Based on this critical assumption, in the year 2000, the City will be producing about 85 percent of its 1998 water production. Further reductions in water production will occur in the year 2004 (down to about 60 percent of the 1998 production). The lowering of the pump and reduction of the pumping capacity of Well No. 6 effectively allows an additional six years of use from the well. The Scenario 2 model predicts that in the year 2007, the City's water production could be about 44 percent of Its 1998 production.

Scenario 3 involves the recommended deepening of Well No. 5. Our assumption is that the deepened well would then be capable of producing the equivalent to Well No. 6. By just deepening Well No. 5 and not lowering the pump in Well No. 3, the City's water production capacity would increase 8 percent in the year 2000, then drop to 76 percent in the year 2001.

Scenario 4 combines both lowering the pump in Well No. 3 and deepening Well No. 5. The same assumptions as used in Scenarios 2 and 3 are used in Scenario 4. Based on these assumptions, we estimate that the water production would drop about 8 percent in the year 2000 and then drop to about 76 percent of the 1998 production in the year 2007.

The last scenario (Scenario 5) combines Scenario 4 with the added production potential of the Spada well. Since no specific capacity data is available on the well log for the Spada well, we made some assumptions. The well log did report that a 400 gpm air lift well test was performed for 4 hours at depth 240 feet (refer, Appendix E). Since the reported static water level was 20 feet and if we assume the well test was stopped when the drawdown reached the pump, there would be 220 feet of drawdown. This translates to a specific capacity of about 1.8 gpm per foot, the equivalent of Well No. 4. Using these critical assumptions, we based our predictions on the Spada well producing the same amount of water as Well No. 4. Based on these assumptions, the City's water production would increase by about 4 percent until the year 2007. In the year 2007, Well No. 3 will lose its ability to pump water and the City's production will be about 13 percent less than its 1998 production.

These predictions are made using simplifying assumptions. In order to further evaluate the well field production capability with greater confidence, more empirical data is necessary. The predictions presented herein have been developed using only readily available, and in some cases incomplete, data. The predictions are being provided to assist the City in making informed decisions, however, the interpretations and predictions should be considered only rough estimations. The accuracy for the included predictions is susceptible to many variables. Variables outside of the City's control include the depletion of water levels due to the use of the ground water by others and unforeseen changes in the subsurface geology. If other users increase or decrease their use, the rates of decline will adjust accordingly. Other variables include dynamic rates of change, which could be caused by an as yet undetermined effect, and a time lag for recharge efforts.

7.0 WATER SUPPLY OPTIONS

The information presented in this report suggests that the City of Sherwood may need to further evaluate alternative water supply options to meet projected needs. We describe herein four possible water supply options that could augment the City's water supply. The four options are:

- Adding new wells
- Transfer of existing water rights from another suitable ground water source
- Aquifer storage and recovery
- Purchase surface water from another source.

7.1 Adding New Wells

The City of Sherwood could expand their current wellfield's production by adding new wells. In order to limit the degree of interference on the existing wells, the City should focus on placing deep wells into fault blocks and untapped water bearing zones that are not currently producing water from existing wells. Examples may be the fault block north of Well No. 6 and the fault block south of Well No. 5.

Alternative locations that present a greater potential for well interference include placing a deep well in the vicinity of Well No. 3 or in the Tualatin Valley area, north of the city. These wells would have the potential of accelerating the depletion of the deep aquifers that are already being used for production.

One key element that makes this option less favorable is the expressed position of the OWRD. OWRD has stated that the City of Sherwood should become less reliant on ground water. If additional wells are considered, additional consultation and negotiation with the OWRD would likely be required prior to expending more resources on this water supply option.

7.2 Water Rights Transfers

A second water supply option would involve the City of Sherwood purchasing land that has a water well and obtaining the water rights for that well. Certificates of water rights are directly transferred with property transactions, although the water rights are based on use. The water uses typically associated with a water right are domestic, municipal, and irrigation.

Domestic water wells used for single farmsteads are exempt from the need for a water right and therefore, typically there is not a water right for those properties. Water use for irrigation purposes of greater than 0.5-acre does require a water right, but the water right is solely for a set volume to be used as irrigation on a given acreage. Municipal water use requires a water right.

Water rights are obtained by the application for a permit of water use. Permits of water use generally take about 9 months to obtain, if the process is uninterrupted by changes. There are a number of stages to the application process including the final certification process. Water use can be undertaken once a permit is granted during the certification process.

Based on our discussions with OWRD, the City of Sherwood would need to apply for a permanent transfer to add wells as additional points of appropriation. It appears that a permanent transfer would provide greater benefit to the City than a temporary transfer (for up to 5 years). In conferring with Mr. Kelly Starnes of OWRD, a Certified Water Rights Examiner (CWRE) would be required to complete the application map. The application would be reviewed by OWRD technical staff and would be subject to a public comment and review period.

7.3 Aquifer Storage and Recovery

Aquifer Storage and Recovery (ASR) is a specialized subset of the more general term artificial recharge. It is an emerging solution that can be viewed as "water banking". ASR technology involves injecting drinking water (typically from a surface water source) into the ground via wells, storing the water underground for a period of time, and withdrawing the water when needed (typically, summer). ASR is an option that is currently being evaluated by a number of water providers in western Oregon. It is a process that can be used to store water underground for reuse, raise ground water levels to reduce pumping costs, and for water quality improvements.

Key concepts in a feasibility evaluation for ASR are the following:

- suitability of potential source water,
- suitability of aquifers to store water, and
- the ability to retrieve the water when it is needed.

Additional feasibility concerns include the potential chemical interaction between the recharge water and the host rock. In preliminary review, the presence of the well field in basalt with declining head shows that there is a suitable aquifer and that there is probably sufficient room to store the water.

Also the discrete faulted blocks of the CRBG underlying the Sherwood Municipal Well Field provide a positive scenario for containing ASR-injected water. The faulted blocks have inherent boundary conditions. In addition, the basalt is a preferred host rock since it is relatively inert and minerals are not readily leached from exposed surfaces.

Potential source waters during the winter months could come from one of several sources. These include Bull Run, Clackamas River, and the Trask / Tualatin Rivers. During the winter months, these water providers tend to have excess water and under utilize their water treatment capacity.

Oregon regulates ASR under Oregon Administrative Rules (OAR) 690 Division 350. In brief, a limited license is provided to conduct ASR Pilot studies after an application is submitted. The application would include the submittal of a preliminary hydrogeologic characterization. After the evaluation of the Pilot Test data a full ASR Permit could be presented. This process is expected to take from 3 to 10 years. There currently are two ASR Limited Licenses in the preliminary pilot test stage in Oregon.

7.4 Purchasing Water From Other Source

This option is very complex and requires evaluation from a multitude of perspectives. It is our understanding that the City of Sherwood is currently evaluating aspects of this option. The study of this option is outside the scope of this hydrogeologic characterization. This option will not be discussed further in this report.

8.0 CONCLUSIONS AND RECOMMENDATIONS

Squier Associates has completed this Hydrogeologic Evaluation for the City of Sherwood, Oregon's Municipal Well Field. The work has been conducted in accordance with generally accepted hydrogeologic practices and included such tasks as were considered necessary, in Squier Associates' professional judgement, to enable us to evaluate and prepare conclusions for the well field. After an evaluation of the available data, Squier Associates has developed the following recommendations concerning the site.

1. Well No. 3's water production zone begins in uncased sandstone of the Hillsboro Formation. This is the geologically youngest producing zone of the Sherwood Municipal Well Field system. The well then extends about 202 feet into basalt, of which about 77 feet produces all the water. The basalt encountered by Well No. 3 belong to the Gingko and Sentinel Bluffs Units, the youngest of the CRBG basalt flows in the Portland Basin. Sherwood's other municipal water supply wells do not produce from these units. A benefit to using the shallow water bearing zones is the probable higher ability for aquifer recharge from surface migration. A detriment for the shallow water bearing zones is the vulnerability to contaminant sources.

   Trend analyses on sets of water levels for Well No. 3 show rates of decline ranging from 0.85 feet per year to 17 feet per year, depending on the time span evaluated. It is our opinion that the use of the greater rates of decline is more appropriate since these provide a more conservative approach and also reflect more current conditions.

   Based on this conclusion, future efforts at routine monthly water level collection should be closely monitored and graphed, so that the actual trends can be compared to our preliminary predictions. It should be noted that the addition of new pumping wells which pump year round (as opposed to irrigation wells, which are pumped only for relatively short periods in summer) may contribute to a further decline in water levels.

2. Well No. 4 produces ground water from the Winter Water, Ortley, and Umtanum Units of the CRBG. The well has a low specific capacity (2 gpm per foot) and is considered a low producer. The hydrographs for Well No. 4 indicate a trend of slightly declining water levels. However, peak season monthly pumping volumes on the order of 4 to 7 million gallons appear to be sustainable.

3. Well No. 5 is also constructed to produce water from the Winter Water, Ortley, and Umtanum Units of the CRBG. In addition, the well also extends to depth sufficient to tap into the Grouse Creek and Wapshilla Units. However, it appears that the well terminated about 20 feet above a primary water-bearing pillow basalt zone of the Wapshilla Ridge Unit. This correlation is based on information obtained from Well No. 6. A videocamera survey has shown that a cascading water zone exists at about depth 253 feet. This corresponds to the basal flow unit of the Winter Water unit.

   The water level data for this well indicates a relatively large well drawdown occurs (about 300 feet) when pumping is commenced. The trend line analyses for this well suggests that there is a likely risk that water levels will decline to a level inadequate to produce the volume of water required by the City of Sherwood prior to the year 2010. A reduction in the design yield and overall pumping volume from this well may be warranted, as further decline in the static water level could result in drawdown below the pump intake of 430 feet.

   In our opinion, enhanced production capacity could be provided by deepening the well an additional 60 to 100 feet in order to penetrate and produce from the projected pillow basalt zone. Well No. 5 is on the opposite side of the Sherwood Fault from Well No. 6 so there should not be well interference because the fault will provide boundary conditions. It is also our opinion that the cascading water zone at depth 253 feet should be cased off. This action would improve the well's efficiency and also provide a check on the decline of an upper water bearing zone, possibly providing an incentive to the OWRD to allow greater extraction from other zones.

4. Well No. 6 is an excellent water production well. If possible, permit adjustments should be undertaken so that the well production could be increased. An application for water rights transfers from existing wells owned by the City would be necessary. For this transfer, consultation with a CWRE who has experience with municipal well systems is recommended.

5. To increase the overall reliability of the system (which is different than increasing yield or capacity) redundancy should be developed. This redundancy would allow for the institution of a regularly scheduled well maintenance activities. The redundancy could be accomplished by either constructing new wells or purchasing other existing wells. It may be possible to apply for a water rights transfer for the other existing wells, should

Sherwood elect to go ahead and purchase these well properties. The transfer would cover type of use (from presumed irrigation to municipal) and point of diversion. A separate application from the existing well transfer would be necessary and consultation with a CWRE would also be required.

6. Before purchasing the Spada Farm Well and its associated water rights, Sherwood should conduct preliminary aquifer and well performance testing to evaluate the yield potential and water quality. At a minimum, a series of specific capacity (step) tests, followed by an 8 hour pumping test should be performed, with the assistance of a hydrogeologist and well drilling contractor. The Spada well in particular appears to be in a good location in terms of proximity to the existing water transmission system (while being far enough away from Sherwood's other wells to minimize interference effects) and had a good reported initial well yield. Should any testing be done on the well, it will be important to monitor response in Well No. 3, Well No. 4 and Well No. 6.

7. A potential water supply solution for Sherwood to consider is ASR. The preliminary review of site characteristics are favorable to an ASR approach to water supply. This could be accomplished by having the water system connected to a regional water supply system. The general approach would be to recharge and store water during the winter season, and extract the stored water during the summer season, ideally at higher production rates than presently achievable without recharge. With well modifications, and augmented storage, it is possible to increase system capacity significantly using this approach. At the City's option, a feasibility analysis of ASR could be undertaken.

8. These preliminary recommendations should be re-evaluated if and when any additional wells are added to the Sherwood system and pumping begins. It is important to note this potential pumping would result in an additional stress on the ground water system that could over time, increase the rate of water level decline.

9.0 REPORT LIMITATIONS

The scope of the investigation presented herein is a Hydrogeologic Evaluation of a Municipal Well Field for the City of Sherwood, Oregon. This report has been prepared to present the results of our evaluation that has been provided by others. Squier Associates cannot verify the validity of the data obtained by others. Squier Associates' conclusions and recommendations are based on this limited available data and observations described herein, and on the assumption that subsurface conditions in other portions of the well field are not significantly different from those disclosed by this study and inferred from the data. However, there remains a level of risk that unforeseen conditions may exist that may not become apparent until later. This level of risk could be reduced, but not eliminated, through systematic subsurface exploration, sampling, and testing.

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Table 1
Well Construction Details

	Year Completed	Total Depth (feet)	Casing Depth (feet)	Casing Diameter (inches)	Pumping Depth (feet)	Specific Capacity (gpm per foot)	Transmissivity (gpd per foot)
Well No. 3	1946	339	77	12	130	44	100,000 (estimated)
Well No. 4	1969	458	99	14	400	2.2	4,000 (estimated)
Well No. 5	1984	800	252	16	430	2.4	4,000 (estimated)
Well No. 6	1997	889	300	16	300	48	65,000

gpm = gallons per minute
gpd = gallons per day

Table 2
Historical Pumping Rate Summary
(volume in million gallons)

	January	February	March	April	May	June	July	August	September	October	November	December	Cumulative	Average
Well No. 3														
1993	7.0	7.0	8.6	12.6	10.1	11.1	12.9	20.9	12.4	6.0	6.9	6.6	122.1	10.2
1994	9.1	7.8	9.1	8.7	10.7	13.7	13.8	13.9	13.4	9.1	6.8	7.2	123.3	10.3
1995	8.7									9.7	8.6	8.4	35.4	8.9
1996	8.7	7.3	8.9	9.9	15.5	26.3	17.2	15.5	12.8	9.7	8.6	8.4	148.8	12.4
1997	12.0	7.4	9.0	9.9	15.5	26.4	17.3	15.6	12.9	10.1	8.9	10.1	155.1	12.9
1998	12.2	7.5	7.3	8.2	8.3	11.7	16.8	17.0	11.8	7.9	7.3	9.3	125.3	10.4
1999	8.1	6.6	6.9										21.6	7.2
Well No. 4														
1993	0.0	2.9	3.8	3.2	4.2	4.6	4.9	6.7	4.3	2.4	1.6	2.2	40.8	3.4
1994	3.6	3.0	3.9	3.5	4.2	5.4	4.8	4.7	4.5	3.1	2.5	3.0	46.2	3.9
1995	3.4									3.1	3.2	2.9	12.6	3.2
1996	3.6	3.2	3.8	3.7	4.3	5.7	6.1	5.3	4.3	3.7	3.4	4.0	51.1	4.3
1997	3.8	3.6	4.3	4.3	5.7	5.4	6.5	3.7	3.7	3.5	3.7	3.4	51.6	4.3
1998	4.5	3.1	3.0	3.2	3.3	4.4	5.3	4.4	3.1	2.6	2.6	2.1	41.6	3.5
1999	3.0	2.7	2.8										8.5	2.8
Well No. 5														
1993	5.3	4.3	5.9	3.6	6.1	7.3	7.7	12.2	7.1	4.6	3.8	4.1	72.0	6.0
1994	5.3	4.9	5.1	5.8	7.1	8.7	8.5	10.0	9.2	5.1	4.1	4.8	78.6	6.6
1995	5.9									6.0	5.8	5.3	23.0	5.8
1996	6.1	5.1	5.8	6	6.5	9.7	11.5	10.7	9.1	7.0	6.3	7.1	90.9	7.6
1997	6.8	6.5	7.4	7.5	10.5	10.5	14.8	9.7	8.7	6.8	7.2	6.7	103.1	8.6
1998	6.2	5.5	6.2	6.9	7.0	9.3	12.8	12.9	10.1	6.6	6	7.4	96.9	8.1
1999	6.8	5.3	5.6										17.7	5.9
Well No. 6														
1997										7.2	6.3	6.5	20.0	6.7
1998	6.9	6.9	6.8	7.3	8.2	10.7	18.3	21.3	19.4	7.5	6.6	8.1	128.0	10.7
1999	7.2	5.6	5.9										18.7	6.2

Table 3
Historical Pumping Rate Summary
(volume in million gallons per day equivalents)

	January	February	March	April	May	June	July	August	September	October	November	December	Cumulative	Average
Well No. 3														
1993	0.23	0.25	0.28	0.42	0.33	0.37	0.42	0.67	0.41	0.19	0.23	0.21	4.0	0.33
1994	0.29	0.28	0.29	0.29	0.35	0.46	0.45	0.45	0.45	0.29	0.23	0.23	4.1	0.34
1995	0.28									0.31	0.29	0.27	1.2	0.29
1996	0.28	0.26	0.29	0.33	0.50	0.88	0.55	0.50	0.43	0.31	0.29	0.27	4.9	0.41
1997	0.39	0.26	0.29	0.33	0.50	0.88	0.56	0.50	0.43	0.33	0.30	0.33	5.1	0.42
1998	0.39	0.27	0.24	0.27	0.27	0.39	0.54	0.55	0.39	0.25	0.24	0.30	4.1	0.34
1999	0.26	0.24	0.22										0.7	0.24
Well No. 4														
1993	0.0	0.10	0.12	0.11	0.14	0.15	0.16	0.22	0.14	0.08	0.05	0.07	1.4	0.11
1994	0.12	0.11	0.13	0.12	0.14	0.18	0.15	0.15	0.15	0.10	0.08	0.10	1.5	0.13
1995	0.11									0.10	0.11	0.09	0.4	0.10
1996	0.12	0.11	0.12	0.12	0.14	0.19	0.20	0.17	0.14	0.12	0.11	0.13	1.7	0.14
1997	0.12	0.13	0.14	0.14	0.18	0.18	0.21	0.12	0.12	0.11	0.12	0.11	1.7	0.14
1998	0.15	0.11	0.10	0.11	0.11	0.15	0.17	0.14	0.10	0.08	0.09	0.07	1.4	0.11
1999	0.10	0.10	0.09										0.3	0.09
Well No. 5														
1993	0.17	0.15	0.19	0.12	0.20	0.24	0.25	0.39	0.24	0.15	0.13	0.13	2.4	0.20
1994	0.17	0.18	0.16	0.19	0.23	0.29	0.27	0.32	0.31	0.16	0.14	0.15	2.6	0.22
1995	0.19									0.19	0.19	0.17	0.7	0.19
1996	0.20	0.18	0.19	0.20	0.21	0.32	0.37	0.35	0.30	0.23	0.21	0.23	3.0	0.25
1997	0.22	0.23	0.24	0.25	0.34	0.35	0.48	0.31	0.29	0.22	0.24	0.22	3.4	0.28
1998	0.20	0.20	0.20	0.23	0.23	0.31	0.41	0.42	0.34	0.21	0.20	0.24	3.2	0.26
1999	0.22	0.19	0.18										0.6	0.20
Well No. 6														
1997										0.23	0.21	0.21	0.7	0.22
1998	0.22	0.25	0.22	0.24	0.26	0.36	0.59	0.69	0.65	0.24	0.22	0.26	4.2	0.35
1999	0.23	0.20	0.19										0.6	0.21

Table 4
Predicted Future Water Levels

Assumptions

	Pumping Level (feet below surface)	Critical Level (Pumping Level minus Drawdown) (feet below surface)	March 1999 Water Level (feet below surface)
Well No. 3	130	100	59
Well No. 4	400	235	78
Well No. 5	430	130	81
Well No. 6	300	275	88

Estimated Water Level in Year 2010

Based on Trendline Data From:

	1962 to 1999 Mean Water Levels (feet below surface)	1992 to 1999 Mean Water Levels (feet below surface)	Earliest Data to 1999 Levels (feet below surface)	1993 to 1999 SCADA Levels (feet below surface)	1997 to 1999 SCADA Levels (feet below surface)
Well No. 3	72	246	68	125	235
Well No. 4	91	265	97	100	133
Well No. 5	94	268	175	103	191
Well No. 6	101	275	not applicable	not applicable	89

Estimated Year to Reach Critical Level

Well No. 3	2033	2001	2047	2006	2002
Well No. 4	2130	2008	2091	2078	2030
Well No. 5	2040	2002	2005	2024	2004
Well No. 6	2155	2010	not applicable	not applicable	3869

TABLE 5

Predictions of Water Supply Based on Critical Assumptions (million gallons)

Scenario 1: No Chages to Current System

	1998	1999	2000	2001	2002	2003	2004	2005	2006	2007	2008	2009	2010
Well No. 3	125	125	125	0	0	0	0	0	0	0	0	0	0
Well No. 4	45	45	45	45	45	45	45	45	45	45	45	45	45
Well No. 5	100	100	100	100	100	100	0	0	0	0	0	0	0
Well No. 6	130	130	130	130	130	130	130	130	130	130	130	130	130
total	**400**	**400**	**400**	**275**	**275**	**275**	**175**	**175**	**175**	**175**	**175**	**175**	**175**

Scenario 2: Lower Pump In Well No. 3

	1998	1999	2000	2001	2002	2003	2004	2005	2006	2007	2008	2009	2010
Well No. 3	125	125	65	65	65	65	65	65	65	0	0	0	0
Well No. 4	45	45	45	45	45	45	45	45	45	45	45	45	45
Well No. 5	100	100	100	100	100	100	0	0	0	0	0	0	0
Well No. 6	130	130	130	130	130	130	130	130	130	130	130	130	130
total	**400**	**400**	**340**	**340**	**340**	**340**	**240**	**240**	**240**	**175**	**175**	**175**	**175**

Scenario 3: Deepen Well No. 5

	1998	1999	2000	2001	2002	2003	2004	2005	2006	2007	2008	2009	2010
Well No. 3	125	125	125	0	0	0	0	0	0	0	0	0	0
Well No. 4	45	45	45	45	45	45	45	45	45	45	45	45	45
Well No. 5	100	100	130	130	130	130	130	130	130	130	130	130	130
Well No. 6	130	130	130	130	130	130	130	130	130	130	130	130	130
total	**400**	**400**	**430**	**305**	**305**	**305**	**305**	**305**	**305**	**305**	**305**	**305**	**305**

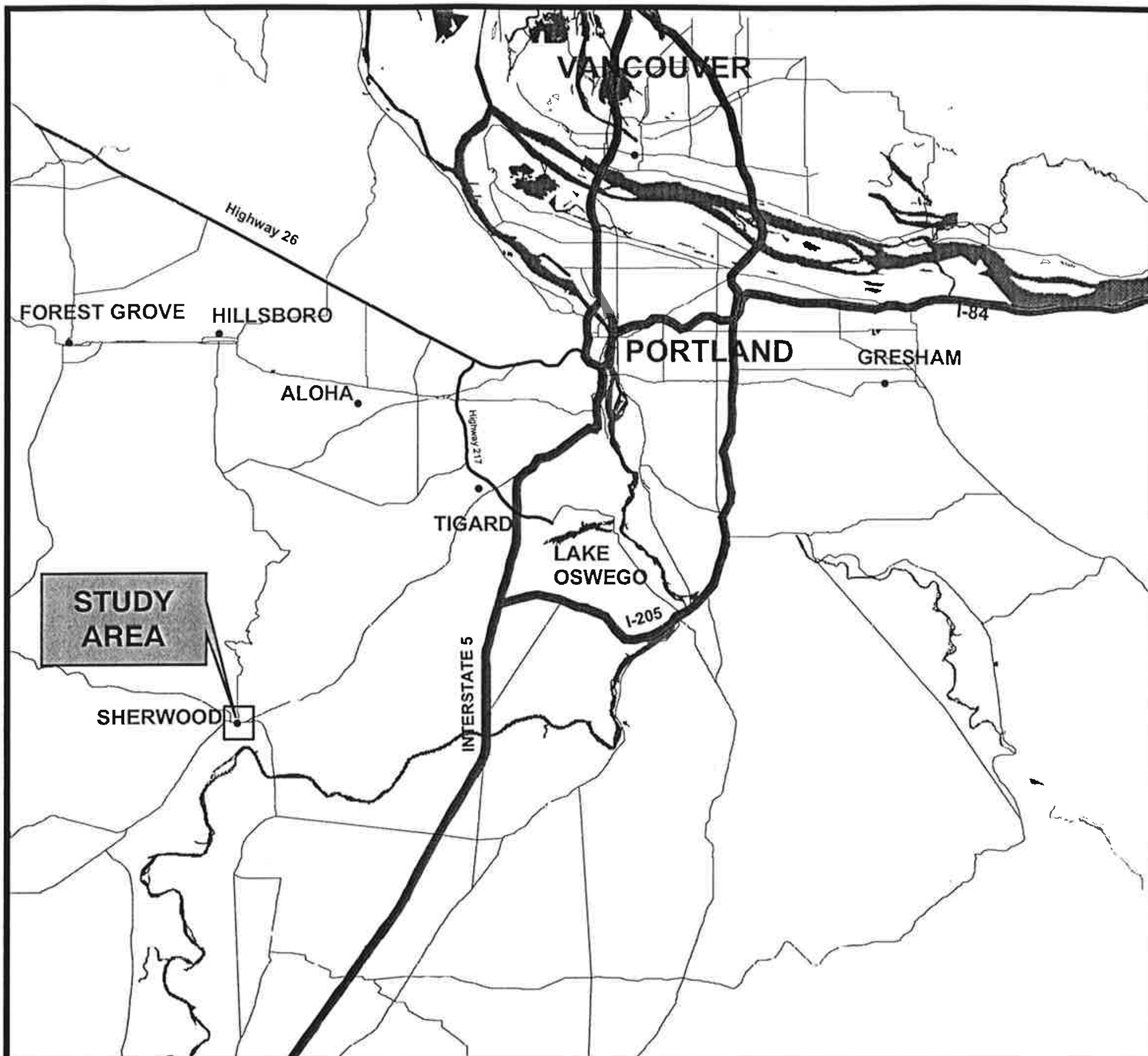
Scenario 4: Lower Pump In Well No. 3 and Deepen Well No. 5

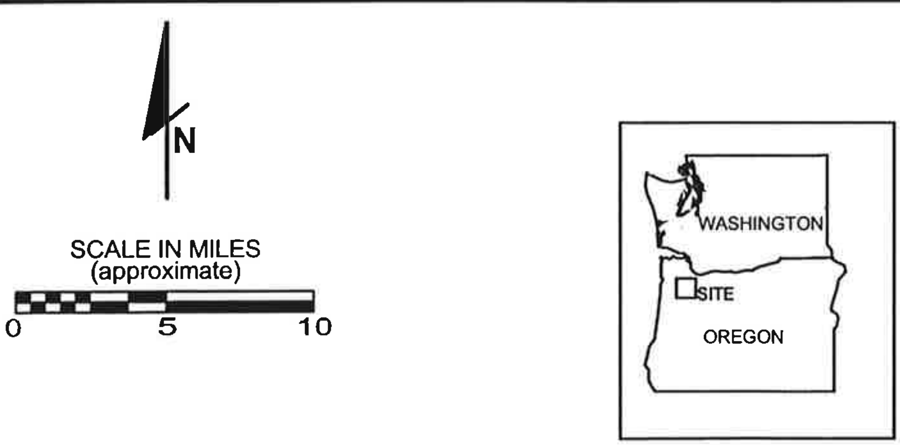
	1998	1999	2000	2001	2002	2003	2004	2005	2006	2007	2008	2009	2010
Well No. 3	125	125	65	65	65	65	65	65	65	0	0	0	0
Well No. 4	45	45	45	45	45	45	45	45	45	45	45	45	45
Well No. 5	100	100	130	130	130	130	130	130	130	130	130	130	130
Well No. 6	130	130	130	130	130	130	130	130	130	130	130	130	130
total	**400**	**400**	**370**	**370**	**370**	**370**	**370**	**370**	**370**	**305**	**305**	**305**	**305**

Scenario 5: Purchase and Produce From Spada Well In Addition to Lowering Pump In Well No. 3 and Deepening Well No. 5

	1998	1999	2000	2001	2002	2003	2004	2005	2006	2007	2008	2009	2010
Well No. 3	125	125	65	65	65	65	65	65	65	0	0	0	0
Well No. 4	45	45	45	45	45	45	45	45	45	45	45	45	45
Well No. 5	100	100	130	130	130	130	130	130	130	130	130	130	130
Well No. 6	130	130	130	130	130	130	130	130	130	130	130	130	130
Spada			45	45	45	45	45	45	45	45	45	45	45
total	**400**	**400**	**415**	**415**	**415**	**415**	**415**	**415**	**415**	**350**	**350**	**350**	**350**

Note: The predictions presented above are were derived using only readily available and, in some cases, incomplete data.
Critical assumptions have been made that can influence the predictions significantly.
Consequently, the above predictions should be considered to represent rough estimates.





HYDROGEOLOGIC EVALUATION
MUNICIPAL WELL FIELD
SHERWOOD, OREGON

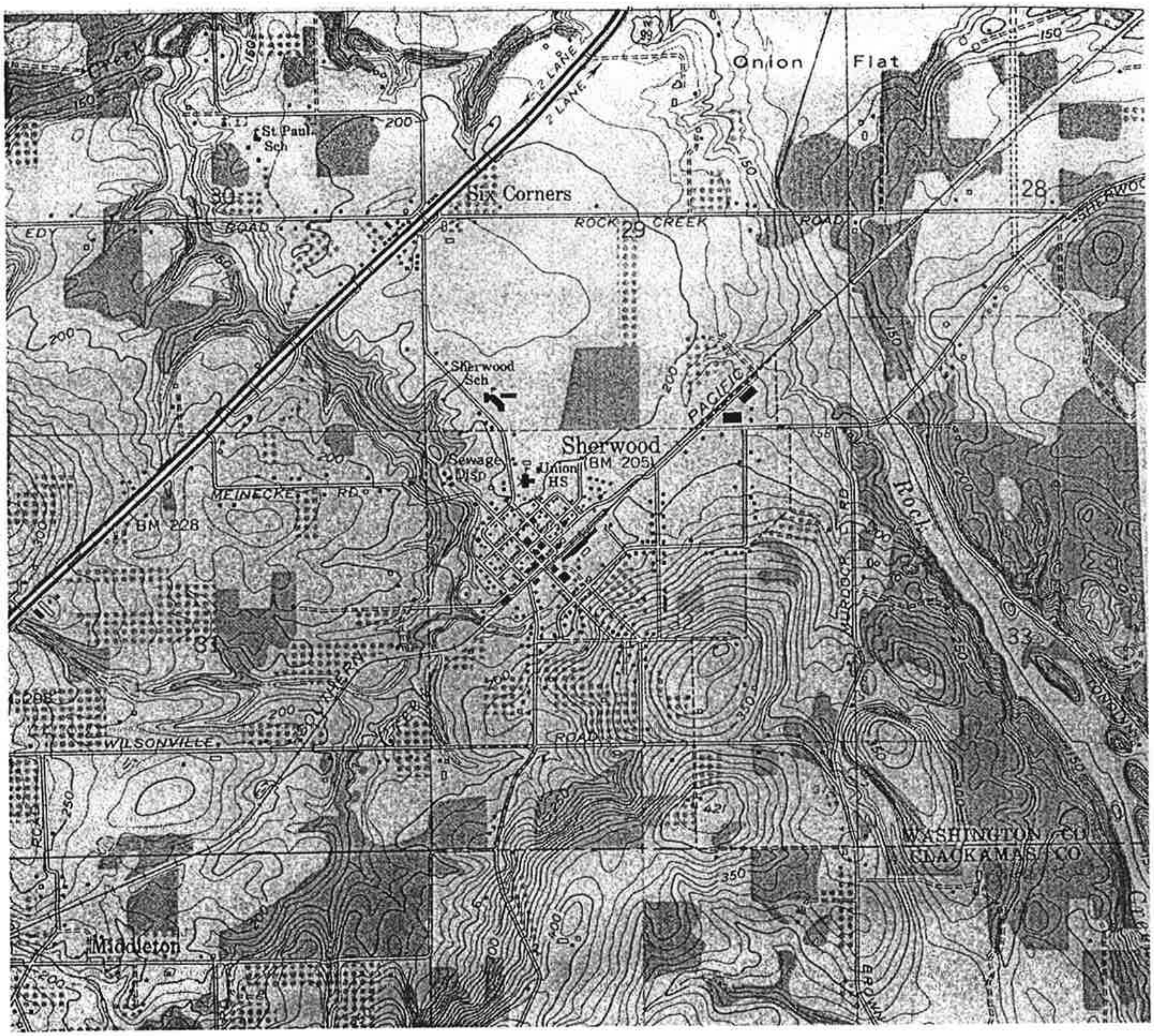
LOCATION MAP

SQUIER ASSOCIATES

FIGURE 1

NOTE: Features shown are for illustrative purposes only and are approximate

Drawn by: KRK	Checked by:	Graphics Check:		File: O:\ENV\GRAPHICS\LOCATION.DSF	Date: 6/3/99	Job. No. 99416



N

SCALE IN FEET

0 1000 2000

HYDROGEOLOGIC EVALUATION
MANICIPAL WELL FIELD
SHERWOOD, OREGON

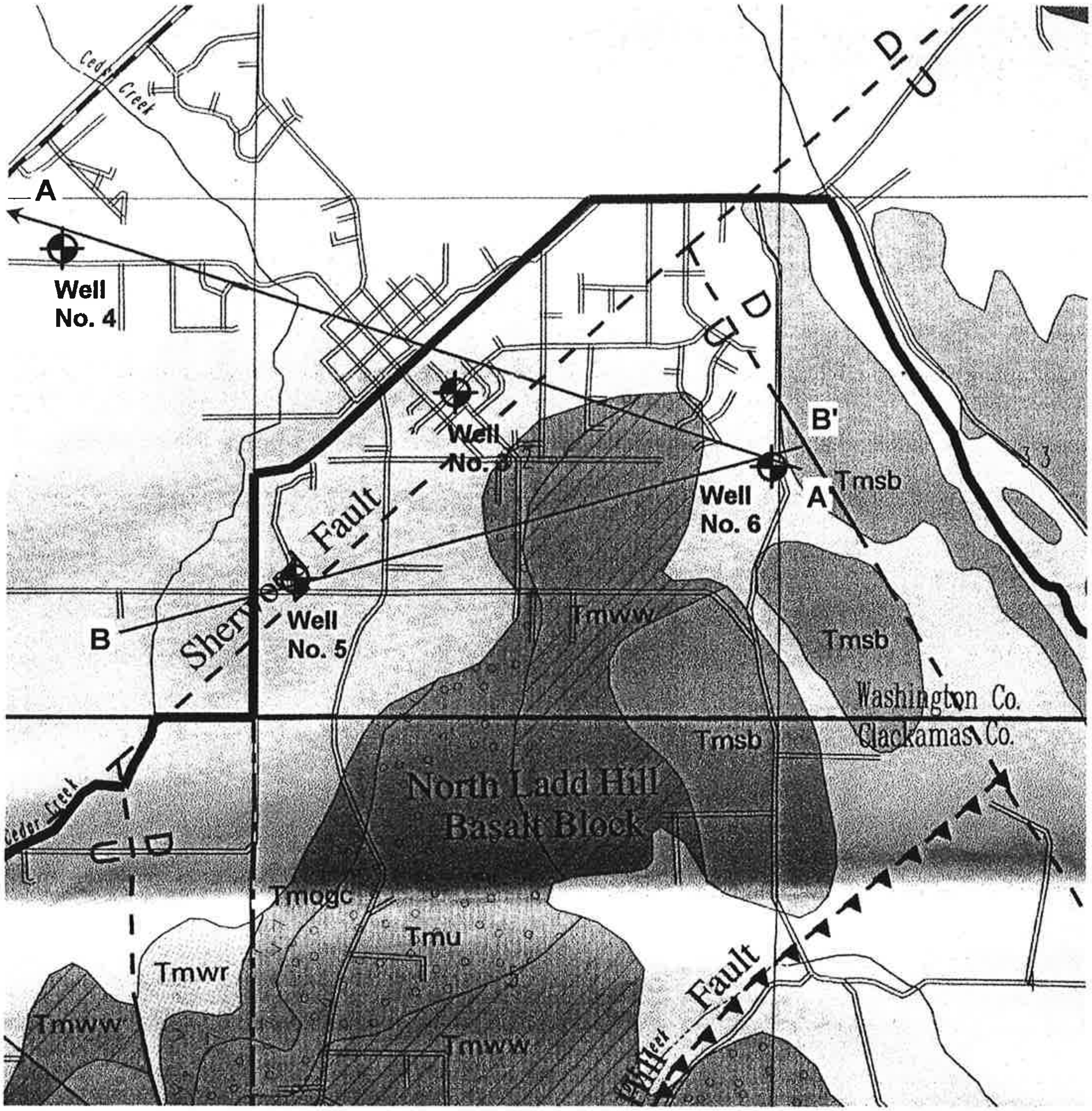
VICINITY MAP

SQUIER ASSOCIATES

FIGURE 2

NOTE: Features shown are for illustrative purposes only and are approximate

Drawn by: WJC	Checked by:	Graphics Check:		File: VININITY.DSF	Date: 06/04/99	Job. No. 99416.1



Source: Ground Water Report No. 40
State of Oregon
Miller, et. al 1994

LEGEND

Qal = Recent Alluvium

Tmsb = Sentinel Bluffs Basalt

Tmww = Winter Water Basalt

Tmu = Umtanum Basalt

Tmwr = Wapshilla Ridge Basalt

Tmogc = Ortley/Grouse Creek Basalt

= Wells

= Faults

A — A' = Cross - Sections

HYDROGEOLOGY MAP
CITY OF SHERWOOD, OREGON

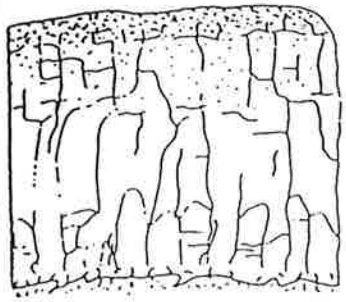
GEOLOGIC MAP

SQUIER ASSOCIATES

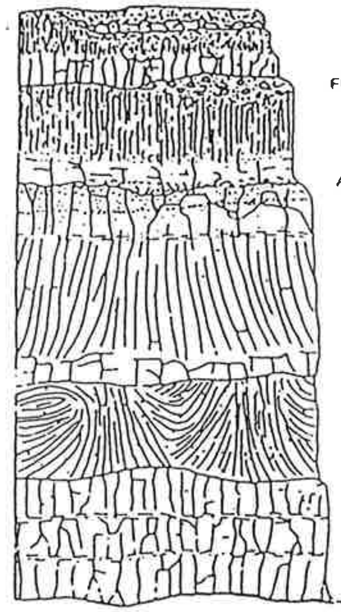
FIGURE 3

NOTE: Features shown are for illustrative purposes only and are approximate

Drawn by: WJC	Checked by:	Graphics Check:		File: \SITE.DSF	Date: 5/27/99	Job. No. 99416



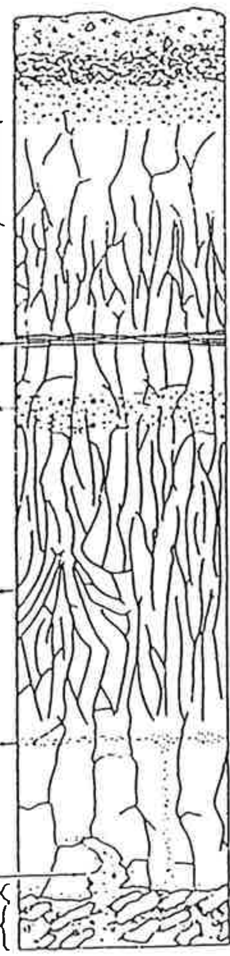
BLOCKY - COLUMNAR FLOW



MULTI-TIERED
ENTABLATURE - COLONNADE FLOW

ENTABLATURE - COLONNADE
FLOW

COMPOSITE OF TYPICAL
INTRAFLOW STRUCTURES



Source: Philip E. Long and Randal W. Cross, Presentation X - Physical Characteristics of Columbia River Basalt Group Flows - The Pasco Basin and Vicinity: in Department of Energy / Nuclear Regulatory Commission, Geologic Workshop and Field Conference on the Stratigraphy of the Columbia River Basalt Group and Ellensburg Formation, Dec. 7-10, 1987.

City of Sherwood
Municipal Well Field Evaluation

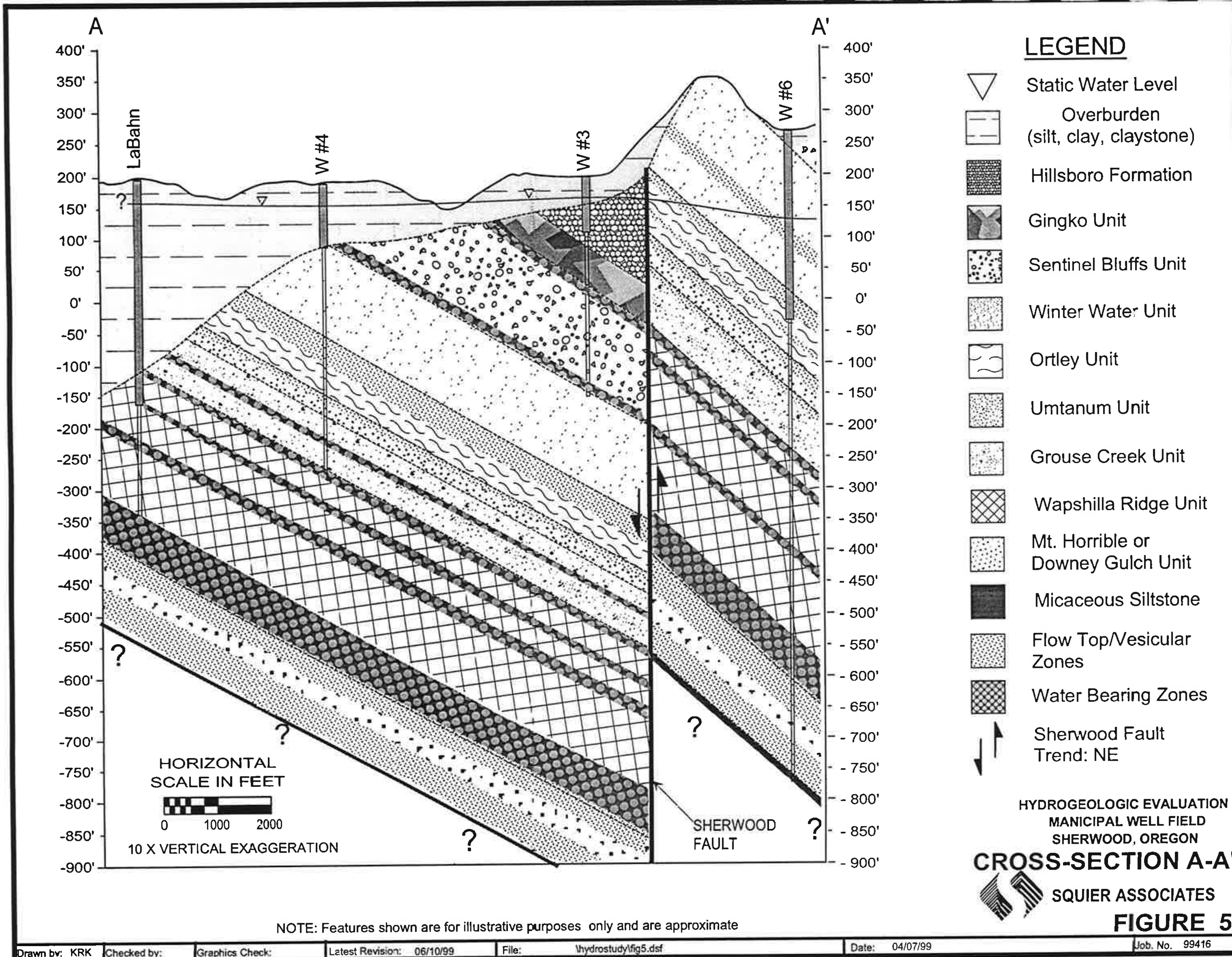
CHARACTERISTIC OF COLUMBIA RIVER BASALT GROUP FLOWS

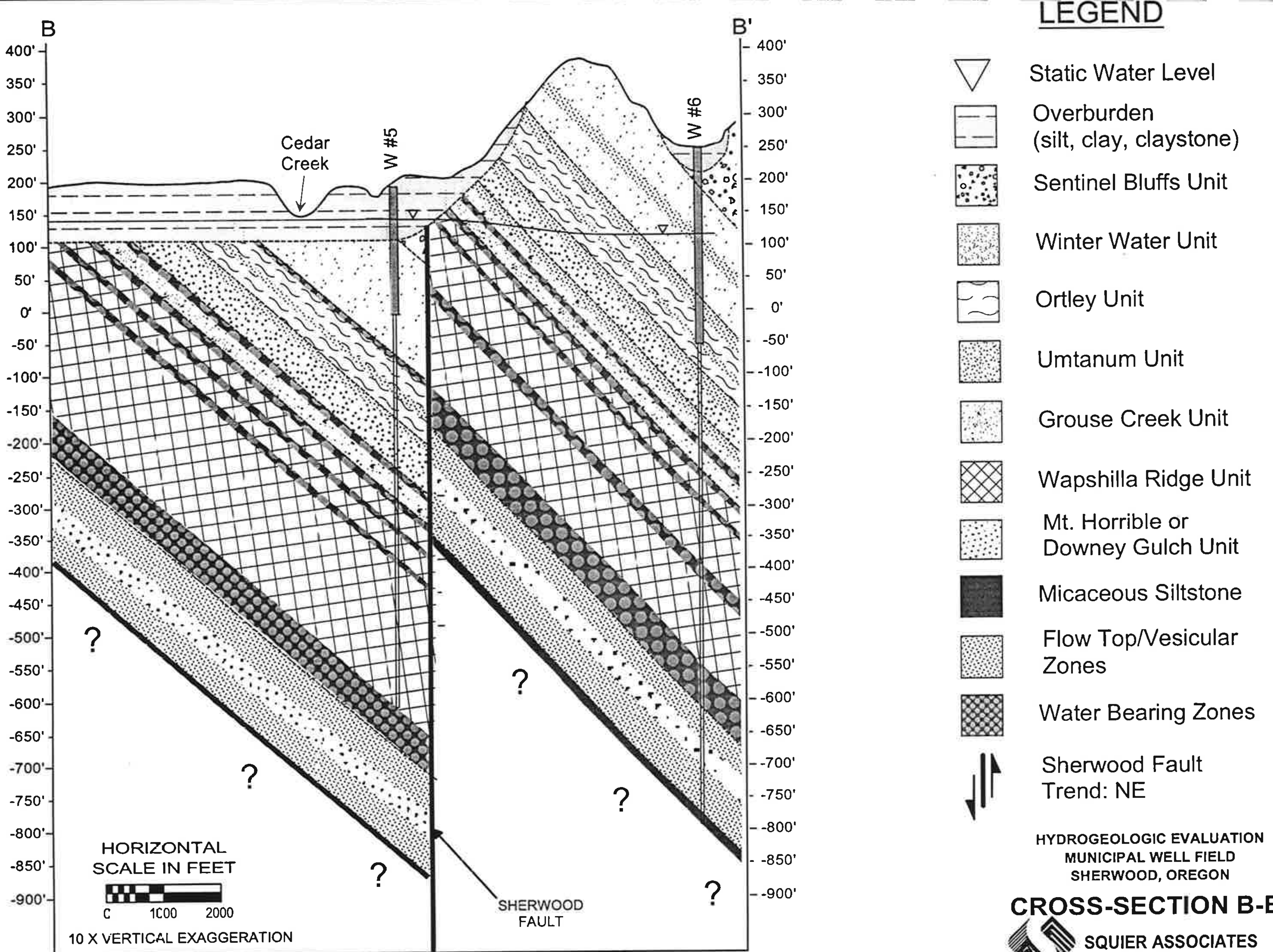
SQUIER ASSOCIATES

NOTE: Features shown are for illustrative purposes only and are approximate

FIGURE 4

Drawn by: KRK	Checked by:	Graphics Check:	Latest Revision:	File: FIGURE4.DSF	Date: 06/11/99	Job. No. 99416.1


A
A'
400'
350'
300'
250'
200'
150'
100'
50'
0'
-50'
-100'
-150'
-200'
-250'
-300'
-350'
-400'
-450'
-500'
-550'
-600'
-650'
-700'
-750'
-800'
-850'
-900'
LaBahn
W #4
W #3
W #6
?
HORIZONTAL
SCALE IN FEET
0 1000 2000
10 X VERTICAL EXAGGERATION
SHERWOOD
FAULT
LEGEND
Static Water Level
Overburden
(silt, clay, claystone)
Hillsboro Formation
Gingko Unit
Sentinel Bluffs Unit
Winter Water Unit
Ortley Unit
Umtanum Unit
Grouse Creek Unit
Wapshilla Ridge Unit
Mt. Horrible or
Downey Gulch Unit
Micaceous Siltstone
Flow Top/Vesicular
Zones
Water Bearing Zones
Sherwood Fault
Trend: NE
HYDROGEOLOGIC EVALUATION
MANICIPAL WELL FIELD
SHERWOOD, OREGON
CROSS-SECTION A-A'
SQUIER ASSOCIATES
FIGURE 5
NOTE: Features shown are for illustrative purposes only and are approximate
Drawn by: KRK
Checked by:
Graphics Check:
Latest Revision: 06/10/99
File: \hydrostudy\fig5.dsf
Date: 04/07/99
Job. No. 99416



HYDROGEOLOGIC EVALUATION
MUNICIPAL WELL FIELD
SHERWOOD, OREGON

CROSS-SECTION B-B'

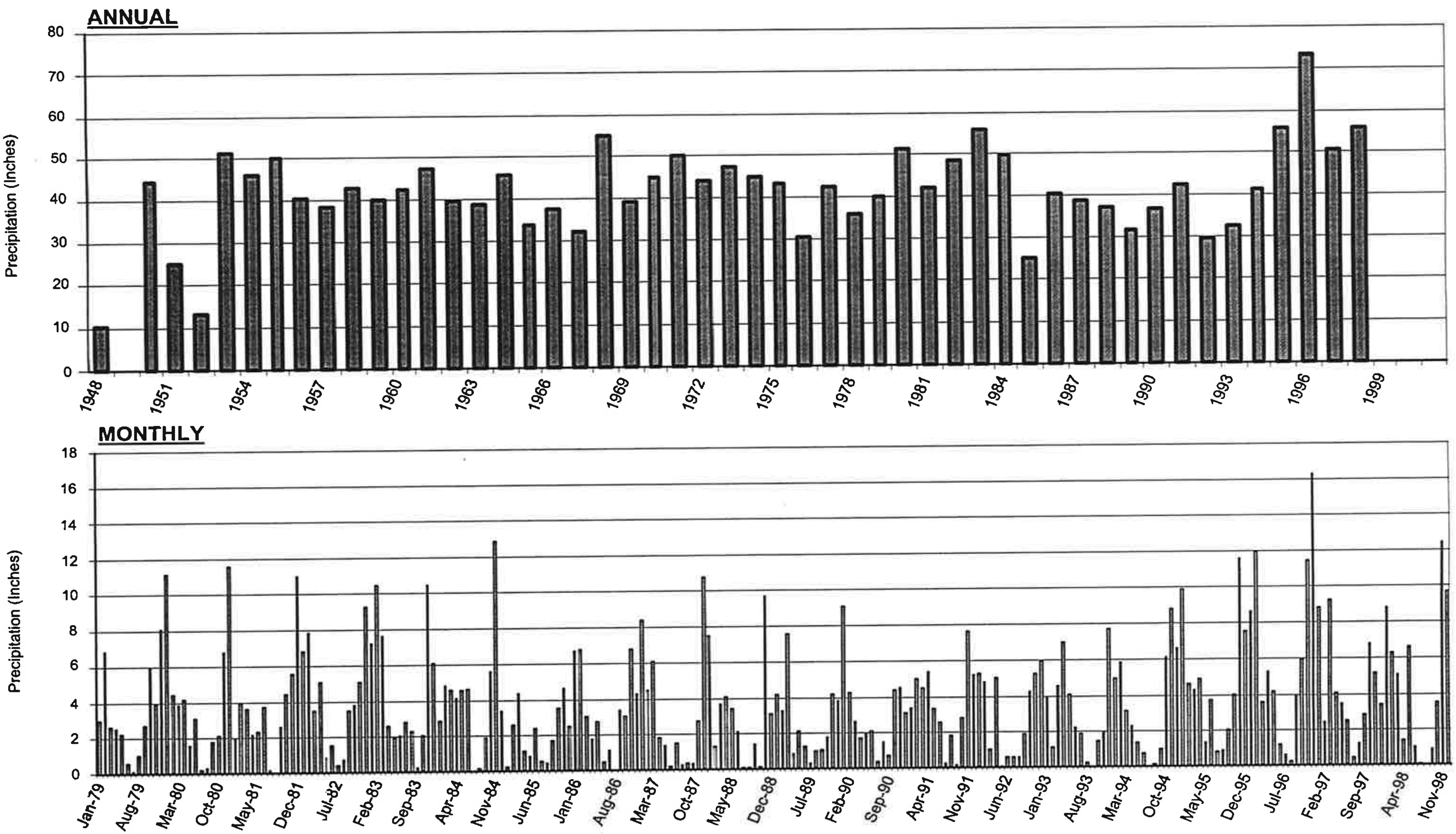
SQUIER ASSOCIATES

FIGURE 6

NOTE: Features shown are for illustrative purposes only and are approximate

Drawn by: KRK	Checked by:	Graphics Check:	Latest Revision:	File: \HYDROSTUDY\FIG6.DSF	Date: 04/07/99	Job. No. 99416

REX 1S
Weather Station Precipition
1948-Present

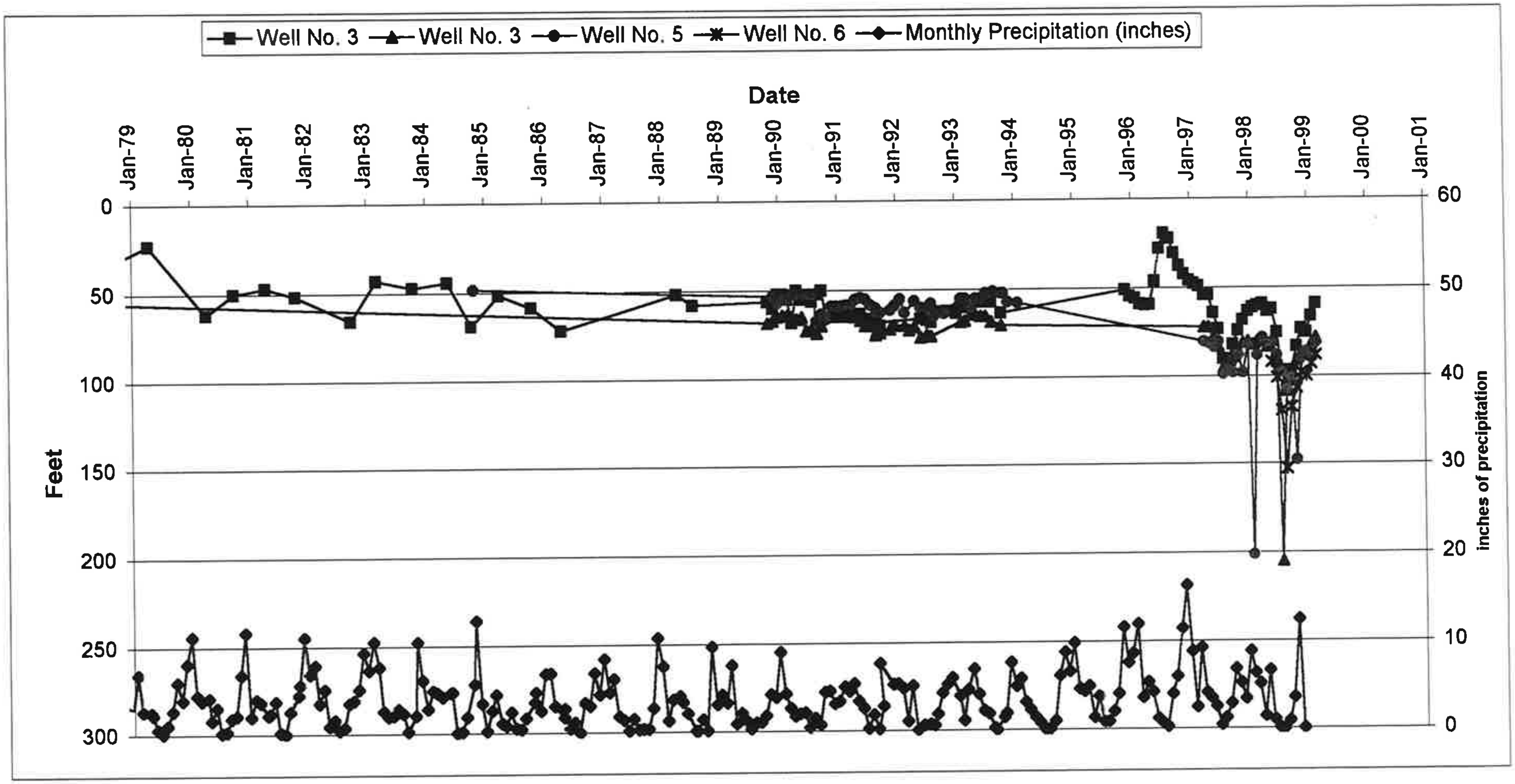


City of Sherwood

SQUIER ASSOCIATES
FIGURE 7

6/2/99 1:00 PM
99416\HYDRO STUDY\FIGURE7.DSF

Precipitation and Water Level Summary

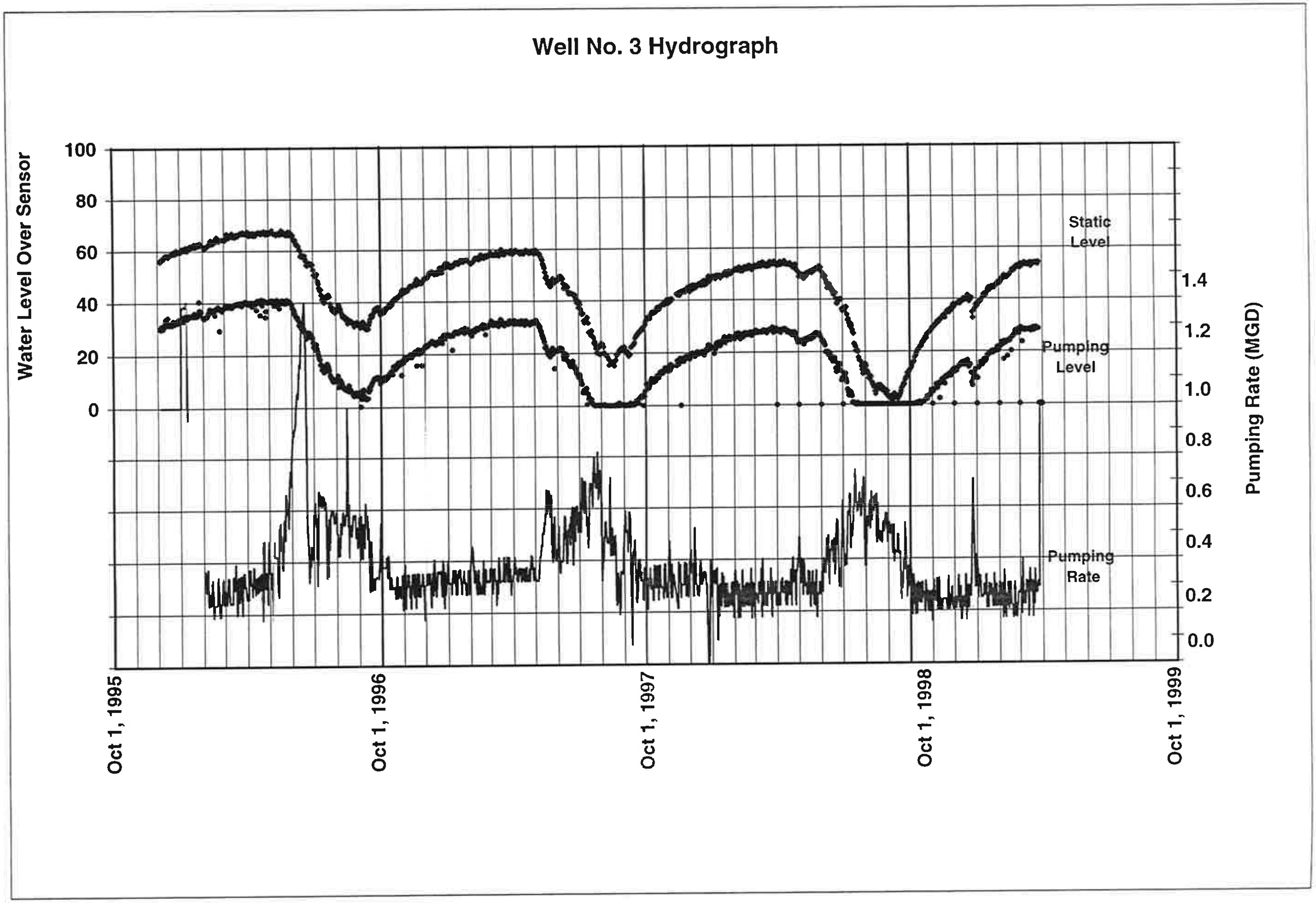


City of Sherwood

SQUIER ASSOCIATES

FIGURE 8

8/20/99 2:04 PM
hydrgraph(2) summary Chart 3



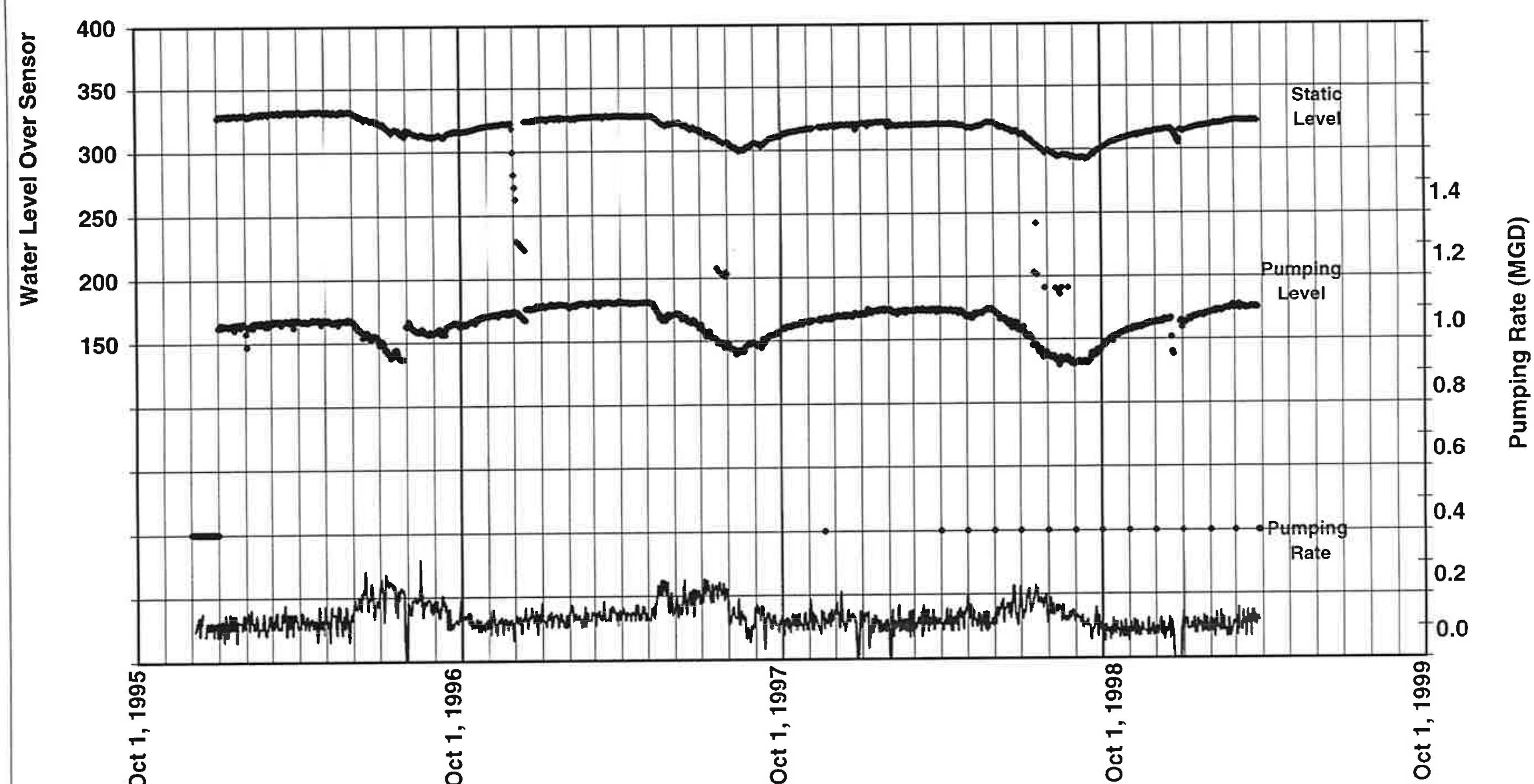
SQUIER ASSOCIATES

FIGURE 9

City of Sherwood

6/11/99 8:23 AM
trends:/ SherwoodWell2 / Graphics Chart 1

Well No. 4 Hydrograph

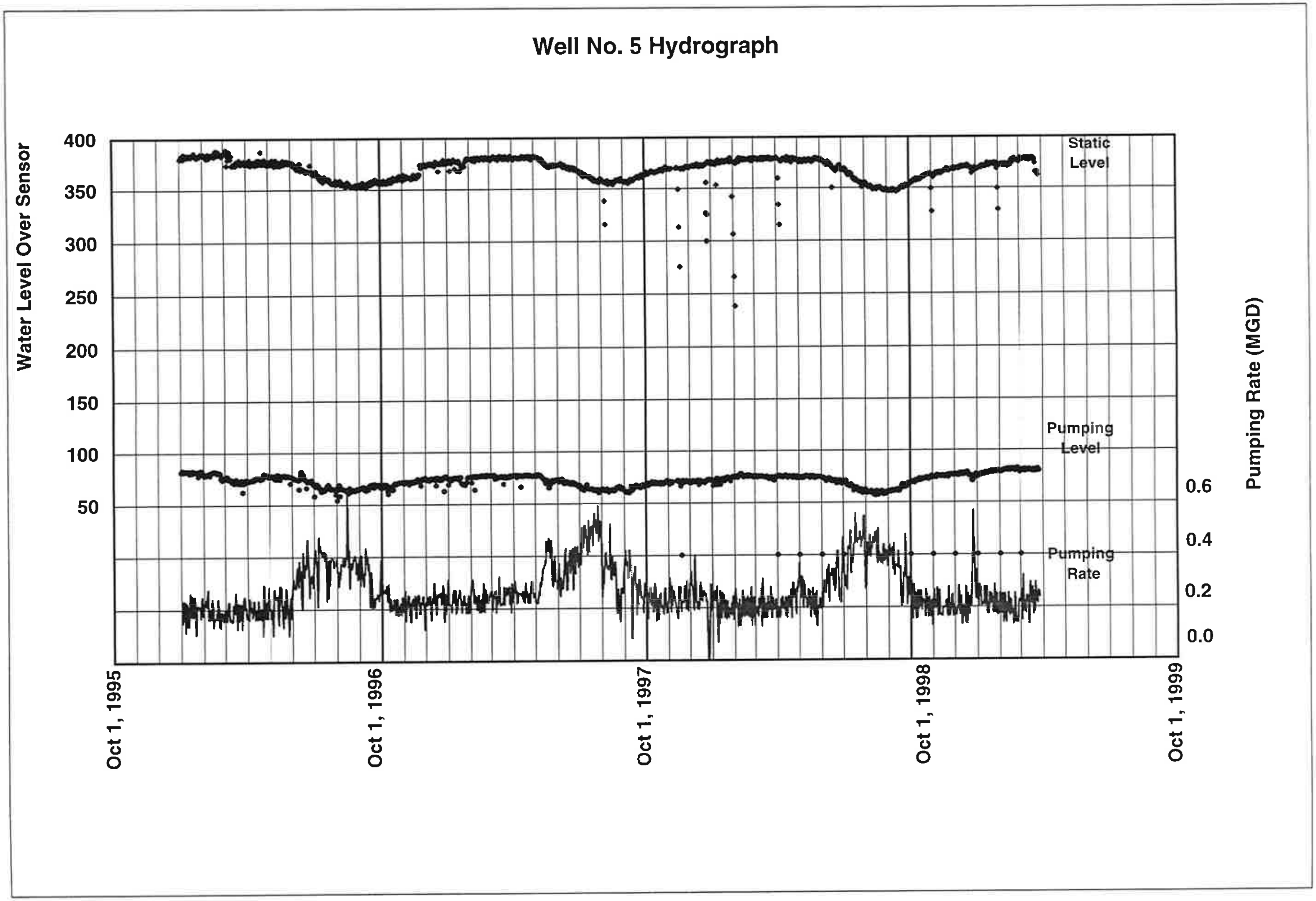


6/11/99 8:24 AM
trends:/SherwoodWell2 / Graphics Chart 2

City of Sherwood

SQUIER ASSOCIATES

FIGURE 10

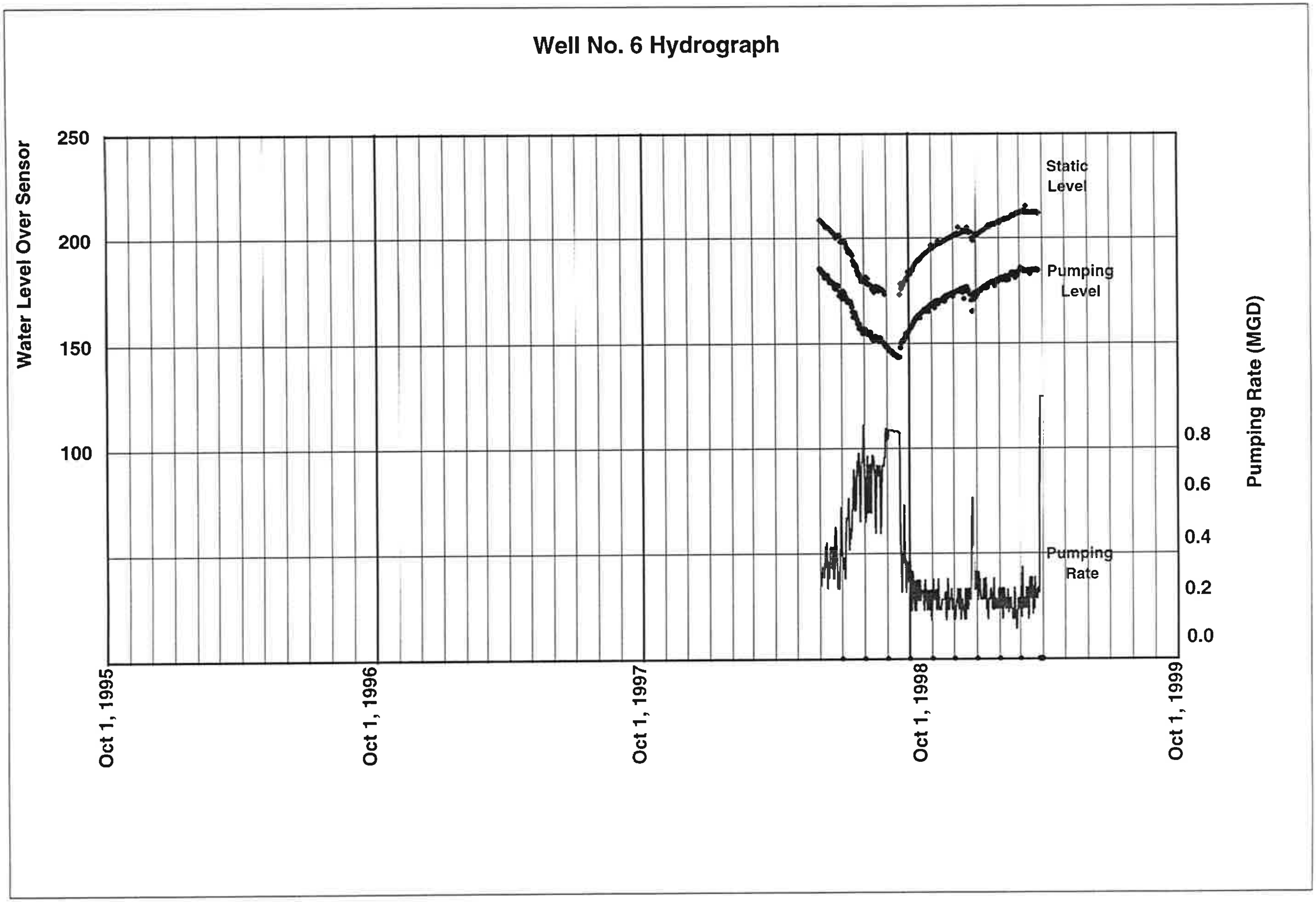


6/11/99 8:27 AM
trends:/SherwoodWell2 / Graphics Chart 3

City of Sherwood

SQUIER ASSOCIATES

FIGURE 11

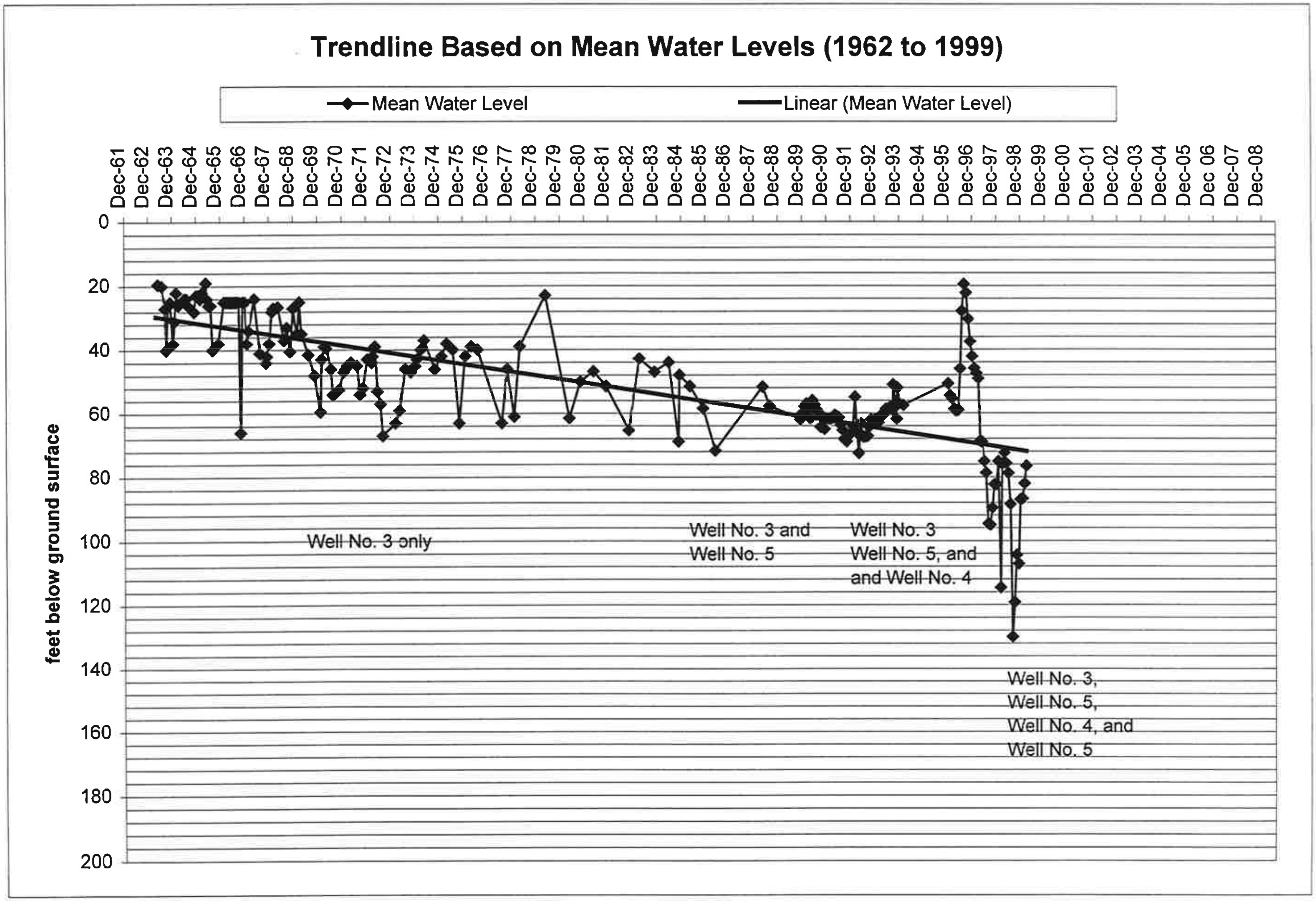


SQUIER ASSOCIATES

FIGURE 12

City of Sherwood

6/11/99 8:22 AM
trends:/SherwoodWell2 / Graphics Chart 4

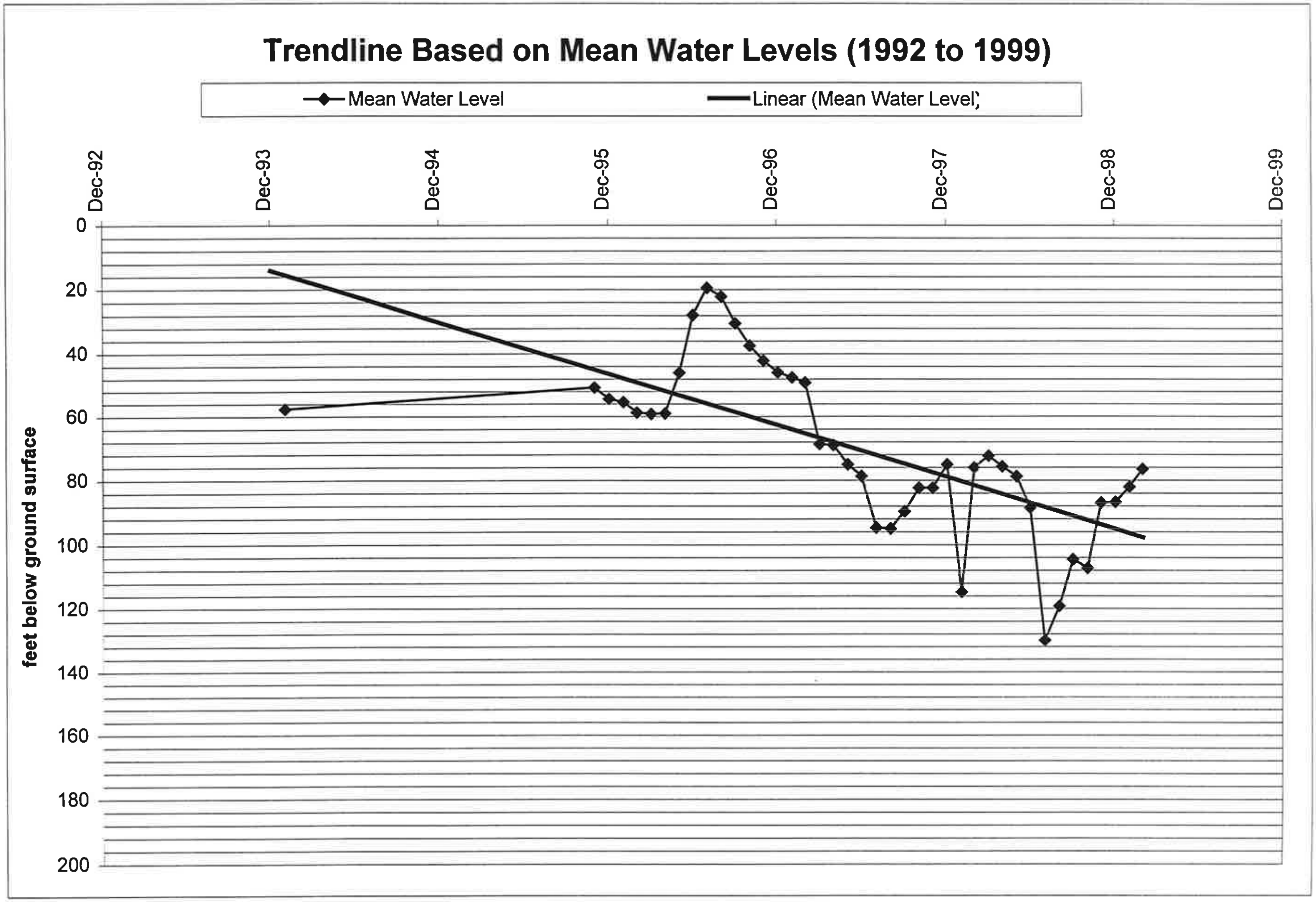


City of Sherwood

SQUIER ASSOCIATES

FIGURE 13

8/20/99 8:44 AM
hydrgraph(2) / summary Chart 4



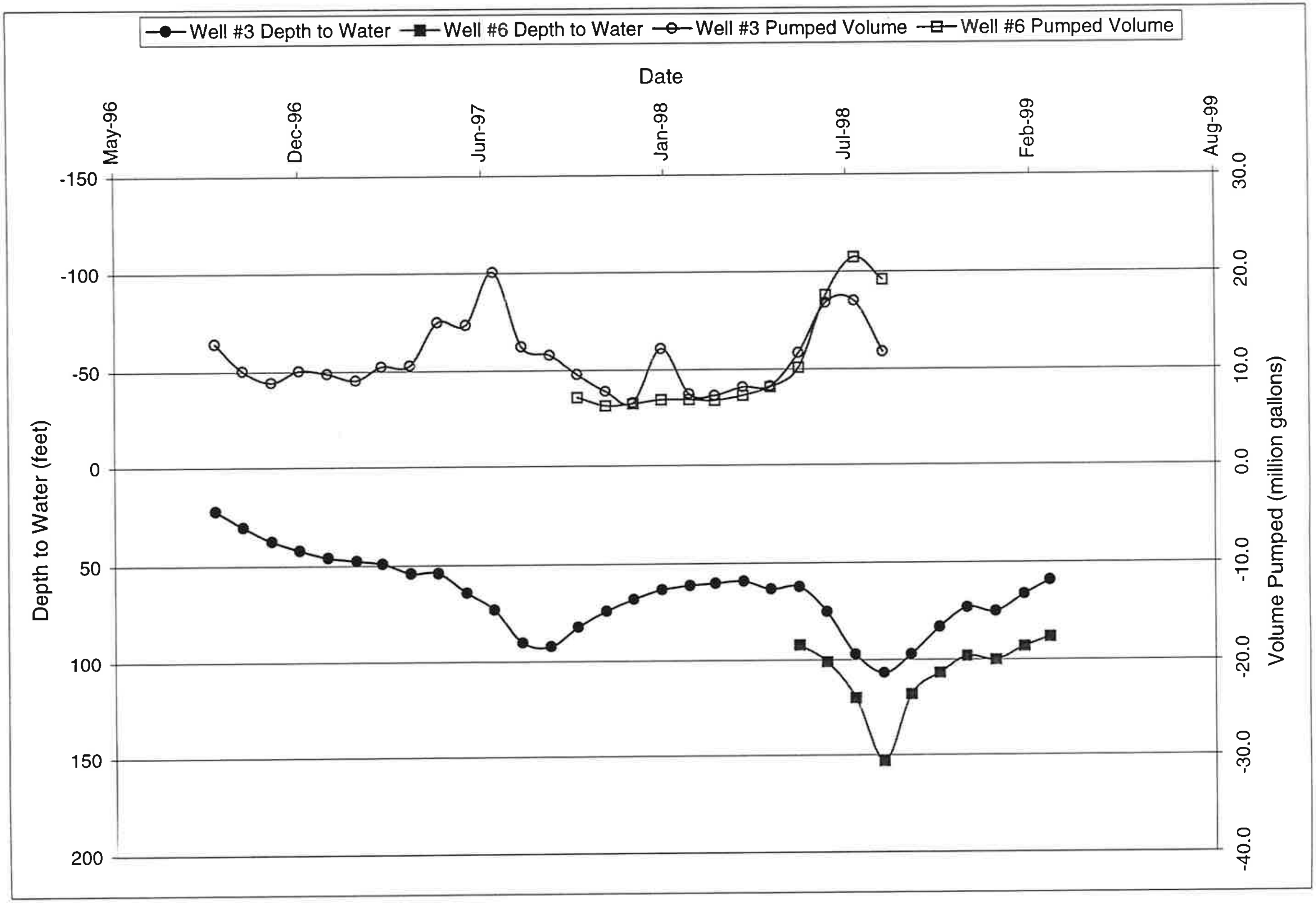
8/20/99 8:45 AM
hydrgraph(2) / recent Chart 4

City of Sherwood

SQUIER ASSOCIATES

FIGURE 14

Comparison of Well No.3 and Well No.6

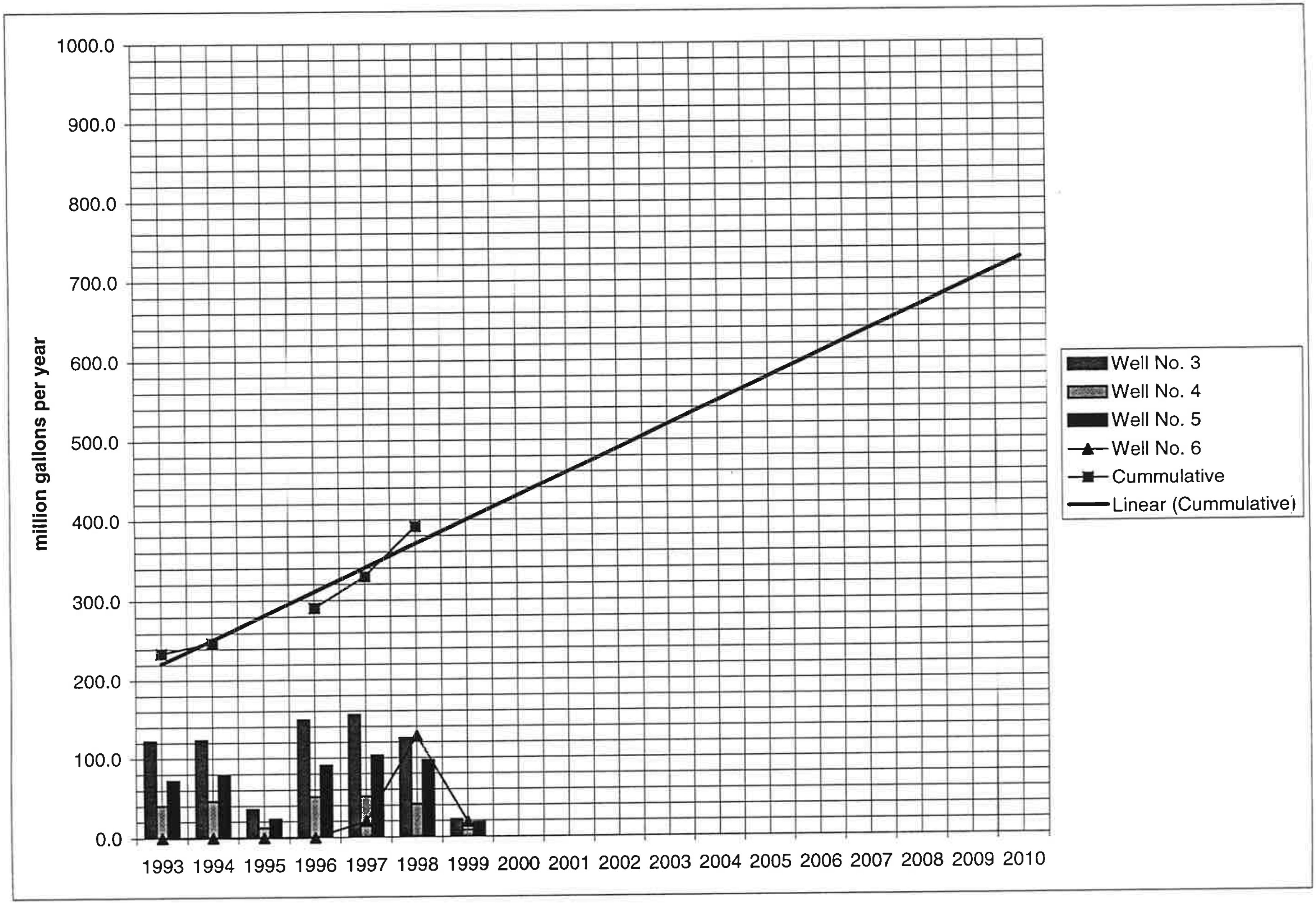


SQUIER ASSOCIATES
FIGURE 15

6/11/99 8:34 AM
hydragraph(2) / #3&#6 Chart 3

City of Sherwood

Estimated Ground Water Production Requirements



SQUIER ASSOCIATES

FIGURE 16

6/11/99 2:52 PM
pumphist.xls Sheet1 Char: 1

City of Sherwood

Appendix A

OBSERVATION WELL

Well Record

STATE ENGINEER
Salem, Oregon

Wash 1823

GR- 1708

STATE WELL NO. 2/1W-32F(3)
COUNTY Washington
APPLICATION NO. GR- 1162

OWNER: H.G. Albert, Sherwood city recorder
MAILING ADDRESS:

LOCATION OF WELL: Owner's No. #3
CITY AND STATE: Sherwood, Oregon

SE ¼ NW ¼ Sec. 32 T. 2 S., R. 1 W., W.M.

Bearing and distance from section or subdivision corner 750' N. & 600' W. from Center of Sec. 32.

Section 32

Altitude at well 190'

TYPE OF WELL: Drilled Date Constructed 1946

Depth drilled 339' Depth cased 122'

CASING RECORD:
16" from 0 to 36'
12" from 35' to 122'

FINISH:
BASALT FROM 137' TO 339'

AQUIFERS:

WATER LEVEL:
26'

PUMPING EQUIPMENT: Type Rotary (TURBINE) H.P. 60
Capacity 510 G.P.M.

WELL TESTS:
Drawdown 42 ft. after ____ hours 510 G.P.M.
Drawdown ____ ft. after ____ hours ____ G.P.M.

USE OF WATER Mun., Man. & Ind. Temp. ____ °F. ____, 19____
SOURCE OF INFORMATION GR Record
DRILLER or DIGGER
ADDITIONAL DATA:
Log X Water Level Measurements ____ Chemical Analysis ____ Aquifer Test ____

REMARKS:

STATE ENGINEER
Salem, Oregon

State Well No. 2/1W-32F (3)
County Washington
Application No. GR- 1162

Well Log

Owner: H. G. Albert, Sherwood city recorder Owner's No. #3

Driller: Date Drilled 1946

CHARACTER OF MATERIAL	(Feet below land surface) From	To	Thickness (feet)
Clay	0	20	20
Quick sand & blue clay	20	38	18
Sand rock	38	137	99
Lava rock	137	175	38
Rock	175	182	7
Cemented gravel	182	206	24
Lava rock	206	254	48
Cemented gravel	254	269	15
Lava rock	269	273	4
Cemented gravel	273	296	23
Broken rock	296	339	43

well #3

2/1W-32 F (3)
Washington

GR - 1162
Cert. GR - 1708

LOG OF WELL

DRILLED FOR THE CITY OF SHERWOOD, SHERWOOD, OREGON.

BY

A. M. JANNSEN DRILLING CO., 319 PITTOCK BLOCK,

PORTLAND 5, ORE.

Work started 6-10-46 - Completed 7-17-46

Formation:

From	0'	to	20'	Clay
	20'		38'	Quick sand and blue clay
	38'		137'	Sand rock
	137'		175'	Lava Rock
	175'		182'	Rock
	182'		206'	Cemented gravel
	206'		254'	Lava Rock
	254'		269'	Cemented gravel
	269'		273'	Lava Rock
	273'		296'	Cemented gravel
	296'		339'	Broken Rock

Pipe:	38'	of	16"
	77'	"	12"

Static Water Level 26'

340 GPM	DD to 33'
460 "	DD to 33'
510 "	DD to 42'

Appendix B

NOTICE TO WATER WELL CONTRACTOR
The original and first copy of this report are to be filed with the
STATE ENGINEER, SALEM, OREGON 97310 within 30 days from the date of well completion.

RECEIVED
MAY 7 1969
STATE ENGINEER
SALEM, OREGON

WASH 3447 Sec 2/1w-31B
6"-400 Samewell

WATER WELL REPORT
STATE OF OREGON
(Please type or print)
(Do not write above this line)

State Well No. 2/1w-31aba
State Permit No.
G-4777

(1) OWNER:
Name City of Sherwood Ore.
Address " "

(2) TYPE OF WORK (check):
New Well ☑ Deepening ☐ Reconditioning ☐ Abandon ☐
If abandonment, describe material and procedure in Item 12.

(3) TYPE OF WELL:
Rotary ☐ Driven ☐
Cable ☑ Jetted ☐
Dug ☐ Bored ☐

(4) PROPOSED USE (check):
Domestic ☐ Industrial ☐ Municipal ☑
Irrigation ☐ Test Well ☐ Other ☐

CASING INSTALLED: Threaded ☐ Welded ☑
14" Diam. from +1 ft. to 99 ft. Gage .330
" Diam. from ft. to ft. Gage
" Diam. from ft. to ft. Gage

PERFORATIONS: Perforated? ☐ Yes ☑ No.
Type of perforator used
Size of perforations in. by in.
perforations from ft. to ft.
perforations from ft. to ft.
perforations from ft. to ft.
perforations from ft. to ft.
perforations from ft. to ft.

(7) SCREENS: Well screen installed? ☐ Yes ☑ No
Manufacturer's Name
Type Model No.
Diam. Slot size Set from ft. to ft.
Diam. Slot size Set from ft. to ft.

(8) WATER LEVEL: Completed well.
Static level 44 ft. below land surface Date 4-28-69
sian pressure lbs. per square inch Date

(9) WELL TESTS: Drawdown is amount water level is lowered below static level
Was a pump test made? ☑ Yes ☐ No If yes, by whom? Pump Pipe & Power
Yield: 414 gal./min. with 249 ft. drawdown after 4 hrs.
376 " 156 " 2 "
" 298 " 131 " 2 "
Bailer test gal./min. with ft. drawdown after hrs.
Artesian flow g.p.m. Date
Temperature of water Was a chemical analysis made? ☐ Yes ☑ No

(10) CONSTRUCTION:
Well seal—Material used Cement
Depth of seal 99 ft.
Diameter of well bore to bottom of seal 20 in.
Were any loose strata cemented off? ☐ Yes ☑ No Depth
Was a drive shoe used? ☐ Yes ☑ No
Did any strata contain unusable water? ☐ Yes ☑ No
Type of water? depth of strata
Method of sealing strata off
Was well gravel packed? ☐ Yes ☑ No Size of gravel:
Gravel placed from ft. to ft.

(11) LOCATION OF WELL:
County Washington Driller's well number
¼ ¼ Section 5 T. 3 S R. 1 W W.M.
Bearing and distance from section or subdivision corner

(12) WELL LOG: Diameter of well below casing 14
Depth drilled 458 ft. Depth of completed well 458 ft.

Formation: Describe color, texture, grain size and structure of materials; and show thickness and nature of each stratum and aquifer penetrated, with at least one entry for each change of formation. Report each change in position of Static Water Level as drilling proceeds. Note drilling rates.

MATERIAL	From	To	SWL
Brown — Clay with Fine Sand	0	45	
Blue — Clay with Black Sand	45	78	
Brown — Decomposed Basalt	78	93	
Brown & Black — Basalt Rock	93	98	
Black & Red — " "	98	116	
" — " "	116	157	
Brown — "	157	167	
Black — Basalt Rock	167	240	
Red — Layers Rock Hard & Soft	240	283	
Black — Basalt Rock	283	300	
Brown — "	300	328	
Gray — Hard "	328	392	
Black — Basalt " (Seamy)	392	415	
Black — Basalt Rock	415	458	

Work started 1-6 — 1969 Completed 4-28 1969
Date well drilling machine moved off of well 4-28 — 1969

Drilling Machine Operator's Certification:
This well was constructed under my direct supervision. Materials used and information reported above are true to my best knowledge and belief.
[Signed] Marvin West (Drilling Machine Operator) Date 4-30-1969
Drilling Machine Operator's License No. 188

Water Well Contractor's Certification:
This well was drilled under my jurisdiction and this report is true to the best of my knowledge and belief.
NAME Miller-Robinson & West (Person, firm or corporation) (Type or print)
Address 5545 Joseph St. S.E. Salem
[Signed] Harlan R. Miller (Water Well Contractor)
Contractor's License No. 37 Date 4-30-1969

(USE ADDITIONAL SHEETS IF NECESSARY)

Appendix C

RECEIVED
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WATER RESOURCES DEPT.
SALEM, OREGON

717 Wash

2S/1W-32cb

STATE OF OREGON
WATER WELL REPORT
(as required by ORS 537.765)

PLEASE TYPE or PRINT IN INK (for official use only)

(1) OWNER:
Name City of Sherwood Well #5
Address 90 NW Park
City Sherwood Or. 97140 State

(2) TYPE OF WORK (check):
New Well ☒ Deepening ☐ Reconditioning ☐ Abandon ☐
If abandonment, describe material and procedure in Item 12.

(3) TYPE OF WELL:
Rotary Air ☒ Driven ☐
Rotary Mud ☒ Dug ☐
[illegible] ☐ Bored ☐

(4) PROPOSED USE (check):
Domestic ☐ Industrial ☐ Municipal ☒
Irrigation ☐ Thermal: Withdrawal ☐ Reinjection ☐
Other: Piezometric ☐ Grounding ☐ Test ☐

(5) CASING INSTALLED: Steel ☒ Plastic ☐ Threaded ☐ Welded ☐
16 " Diam. from +1.5 ft. to 200 ft. Gauge .375
" Diam. from ft. to ft. Gauge

LINER INSTALLED: Steel ☐ Plastic ☐ Threaded ☐ Welded ☐
NONE
" Diam. from ft. to ft. Gauge

(6) PERFORATIONS: Perforated? ☐ Yes ☒ No
Size of perforations in. by in.
perforations from ft. to ft.
perforations from ft. to ft.
perforations from ft. to ft.

(7) SCREENS: Well screen installed? ☐ Yes ☒ No
Manufacturer's Name
Type Model No.
Diam. Slot Size Set from ft. to ft.
Diam. Slot Size Set from ft. to ft.

(8) WELL TESTS: Drawdown is amount water level is lowered below static level
Was a pump test made? ☒ Yes ☐ No If yes, by whom? A&H Pump
[illegible]: 700 gal./min. with 309 ft. drawdown after 24 hrs.
Air test gal./min. with drill stem at ft. hrs.
Bailer test gal./min. with ft. drawdown after hrs.
Artesian flow g.p.m.
[illegible]perature of water Depth artesian flow encountered ft.

(9) CONSTRUCTION: Special standards: Yes ☐ No ☒
Well seal—Material used Cement Grout
Well sealed from land surface to 200 ft.
Diameter of well bore to bottom of seal 20 in.
Diameter of well bore below seal 15-460 in. 8"-800
Amount of sealing material 140 sacks ☒ pounds ☐
How was cement grout placed? With 2inch pipes from bottom

Was pump installed? no Type HP Depth ft.
Was a drive shoe used? ☐ Yes ☒ No Plugs Size: location ft.
Did any strata contain unusable water? ☐ Yes ☒ No
Type of Water? depth of strata
Method of sealing strata off
Was well gravel packed? ☐ Yes ☒ No Size of gravel:
Gravel placed from ft. to ft.

(10) LOCATION OF WELL by legal description:
County Washington NW ¼ SW ¼ of Section 32 of
Township 2s (Township is North or South) Range 1w (Range is East or West) WM.
Tax Lot Lot Block Subdivision
MAILING ADDRESS OF WELL (or nearest address)
Same as owner

(11) WATER LEVEL of COMPLETED WELL:
Depth at which water was first found 252 ft.
Static level 48 ft. below land surface. Date 10-25-84
Artesian pressure lbs. per square inch. Date

(12) WELL LOG: Diameter of well below casing 15"-460 8"-800
Depth drilled 800 ft. Depth of completed well 800 ft.
Formation: Describe color, texture, grain size and structure of materials; and show thickness and nature of each stratum and aquifer penetrated, with at least one entry for each change of formation. Report each change in position of Static Water Level and indicate principal water-bearing strata.

MATERIAL	From	To	SWL
Top soil	0	3	
Clay hard	3	15	
Clay grey sandy w/rock	15	47	
Clay grey sticky w/green claystone streaks	47	83	
Basalt green grey brown porous-broken	83	189	
Basalt grey hard	189	252	
Basalt brn. wthrd.	252	266	W.B.
Basalt grey hrd	266	283	
Basalt grey brn. porous	283	288	W.B.
Basalt grey hrd.	288	317	
Basalt brn. porous-frctrd	317	324	
Basalt grey w/brn frctrd	324	331	
Basalt grey hrd.	331	355	
Basalt wthrd.brn. grey yellow red	355	378	
Basalt grey brn. med. hrd	378	405	
Basalt grey brn. broken	405	407	
Basalt grey hrd.	407	424	

Date work started 9-14-84 /completed 10-26-84
Date well drilling machine moved off of well 10-18-84 19

(unbonded) Water Well Constructor Certification (if applicable):
This well was constructed under my direct supervision. Materials used and information reported above are true to my best knowledge and belief.
[Signed] Steven N. Stadel Date 11-5, 19 84

(bonded) Water Well Constructor Certification:
Bond (number) Issued by: Union Indemnity (Surety Company Name)
On behalf of STACO WELL SERVICES INC. (type or print name of Water Well Constructor)

This well was drilled under my jurisdiction and this report is true to the best of my knowledge and belief.
(Signed) [signature] (Water Well Constructor)
(Dated) 11-5-84

NOTICE TO WATER WELL CONSTRUCTOR
The original and first copy of this report are to be filed with the

WATER RESOURCES DEPARTMENT, SALEM, OREGON 97310
within 30 days from the date of well completion.

SP*46808-690

RECEIVED
NOV 19 1984
WATER RESOURCES DEPT
SALEM, OREGON

2S/1W - 32cb

STATE OF OREGON
WATER WELL REPORT
(as required by ORS 537.765)

WELL LOG CONTINUED #5
PLEASE TYPE or PRINT IN INK

(for official use only)

(1) OWNER:
Name City of Sherwood #5
Address
City State

(2) TYPE OF WORK (check):
New Well ☐ Deepening ☐ Reconditioning ☐ Abandon ☐
If abandonment, describe material and procedure in Item 12.

(3) TYPE OF WELL:
Rotary Air ☐ Driven ☐
Rotary Mud ☐ Dug ☐
☐ Bored ☐

(4) PROPOSED USE (check):
Domestic ☐ Industrial ☐ Municipal ☐
Irrigation ☐ Thermal: Withdrawal ☐ Reinjection ☐
Other: Piezometric ☐ Grounding ☐ Test ☐

(5) CASING INSTALLED:
Steel ☐ Plastic ☐
Threaded ☐ Welded ☐
" Diam. from ft. to ft. Gauge
" Diam. from ft. to ft. Gauge

LINER INSTALLED:
Steel ☐ Plastic ☐
Threaded ☐ Welded ☐
" Diam. from ft. to ft. Gauge

(6) PERFORATIONS:
Perforated? ☐ Yes ☐ No
Size of perforations in. by in.
perforations from ft. to ft.
perforations from ft. to ft.
perforations from ft. to ft.

(7) SCREENS:
Well screen installed? ☐ Yes ☐ No
Manufacturer's Name
Type Model No.
Diam. Slot Size Set from ft. to ft.
Diam. Slot Size Set from ft. to ft.

(8) WELL TESTS:
Drawdown is amount water level is lowered below static level
Was a pump test made? ☐ Yes ☐ No If yes, by whom?
l: gal./min. with ft. drawdown after hrs.
Air test gal./min. with drill stem at ft. hrs.
Bailer test gal./min. with ft. drawdown after hrs.
Artesian flow g.p.m.
perature of water Depth artesian flow encountered ft.

(9) CONSTRUCTION:
Special standards: Yes ☐ No ☐
Well seal—Material used
Well sealed from land surface to ft.
Diameter of well bore to bottom of seal in.
Diameter of well bore below seal in.
Amount of sealing material sacks ☐ pounds ☐
How was cement grout placed?

Was pump installed? Type HP Depth ft.
Was a drive shoe used? ☐ Yes ☐ No Plugs Size: location ft.
Did any strata contain unusable water? ☐ Yes ☐ No
Type of Water? depth of strata
Method of sealing strata off
Was well gravel packed? ☐ Yes ☐ No Size of gravel:
Gravel placed from ft. to ft.

(10) LOCATION OF WELL by legal description:
County ¼ ¼ of Section of
Township (Township is North or South), Range (Range is East or West), WM.
Tax Lot Lot Block Subdivision
MAILING ADDRESS OF WELL (or nearest address)

(11) WATER LEVEL of COMPLETED WELL:
Depth at which water was first found ft.
Static level ft. below land surface. Date
Artesian pressure lbs. per square inch. Date

(12) WELL LOG:
Diameter of well below casing
Depth drilled ft. Depth of completed well ft.
Formation: Describe color, texture, grain size and structure of materials; and show thickness and nature of each stratum and aquifer penetrated, with at least one entry for each change of formation. Report each change in position of Static Water Level and indicate principal water-bearing strata.

MATERIAL	From	To	SWL
Basalt grey brn broken	424	432	
Basalt grey hrd.	432	478	
Basalt grey w/red broken	478	481	
Basalt grey some red hrd.	481	493	
Basalt grey hrd.	493	531	
Basalt black brn. red prs.	531	544	
Basalt grey red some porous	544	551	
Basalt grey hard	551	567	
Basalt black red porous			
semi broken	567	583	
Basalt grey hrd	583	651	
Basalt grey hrd fractured	651	654	
Basalt grey hard	654	738	
Basalt grey hard w/frctrd	738	788	
Basalt grey hard	788	800	

Date work started /completed
Date well drilling machine moved off of well 19

(unbonded) Water Well Constructor Certification (if applicable):
This well was constructed under my direct supervision. Materials used and information reported above are true to my best knowledge and belief.
[Signed] Date 19

(bonded) Water Well Constructor Certification:
Bond (number) Issued by: (Surety Company Name)
On behalf of STACO WELL SERVICES INC.
(type or print name of Water Well Constructor)
This well was drilled under my jurisdiction and this report is true to the best of my knowledge and belief.
(Signed) (Water Well Constructor)
(Dated)

NOTICE TO WATER WELL CONSTRUCTOR
The original and first copy of this report are to be filed with the
WATER RESOURCES DEPARTMENT, SALEM, OREGON 97310
within 30 days from the date of well completion.

SP*46866-690

OCTOBER 17, 1984 SHERWOOD WELL #5

STATIC-WATER-LEVEL-- 45.33 FT. LOG WELL--TV CAMERA--MARK CHRISTENSEN

Footage	Comments
3'	Casing weld
14'	Casing weld (35.7 feet)
61.2'	Suspension material (iron, bright stuff)
99'	Water muddy
119'	Pick up another well
140'	Pick up another well (40.5)
181.1'	Casing joint
201.7'	Bottom casing depth, rough rock below it (cemented in)
209-210'	Nice vertical fractures
215	Another fracture
217'	More fractures, horizontal fracture
218-219'	Interesting "stuff"
221'	More fractures
225'	On side, hexagonal prism; nice flat fractures
225-230'	Fairly tight basalt, too many fractures, water clearing up
234	Some fractures starting to show up, water more clearer
236	Fairly smooth hole
237-238	Horizontal fracture
239	Horizontal fracture, maybe
241	Horizontal fracture, maybe
242,243,244'	Line fracture, occasional horizontal fractures
246'	Nice broken zone
246.5	Very interesting vertical fractures in rubble zone, (bouldery-looking)
248'	Light fractures, fairly & narrow broken zone; smooth hole
252'	Nice smooth hole
253'	Nice broken zone, lots of caving, very rough hole, and looks like hit in drilling
259	Rough
264-265	Rough, broken zone
267'	Rough, broken zone
270'	Rough basalt broken zone
274'	Broken zone
280	Nice fractures
284'	Nice fractures
285'	Rough fracture, broken zones
286'	Rugged looking broken zones, vertical fractures, looks like boulders
296	Vertical fractures, horizontal
303	Nice smooth hole
309'	Cross-cutting fractures; vertical block, horizontal

✓310'	Light fractures
✓316' ?	Cavern-blocking, "pillow basalt", clay in between, very interesting
✓319' ?	Smooth hole, but rough texture "pillow basalt" rougher, then top of hole
✓322' ?	Coarse texture, but smooth hole, soft drilling, some fractures
✓325' ?	Rough texture, "pillow basalt", number of fractures, rough texture to walls
✓329' ?	Some discontinuity, still rough
✓330'	Harder basalt, less fractures, smooth hole
✓342' ?	Broken zone
✓344'	Followed by dense basalt
✓353'	Smooth hole, some roughness
✓356'	Broken zone
	Broken zone, fractures
	Broken zone
	Rounded smooth, edge smooth; embedded in a matrix; limestone large holes; bouldery "stuff"
✓367'	Kind of broken zone, rounded (like limestone) (large pillows, fresh "pillow basalt"), round textured; broken zone
✓368'	Big caved area
✓384'	Vesicular basalt; hole not too broken; vesicules and holes sides of wall
386,387'	Nice smooth well bore, "pillow basalt", but rough texture --smooth textured wall
✓389'	Nice, rough "pebbly-texture" on wall
✓390,391'	Smooth well bore, harder basalt
✓393'	Nice cavern off to side
✓394'	Very blocky, rugged zone, big and blocky
✓398' ?	Nice, smooth hole, occasional fractures
	Notes: Water is coming out of all these broken zones, saturated rocks
✓404'	Nice-looking broken zone, very rugged
✓406'	Nice-looking broken blocky zone
✓407'	Very smooth hole
✓410'	Nice, smooth hole; some vertical fractures (at least one)
	Note: Most occur top & bottom basalt flows; looking at top and bottom where come together--water sets there; drill through; bit bounce around
✓418'	Nice, smooth hole
✓420'	Got flooded out, broke into a zone--not too thick--fair amount of water
✓428'	Nice, broken zone (fractures, still broken)
✓433' ?	Nice, broken zone; not as rough as other ones; finer texture, more fractures; rough fractures
	Note: Alot of foam and air went through that hole

✓438′	Little rough
✓448′	Nice, smooth hole
✓451′	Very smooth hole
✓456′ 16" DOWN TO 8"	Broken zone area; smooth hole; casing reduction, 16" to 8"
~~464~~	Nice fracture in the 8" hole
472′	"Pillow basalt", 12′ below that too
473′	Broken zone, vuggy broken zone; very interesting
475,476′	Nice, vuggy material; rough hole; nice, broken zone; "pillow basalt"
482′	Nice fractures; broken basalt
486′	Big hole, vuggy basalt (almost look like limestone again) very smooth hole; lots of vugs; broken stuff; blocky-fractures
489,490′	Nice fractures
491′	Nice, big vugs
514′	Very interesting fracture pattern
515	Horizontal fractures intersecting diagonal, some vertical Nice, smooth basalt; dark bands near very interesting fractures
510 to 519′	Nice, long fracture, ending up in a horizontal fractures, picking up some more vertical fractures; interesting fractur patterns
521′	Real smooth hole
522′	Rough broken zone
524′	Rough broken zone
526	Still broken, some cavity fillings, "pillow basalt" appearance
531,536′	"Pillow basalt"
532	Broken zone abandoned
545	Water has settled down
546	Broken zone
549	Real smooth hole
550	Fractures, broken zone; clearing up water
552	Broken zone
557	Broken basalt
560	Water clearing up
570	"Pillow basalt"
580	Pretty muddy, below

Appendix D

WASH 51903

STATE OF OREGON
WATER SUPPLY WELL REPORT
(as required by ORS 537.765)

WELL I.D. # L02186

(START CARD) # 87126

Instructions for completing this report are on the last page of this form.

(1) OWNER: Well Number 6
Name City of Sherwood
Address 540 NW Washington St.
City Sherwood State OR Zip 97140

(2) TYPE OF WORK
[X] New Well [ ] Deepening [ ] Alteration (repair/recondition) [ ] Abandonment

(3) DRILL METHOD:
[ ] Rotary Air [X] Rotary Mud [ ] Cable [ ] Auger
[X] Other Reverse Circulation Air Rotary

(4) PROPOSED USE:
[ ] Domestic [X] Community [ ] Industrial [ ] Irrigation
[ ] Thermal [ ] Injection [ ] Livestock [ ] Other

(5) BORE HOLE CONSTRUCTION:
Special Construction approval [ ] Yes [X] No Depth of Completed Well 889 ft.
Explosives used [ ] Yes [X] No Type ______ Amount ______

HOLE Diameter	From	To	SEAL Material	From	To	Sacks or pounds
24	0	9	Cement	0	301	266 sks
20	9	301				
15	301	1030	Abandonment of lower borehole:			
			Cement	889	1030	125 sks

How was seal placed: Method [ ] A [ ] B [X] C [ ] D [ ] E
[ ] Other
Backfill placed from ____ ft. to ____ ft. Material ______
Gravel placed from ____ ft. to ____ ft. Size of gravel ______

(6) CASING/LINER:

	Diameter	From	To	Gauge	Steel	Plastic	Welded	Threaded
Casing:	16	+1	301	.375	[X]	[ ]	[X]	[ ]
					[ ]	[ ]	[ ]	[ ]
					[ ]	[ ]	[ ]	[ ]
	*'J' receptor & conc. reducer				[ ]	[ ]	[ ]	[ ]
Liner:	14x12 *	289	294	Std	[X]	[ ]	[X]	[ ]
	12	294	889	.250	[X]	[ ]	[X]	[ ]

Final location of shoe(s) ______

(7) PERFORATIONS/SCREENS:
[X] Perforations Method Factory Mill Cut
[ ] Screens Type ______ Material ______

From	To	Slot size	Number	Diameter	Tele/pipe size	Casing	Liner
430	550	.19x3	5616			[ ]	[X]
640	680	.19x3	1872			[ ]	[X]
769	889	.19x3	5616			[ ]	[X]
						[ ]	[ ]
						[ ]	[ ]

(8) WELL TESTS: Minimum testing time is 1 hour

[X] Pump [ ] Bailer [ ] Air [ ] Flowing Artesian

Yield gal/min	Drawdown	Drill stem at	Time
See attached graphs			1 hr.

Temperature of water 53°F Depth Artesian Flow Found ______
Was a water analysis done? [X] Yes By whom Owner
Did any strata contain water not suitable for intended use? [ ] Too little
[ ] Salty [ ] Muddy [ ] Odor [ ] Colored [X] Other Rising SP cond. & cloudy video.
Depth of strata: 910'-1030'

SET 9637

(9) LOCATION OF WELL by legal description:
County Washington Latitude ______ Longitude ______
Township 2S N or S Range 1W E or W. WM.
Section 32 SE 1/4 NE 1/4
Tax Lot 12100 Lot ______ Block ______ Subdivision ______
Street Address of Well (or nearest address) 1830 Roy St.
Sherwood, OR 97140

(10) STATIC WATER LEVEL:
131 ft. below land surface. Date 2/7/97
Artesian pressure ______ lb. per square inch. Date ______

(11) WATER BEARING ZONES:
Depth at which water was first found indeterminate because upper 301' was drilled with mud rotary

From	To	Estimated Flow Rate	SWL
438	466		
491	501		
527	545	see (8)	see
839 788	879 801		(10)
813	860		

(12) WELL LOG:
Ground Elevation approx. 250'

Material	From	To	SWL
See attached Log			

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WATER RESOURCES DEPT.
SALEM, OREGON

Date started 9/16/96 Completed 2/7/97

(unbonded) Water Well Constructor Certification:
I certify that the work I performed on the construction, alteration, or abandonment of this well is in compliance with Oregon water supply well construction standards. Materials used and information reported above are true to the best of my knowledge and belief.
WWC Number 1367
Signed ______ Date 2/27/97

(bonded) Water Well Constructor Certification:
I accept responsibility for the construction, alteration, or abandonment work performed on this well during the construction dates reported above. All work performed during this time is in compliance with Oregon water supply well construction standards. This report is true to the best of my knowledge and belief.
WWC Number 649
Signed ______ Date 2/27/97

ORIGINAL & FIRST COPY-WATER RESOURCES DEPARTMENT SECOND COPY-CONSTRUCTOR THIRD COPY-CUSTOMER

City of Sherwood Well No. 6 Well Log

SC#87126 Label #L02186

By Schneider Drilling Co.

1996-1997

From	To	Description
0	1	Top soil
1	2	Gravel 3/4"-
2	12	Clay, brown, medium
12	17	Claystone, brown, medium-soft, broken
17	30	Claystone, brown & gray layers, medium-soft
30	35	Basalt, brown, medium-hard, fractured
35	45	Basalt, gray, medium-hard
45	59	Basalt, gray & brown, medium-soft, fractured, small layer of claystone
59	63	Basalt, gray, medium-hard, fractured
63	65	Basalt, gray, hard
65	88	Basalt, brown w/some gray, medium, fractured
88	96	Basalt, brown, med-soft, fractured
96	100	Basalt, gray, hard
100	105	Basalt, gray, hard, fractured
105	109	Basalt, brown, medium, fractured
109	131	Basalt, gray, hard
131	139	Basalt, gray, hard, some fractures
139	160	Basalt, brown-red, vesicular, medium, w/claystone layers
160	164	Basalt, gray & brown, medium-hard
164	195	Basalt, gray & brown, medium-hard, fractured
195	233	Basalt, gray, hard, fractured
233	241	Basalt, gray, hard
241	251	Basalt, black, soft, vesicular
251	260	Basalt, black & red, soft, vesicular
260	266	Basalt, black, medium-hard, fractured
266	303	Basalt, gray, hard
303	311	Basalt, gray & black, med-soft, vesicular
311	315	Basalt, gray, medium, fractured, some vesicular
315	334	Basalt, gray, hard
334	342	Basalt, black, med-soft, fractured, vesicular, small claystone layers
342	347	Basalt, black, medium, fractured
347	354	Basalt, brown & red, broken, vesicular, some claystone
354	358	Basalt, brown, fractured, vesicular
358	370	Basalt, gray, hard
370	373	Basalt, gray, med-hard, fractured, some vesicular
373	377	Basalt, gray & brown, medium, fractured
377	396	Basalt, gray, hard

RECEIVED
MAR - 4 1997
WATER RESOURCES DEPT.
SALEM, OREGON

396	399	Basalt, black, med-soft, well-fractured
399	401	Basalt, black & red, soft, vesicular, well fractured
401	412	Basalt, black, med-soft, fractured, vesicular
412	416	Basalt, gray, med-hard, fractured, some vesicular
416	435	Basalt, gray, hard
435	441	Basalt, black, med-hard, fractured
441	447	Basalt, black & red, soft, well fractured, vesicular, broken
447	451	Basalt, brown & black, soft, well fractured
451	453	Basalt, brown, red & black, soft, well broken, vesicular, some claystone
453	470	Basalt, black & gray, med-hard, fractured
470	474	Basalt, gray, hard, some fractures
474	490	Basalt, gray, hard
490	493	Basalt, black, med-hard, fractured
493	494	Basalt, red, soft, broken, vesicular, some claystone
494	496	Basalt, black & brown, soft, broken, vesicular, some claystone
496	502	Basalt, black, med-soft, well fractured, some vesicular
502	506	Basalt, black, med-hard, some fractures, some vesicular
506	529	Basalt, gray, hard, fractured
529	533	Basalt, black, med-soft, well fractured, vesicular, some claystone
533	563	Basalt, black, med, fractured, vesicular
563	574	Basalt, gray, med-hard, fractured
574	575	Basalt, gray, hard, fractured
575	605	Basalt, gray w/red streaks, hard, fractured
605	642	Basalt, gray, hard, fractured
642	652	Basalt, black-brown, soft, well fractured, some vesicular
652	657	Basalt, black & red, soft, broken, vesicular
657	674	Basalt, gray & black, med-soft, well fractured
674	678	Basalt, black, med-soft, fractured
678	688	Basalt, gray, hard, some fractures
688	700	Basalt, black, hard
700	737	Basalt, black, hard, some fractures
737	761	Basalt, gray-black, very hard, some fractures
761	762	Basalt, dark gray, very hard
762	767	Basalt, gray, very hard
767	788	Basalt, gray, hard, fractured
788	797	Basalt, black, soft, fractured, vesicular
797	805	Basalt, dark gray, med-hard, fractured, some vesicular
805	813	Basalt, gray, hard, fractured
813	830	Basalt, black, med, well-fractured, vesicular
830	853	Basalt, black, soft, well fractured, vesicular, some claystone layers
853	862	Basalt, black, med, well-fractured, some vesicular
862	892	Basalt, black & brown, soft, well fractured, vesicular
892	901	Basalt, black & brown, soft, broken, some claystone
901	936	Basalt, gray, hard, fractured

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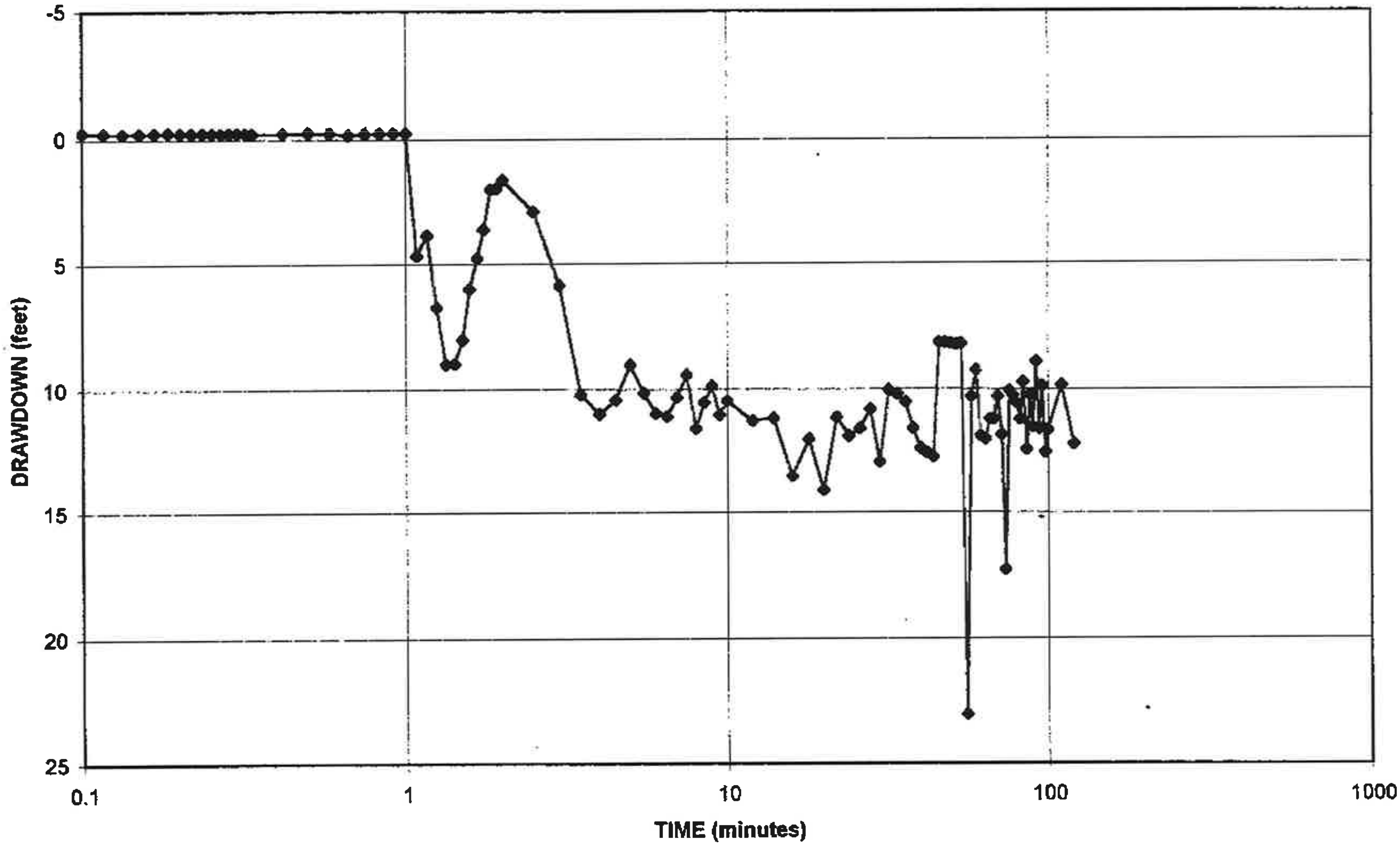
936	938	Basalt, black, med-hard, fractured, some claystone
938	943	Basalt, black, med-soft, fractured
943	948	Basalt, gray, med-hard, fractured, some vesicular
948	953	Basalt, gray, hard, fractured
953	960	Basalt, gray, med-soft, fractured, some vesicular, some claystone
960	971	Basalt, gray, med, fractured, w/some quartz
971	986	Basalt, black, med, fractured, some vesicular
986	995	Basalt, gray, hard
995	1014	Basalt, gray, med-soft, broken, vesicular, some claystone
1014	1016	Clay, gray & brown, soft
1016	1023	Clay, gray & brown, soft, little sandy
1023	1030	Clay, blue, med-soft

RECEIVED

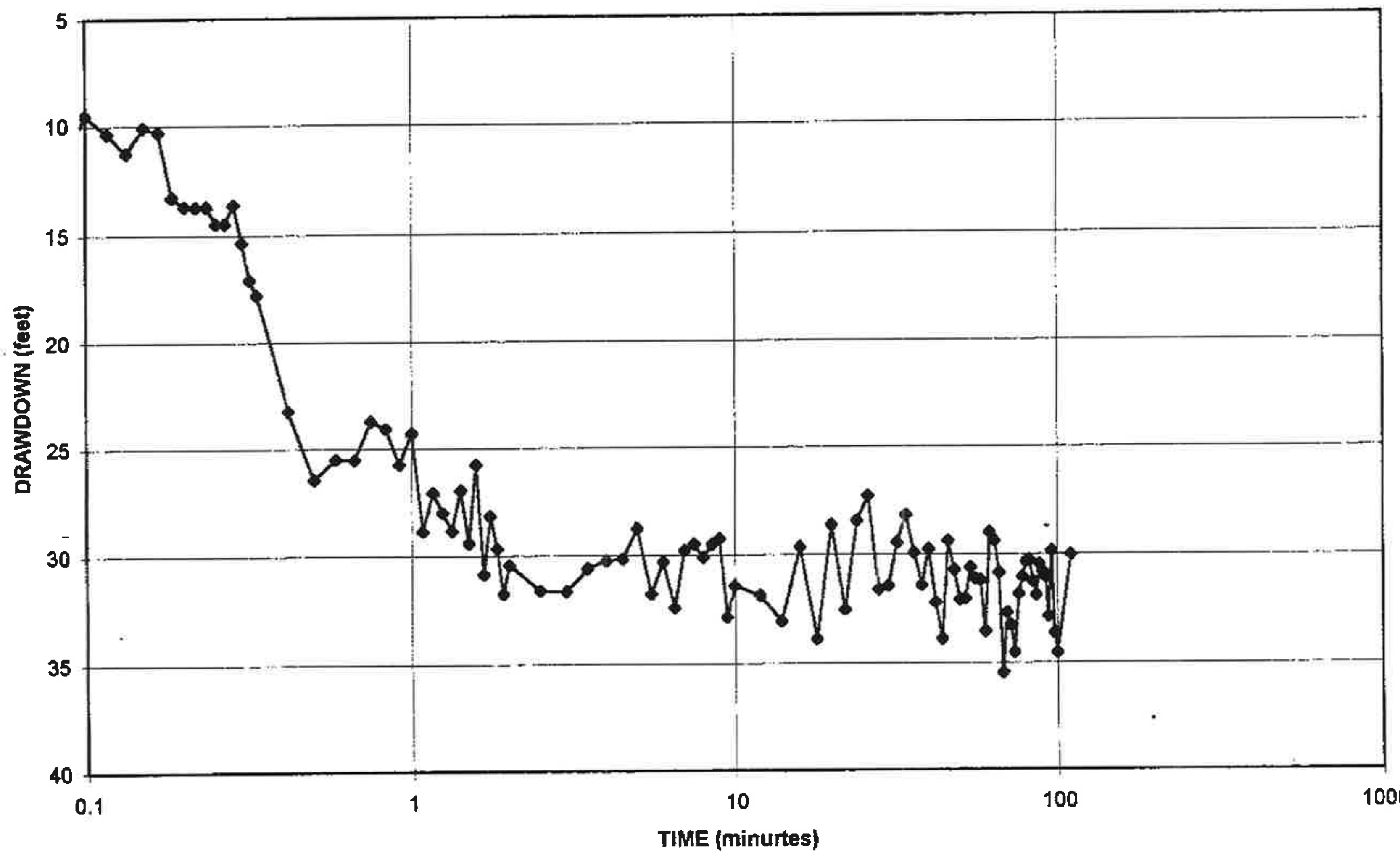
MAR - 4 1997

WATER RESOURCES DEPT.
SALEM, OREGON

SHERWOOD WELL NO.6
1/24/97 STEP 1 - 500 GPM



SHERWOOD WELL NO.6
1/24/97 STEP 2 - 1000 GPM



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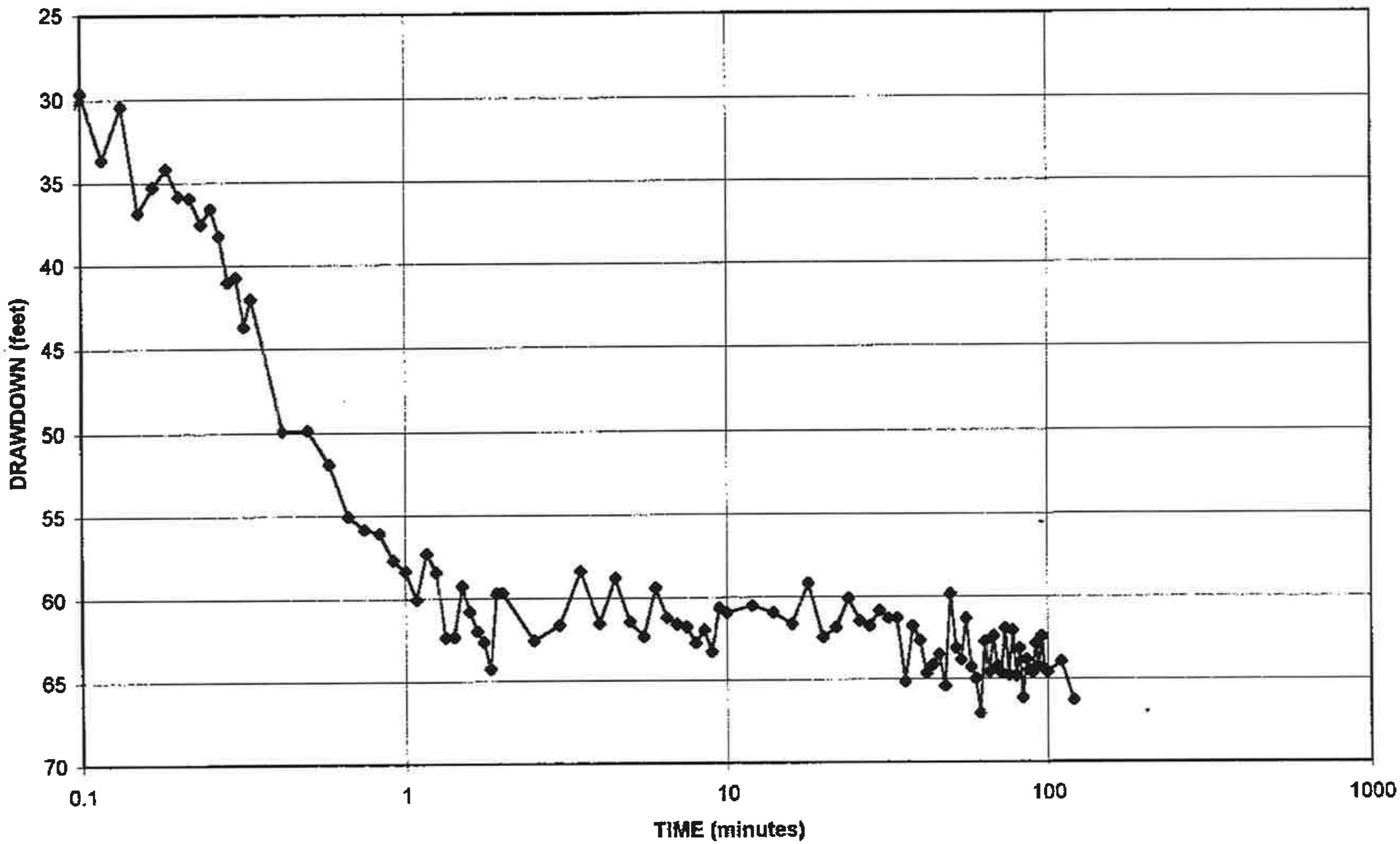
WATER RESOURCES DEPT.
SALEM, OREGON

SHERWOOD WELL NO.6
1/24/97 STEP 3 - 1500 GPM



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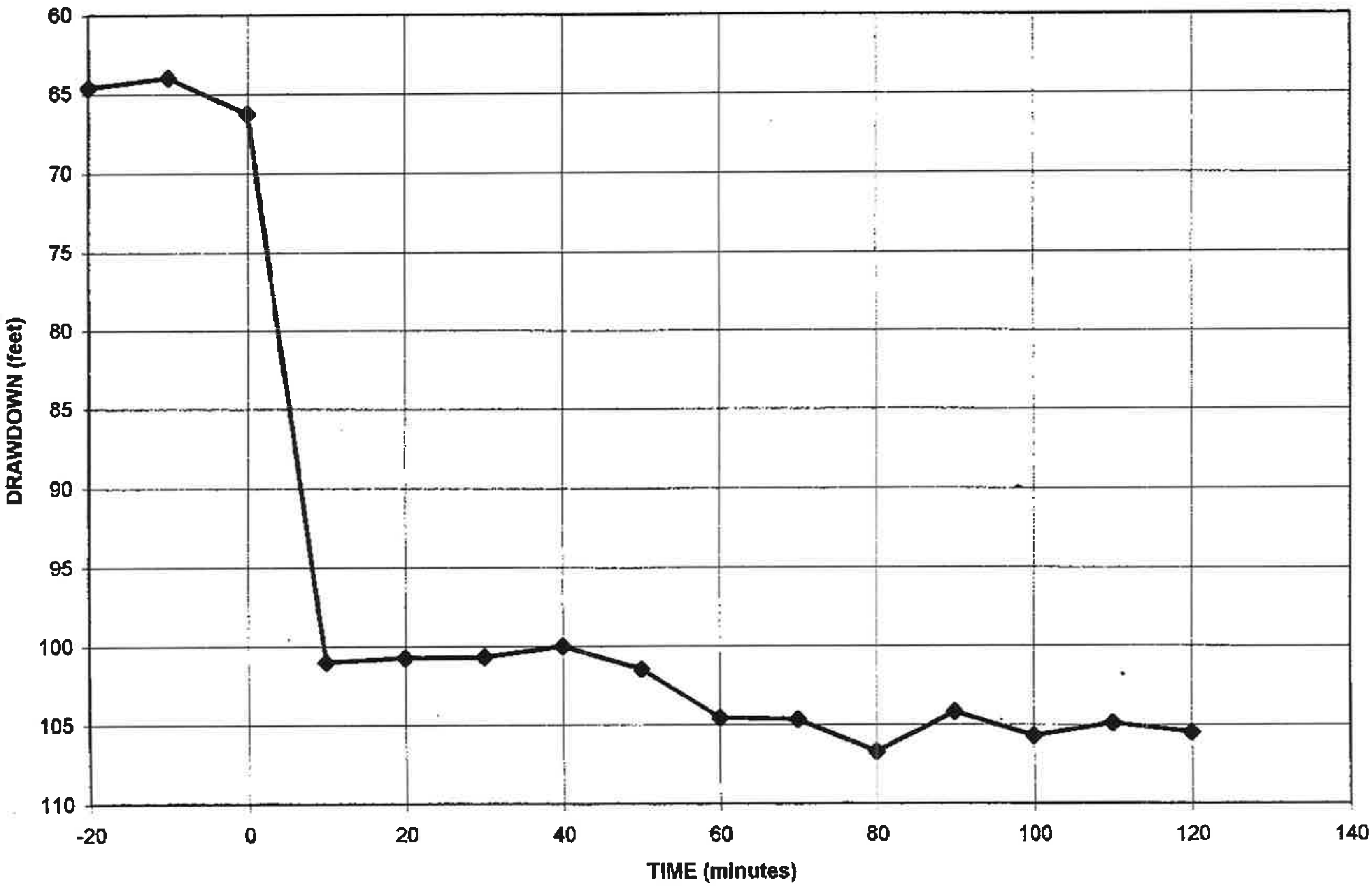
SHERWOOD WELL NO. 6
1/24/97 STEP 4 - 2000 GPM



Note: Not logarithmic scale

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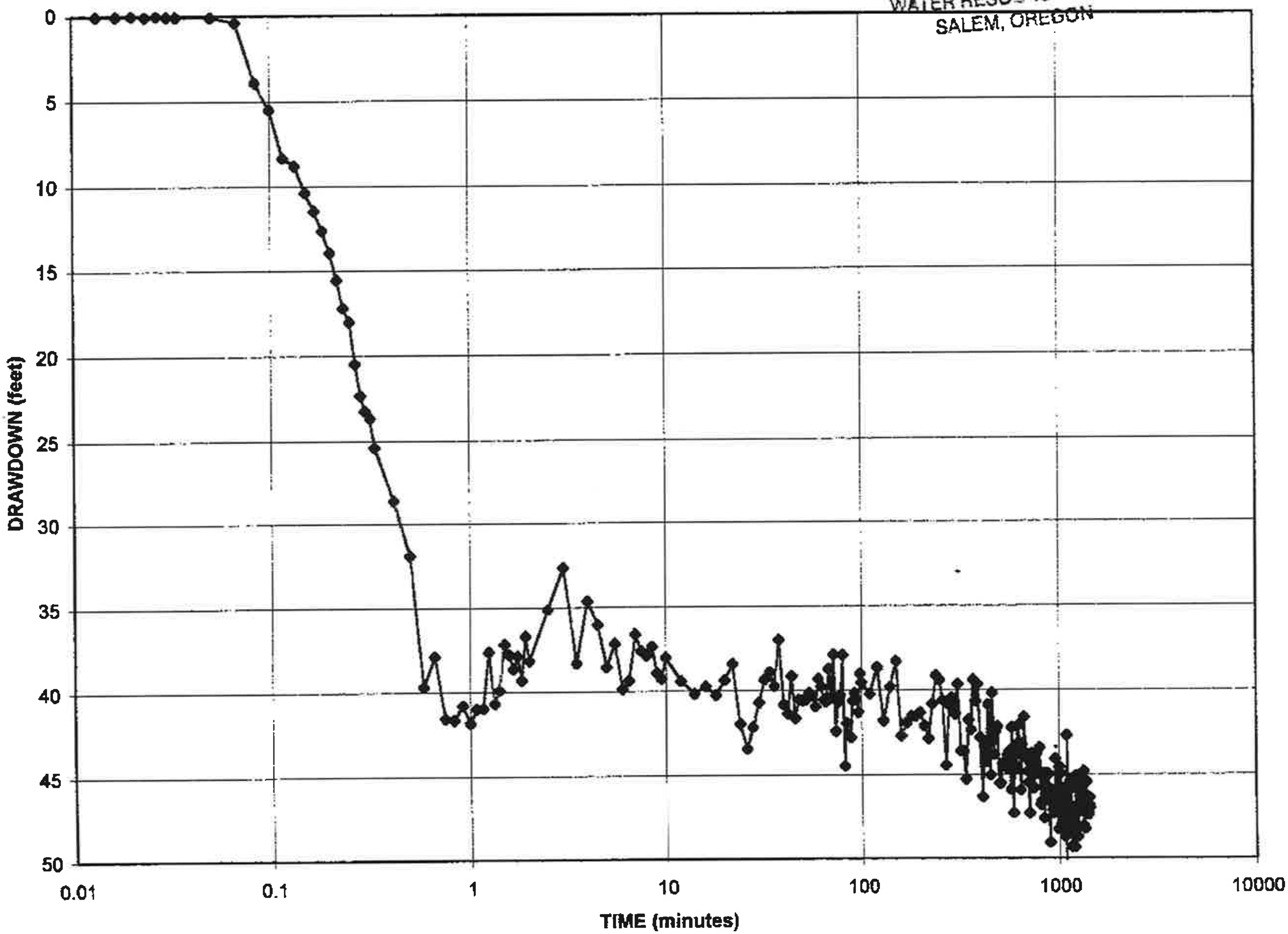
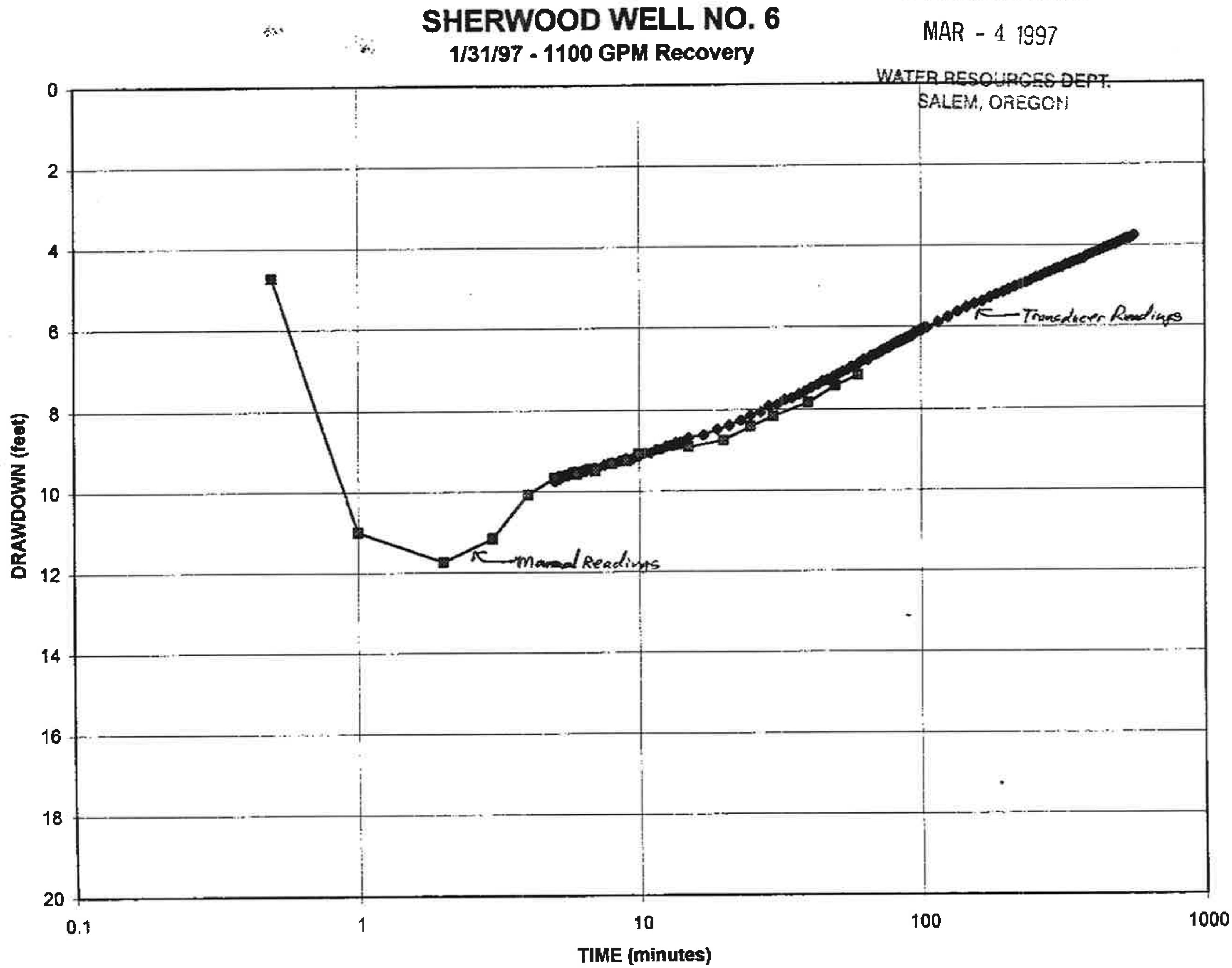
WATER RESOURCES DEPT.
SALEM, OREGON

SHERWOOD WELL NO. 6

1/30-31/97 - 1100 GPM Test




SHERWOOD WELL NO. 6
1/31/97 - 1100 GPM Recovery
DRAWDOWN (feet)
0
2
4
6
8
10
12
14
16
18
20
TIME (minutes)
0.1
1
10
100
1000
Manual Readings
Transducer Readings

Appendix E

Sunday, June 27, 1999

Squier Associates
4260 Galewood Street
Lake Oswego, OR 97035

Attn: David King

Re: Summary of City of Sherwood water well logs

This report is a summary of the stratigraphic interpretation of the water wells (Sherwood #5 and Sherwood #6) drilled for the city of Sherwood, Oregon. These logs were possible because I was given cuttings from these wells and had cleaned, logged, and saved them for my research on the distribution of Columbia River Basalt Group (CRBG) flows. The stratigraphic sections are the result of a visual inspection of the cuttings, in order to characterize the lithology of the lava flows, and chemical analysis of selected samples by Concord Analytical Services. Enclosed are logs for the Sherwood #5, Sherwood #6, and a nearby well along Cedar Creek. Chemical analyses of selected samples from these drill holes, and graphs illustrating chemical compositional changes with depth are also presented. The chemical analyses generally confirm stratigraphic identifications of the Columbia River Basalt Group (CRBG) lava flows made on the basis of visual inspection and relative position except for the lowermost flow in the Sherwood #6, which is occurs below the Wapshilla Ridge flows.

The chemical data enclosed for Sherwood # 6 was obtained specifically for this project from Concord Analytical Services. Chemical data for Sherwood # 5 and the Cedar Creek well were obtained several years ago from the Washington State University GeoAnalytical Laboratory for my research. Since there are systematic differences in data from different laboratories or even from the same laboratory at different times, one should not attempt to compare the absolute concentration values between batches of data. However, inspection of the graphs of TiO_2 concentrations plotted against depth clearly shows that relative changes with depth is similar for all of the wells. Data from the earlier analyses are unpublished and are presented here for use on this project only.

The graphs of TiO_2 show how chemical differences help to confirm and refine the stratigraphy based on lithology. The chemical stratigraphy of the low-MgO part of the section between the Sentinel Bluffs unit and the Wapshilla Ridge unit is very subtle and might better be left undifferentiated except for the Winter Water, which has distinctive small, blocky plagioclase phenocrysts. There are subtle differences in TiO_2 concentrations, changing from slightly higher in the Winter Water to slightly lower in the Ortley to slightly higher in the Umtanum to slightly lower again in the Grouse Creek. I identified these units on the logs on this basis, and although similar chemical variations are seen in both of these wells, one should not place too much confidence in these fine distinctions unless further drilling corroborates it. The differences in concentrations between units is not much greater than the average analytical uncertainty and the vertical variation within a flow, meaning that it is statistical possible that any given sample might not be identifiable with certainty. TiO_2 is distinctly higher in the Wapshilla Ridge, making this contact more certain. The Grouse Creek and Wapshilla Ridge units display a reversed magnetic polarity, but this characteristic cannot be measured from cuttings. Although the Sentinel Bluffs Member was not detected in the wells, it crops out very near the Sherwood #6 locality and would therefore be in its proper position at the top of the drill hole stratigraphy.

Hydrogeology - it is interesting to note that every water-bearing zone indicated on the driller's log for Sherwood #6 is also a vesicular flow top or basal pillow basalt (813' – 860'). This emphasizes the strong control that the stratigraphy and physical characteristics of the basalt flows have on hydrology in the Columbia River basalt. Although tectonic fracturing and faulting often produces secondary permeability, this does not seem to be a factor in this well. Comparing the well logs for Sherwood #6 with Sherwood #5 indicates that Sherwood #5 terminated before reaching the water-rich pillow basalt encountered in Sherwood #6. Pillow basalts in the CRBG were formed where the lava flows encountered standing water and their lateral extents are limited by the topography at the time. Therefore, although pillow basalts probably also occur below the bottom of Sherwood #5, there is no guarantee.

In conclusion, I think that the combination of lithology and chemistry has allowed us to construct an accurate stratigraphic section that can help form a stratigraphic architecture suitable for hydrologic modeling and that can also be used as a standard for comparing future well logs. Please let me know if there are other points that you would like me to address or if there are questions and comments on these results.

Sincerely,

Marvin H. Beeson

Marvin H. Beeson
Geology Consultant (Oregon G493)
7264 SE Wilshire Court
Milwaukie, OR 97267

Log of Borehole Sherwood #6

Project: City of Sherwood

Street Location: Murdock Road (near 1830 Roy Street)

Township, Range, Section: 2S, 1W, 32, SE of NE

Geologist: Marvin H. Beeson

SUBSURFACE PROFILE

Depth	Symbol	Description	Elevation	Remarks
0		Ground Surface	250	
		Clay and claystone (no samples saved)	220	
		basalt, brown (no cuttings saved)	205	
		Sentinel Bluffs flow ? gray basalt (no cuttings saved)		
		basalt, brown and gray (no cuttings saved)	186	
		claystone?		
		brown basalt (no cuttings saved)	154	
100		***basalt, gray*** (no cuttings saved)	141	
		Winter Water I flow	111	XRF sample - 120'
		Winter Water II flow top Oxidized, weathered, cavity fillings	90	
200		***Winter Water II flow*** a few small phenocrysts		

Drilled By: Schnieder
Drill Method: Air Rotary
Drill Date: 2/7/97
Contacts: are approximate, may be gradational

Marvin H. Beeson
7264 SE Wilshire Court
Milwaukie, OR 97267

Hole Size: 24" to 20" to 15"
Sampling Interval: 5 ft.

Log of Borehole Sherwood #6

Project: City of Sherwood

Street Location: Murdock Road (near 1830 Roy Street)

Township, Range, Section: 2S, 1W, 32, SE of NE

Geologist: Marvin H. Beeson

SUBSURFACE PROFILE

Depth	Symbol	Description	Elevation	Remarks
			50	
200		***Winter Water II flow*** a few small phenocrysts	9	XRF sample - 225'
		Ortley flow top little oxidation or weathering, a few chips of sediment	-15	
		Ortley flow	-53	XRF sample - 296'
300		***Vesicular zone***	-61	
		Ortley flow	-84	XRF sample - 315'
		Vesicular zone		
		Ortley flow	-97	XRF sample - 342'
		Umtanum flow top red, oxidized, few vesicles	-108	
		Umtanum flow fresh gray rock with a slightly open texture	-146	XRF sample - 377'
400				

Drilled By: Schnieder

Drill Method: Air Rotary

Drill Date: 2/7/97

Contacts: are approximate, may be gradational

Marvin H. Beeson
7264 SE Wilshire Court
Milwaukie, OR 97267

Hole Size: 24" to 20" to 15"

Sampling Interval: 5 ft.

Sheet: 2 of 6

Log of Borehole Sherwood #6

Project: City of Sherwood

Street Location: Murdock Road (near 1830 Roy Street)

Township, Range, Section: 2S, 1W, 32, SE of NE

Geologist: Marvin H. Beeson

SUBSURFACE PROFILE

Depth	Symbol	Description	Elevation	Remarks
			-150	
400		***Grouse Creek flow top*** slightly oxidized, black basalt, few sediment chips, pyrite	-165	
		Grouse Creek I flow a few microphenocrysts	-191	XRF sample - 426'
		Vesicular flow top weathered and oxidized	-210	W.B. zone - 438'-466'
		Grouse Creek II flow a few microphenocrysts	-243	XRF sample - 480' W.B. zone - 491'-501'
500		***Wapshilla Ridge flow top*** weathered, oxidized, some sediment	-255	XRF sample - 506'
		Wapshilla Ridge flow microphyric	-279	
		Vesicular zone not oxidized (not flow top)	-293	W.B. zone - 527'-545' XRF sample - 542'
		Wapshilla Ridge flow microphyric		XRF sample - 573'
600				

Drilled By: Schnieder

Drill Method: Air Rotary

Drill Date: 2/7/97

Contacts: are approximate, may be gradational

Marvin H. Beeson
7264 SE Wilshire Court
Milwaukie, OR 97267

Hole Size: 24" to 20" to 15"

Sampling Interval: 5 ft.

Sheet: 3 of 6

Log of Borehole Sherwood #6

Project: City of Sherwood

Street Location: Murdock Road (near 1830 Roy Street)

Township, Range, Section: 2S, 1W, 32, SE of NE

Geologist: Marvin H. Beeson

SUBSURFACE PROFILE

Depth	Description	Elevation	Remarks
600		-350	
			XRF sample - 625'
		-392	
	Vesicular flow top slight weathering and oxidation		W.B. zone - 639'-676'
		-415	
700			
	Wapshilla Ridge II flow microphyric		
			XRF sample - 758'
		-538	
			W.B. zone - 788'-801'
800			XRF sample - 797'

Drilled By: Schnieder

Drill Method: Air Rotary

Drill Date: 2/7/97

Contacts: are approximate, may be gradational

Marvin H. Beeson
7264 SE Wilshire Court
Milwaukie, OR 97267

Hole Size: 24" to 20" to 15"

Sampling Interval: 5 ft.

Log of Borehole Sherwood #6

Project: City of Sherwood

Street Location: Murdock Road (near 1830 Roy Street)

Township, Range, Section: 2S, 1W, 32, SE of NE

Geologist: Marvin H. Beeson

SUBSURFACE PROFILE

Depth	Symbol	Description	Elevation	Remarks
			-550	
800		***Vesicular zone*** not oxidized (not flow top) (Vesicles common to base of basalt)	-573	
		Pillow basalt glass chips and pyrite common	-612	W.B. zone - 813'-860'
900		***Vesicular flow top*** some weathering and oxidation not microphyric	-660	
		Mt. Horrible/Downey Gulch? This flow is not microphyric and has a lower TiO2 content than Wapshilla Ridge. It could be either Mt. Horrible or Downey Gulch. clay and siltstone chips (mica)	-703	XRF sample - 921'
1e+03		***Vesicular flow base*** not microphyric, secondary calcite mix - sediment, vesicular basalt		

Drilled By: Schnieder
Drill Method: Air Rotary
Drill Date: 2/7/97
Contacts: are approximate, may be gradational

Marvin H. Beeson
7264 SE Wilshire Court
Milwaukie, OR 97267

Hole Size: 24" to 20" to 15"
Sampling Interval: 5 ft.
Sheet: 5 of 6

Log of Borehole Sherwood #6

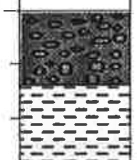
Project: City of Sherwood

Street Location: Murdock Road (near 1830 Roy Street)

Township, Range, Section: 2S, 1W, 32, SE of NE

Geologist: Marvin H. Beeson

SUBSURFACE PROFILE

Depth	Symbol	Description	Elevation
1e+03			-750
			-764
		Micaeous siltstone glass in sediment - (peperite)	-778
		End of Borehole	
1.1e+03			
1.2e+03			

Drilled By: Schnieder

Drill Method: Air Rotary

Drill Date: 2/7/97

Contacts: are approximate, may be gradational

Marvin H. Beeson
7264 SE Wilshire Court
Milwaukie, OR 97267

Hole Size: 24" to 20" to 15"

Sampling Interval: 5 ft.

Log of Borehole Sherwood #5

Project: City of Sherwood

Street Location: Wilsonville Road (SW Sunset Blvd.)

Township, Range, Section: T2S, R1W, Sec 32, NW of SW

Geologist: Marvin H. Beeson

SUBSURFACE PROFILE

Depth	Symbol	Description	Elevation	Remarks
0		Ground Surface	190	
		Clay topsoil over hard clay	175	
		Sandy clay with rock (no cuttings saved)	143	
		Clay and claystone gray sticky clay w/ green claystone streaks	107	
100		***Winter Water flow*** (most of this flow appears to be weathered and oxidized and broken. A few fresher zones occur within the flow and near the base.) few blocky phenocrysts		XRF sample - SH1 XRF sample - SH2
200				XRF sample - SH3

Drilled By: Staco (Steven Stadli)

Drill Method: Air Rotary

Drill Date: 10/18/84

Contacts: are approximate, may be gradational

Marvin H. Beeson
7264 SE Wilshire Court
Milwaukie, OR 97267

Hole Size: 15" to 8"

Sampling Interval: 5 ft.

Sheet: 1 of 4

Log of Borehole Sherwood #5

Project: City of Sherwood

Street Location: Wilsonville Road (SW Sunset Blvd.)

Township, Range, Section: T2S, R1W, Sec 32, NW of SW

Geologist: Marvin H. Beeson

SUBSURFACE PROFILE

Depth	Symbol	Description	Elevation	Remarks
200			-10	
			-65	XRF sample - SH4
		Vesicular flow top mix of black scoria and oxidized chips	-90	W.B. zone - 252'-266'
300		***Ortley flow?***	-127	XRF sample - SH5 W.B. zone - 283'-288'
		Vesicular flow top black with little oxidation	-138	
		Ortley flow gray, open textured, becoming denser toward the base	-170	XRF sample - SH6
		Vesicular flow top mix of black and oxidzed scoria chips	-185	XRF sample - SH7
400		***Umtanum flow*** gray basalt w/ few vugs and vesicles becoming fewer w/ depth		

Drilled By: Staco (Steven Stadli)

Drill Method: Air Rotary

Drill Date: 10/18/84

Contacts: are approximate, may be gradational

Marvin H. Beeson
7264 SE Wilshire Court
Milwaukie, OR 97267

Hole Size: 15" to 8"

Sampling Interval: 5 ft.

Sheet: 2 of 4

Log of Borehole Sherwood #5

Project: City of Sherwood

Street Location: Wilsonville Road (SW Sunset Blvd.)

Township, Range, Section: T2S, R1W, Sec 32, NW of SW

Geologist: Marvin H. Beeson

SUBSURFACE PROFILE

Depth	Symbol	Description	Elevation	Remarks
			-210	
400		***Umtanum flow*** gray basalt w/ few vugs and vesicles becoming fewer w/ depth	-240	XRF sample - SH8
		Vesicular flow top oxidized	-246	
		Grouse Creek flow	-285	XRF sample - SH9 XRF sample - SH10
		Vesicular flow top oxidized	-295	
500		***Wapshilla Ridge flow*** microphyric	-332	XRF sample - SH11
		Vesicular flow top oxidized; tuff fragments?	-350	
		Wapshilla Ridge flow	-370	XRF sample - SH12
		Vesicular flow top oxidized	-395	
600				

Drilled By: Staco (Steven Stadli)
Drill Method: Air Rotary
Drill Date: 10/18/84
Contacts: are approximate, may be gradational

Marvin H. Beeson
7264 SE Wilshire Court
Milwaukie, OR 97267

Hole Size: 15" to 8"
Sampling Interval: 5 ft.
Sheet: 3 of 4

Log of Borehole Sherwood #5

Project: City of Sherwood

Street Location: Wilsonville Road (SW Sunset Blvd.)

Township, Range, Section: T2S, R1W, Sec 32, NW of SW

Geologist: Marvin H. Beeson

SUBSURFACE PROFILE

Depth	Symbol	Description	Elevation	Remarks
600			-410	
				XRF sample - SH13
				XRF sample - SH14
700		***Wapshilla Ridge flow*** very microphyric		
				XRF sample - SH15
				XRF sample - SH16
800			-610	XRF sample - SH17

Drilled By: Staco (Steven Stadli)

Drill Method: Air Rotary

Drill Date: 10/18/84

Contacts: are approximate, may be gradational

Marvin H. Beeson
7264 SE Wilshire Court
Milwaukie, OR 97267

Hole Size: 15" to 8"

Sampling Interval: 5 ft.

Sheet: 4 of 4

Log of Borehole Cedar Creek Well

Project: City of Sherwood

Street Location:

Township, Range, Section: T3S, R2W, section 11, NE of SE

Geologist: Marvin H. Beeson

SUBSURFACE PROFILE

Depth	Symbol	Description	Elevation	Remarks
		Ground Surface	290	
0		***Silt*** brown soil grading to gray silt to brown silt at 60'	225	
100		***Winter Water flow*** brown weathered basalt to 90' (altered zone at 120'; contact or fracture?) one or two small phenocrysts seen	115	XRF sample - 100' XRF sample - 140'
		Ortley flow top black, few vesicles		
200		***Ortley flow*** gray to black		

Drilled By: Ross A. Jannsen

Drill Method:

Drill Date: 8/21/78

Contacts: Are approximate and may be gradational

Marvin H. Beeson
7264 SE Wilshire Court
Milwaukie, OR 97267

Hole Size: 8 in.

Sampling Interval: 10 ft.

Sheet: 1 of 3

Log of Borehole Cedar Creek Well

Project: City of Sherwood

Street Location:

Township, Range, Section: T3S, R2W, section 11, NE of SE

Geologist: Marvin H. Beeson

SUBSURFACE PROFILE

Depth	Symbol	Description	Elevation	Remarks
200			90	
		Ortley flow gray to black	65	XRF sample - 210'
		Vesicular zone weathered, vesicular, no oxidation	55	
300		*Ortley flow* gray	-45	XRF sample - 250' XRF sample - 280' XRF sample - 320'
		Vesicular zone no oxidation	-55	XRF sample - 340'
		Umtanum flow ?	-85	
		Vesicular flow top vesicular, oxidized	-98	
400		***Umtanum/Grouse Creek ?*** not microphyric, dense		

Drilled By: Ross A. Jannsen

Drill Method:

Drill Date: 8/21/78

Contacts: Are approximate and may be gradational

Marvin H. Beeson
7264 SE Wilshire Court
Milwaukie, OR 97267

Hole Size: 8 in.

Sampling Interval: 10 ft.

Log of Borehole Cedar Creek Well

Project: City of Sherwood

Street Location:

Township, Range, Section: T3S, R2W, section 11, NE of SE

Geologist: Marvin H. Beeson

SUBSURFACE PROFILE

Depth	Symbol	Description	Elevation	Remarks
400			-110	
		Umtanum/Grouse Creek ? not microphyric, dense	-125	XRF sample - 410'
		Wapshilla Ridge flow top oxidized, vesicular, microphyric	-135	
		non vesicular zone	-145	
		Vesicular zone not oxidized	-165	
		Wapshilla Ridge flow microphyric, dense	-210	XRF sample - 470' XRF sample - 480'
500		End of Borehole		
600				

Drilled By: Ross A. Jannsen	**Marvin H. Beeson**	Hole Size: 8 in.
Drill Method:	**7264 SE Wilshire Court**	Sampling Interval: 10 ft.
Drill Date: 8/21/78	**Milwaukie, OR 97267**	Sheet: 3 of 3
Contacts: Are approximate and may be gradational		

CONCORD ANALYTICAL SERVICES LIMITED

SHERWOOD #6 WELL

8540 Keele Street, Unit 38, Concord, Ont. L4K 2N2 CANADA
phone: (905) 660-5171, fax: (905) 660-9474
email: chemtest@pathcom.com, web: www.pathcom.com/~chemtest

Marvin H. Beeson
7264 SE Wilshire Court
Milwaukie, OR 97267

April 5,1999

CASL WO# MHB7016

SAMPLE ID	SiO2 %	Al2O3 %	TiO2 %	Fe2O3 %	MnO %	CaO %	MgO %	K2O %	Na2O %	P2O5 %	LOI %	TOTAL %	FLOW ID
SHER #6-120	55.1	13.2	2.09	13.9	0.20	6.91	3.30	1.69	3.65	0.36	-0.39	99.99	Winter Water
SHER #6-225	54.8	13.3	2.15	14.3	0.21	7.07	3.40	1.64	3.51	0.36	-0.47	100.22	Winter Water
SHER #6-296	55.1	13.5	1.94	13.3	0.20	6.95	3.43	1.82	3.26	0.33	0.15	99.94	Ortley
SHER #6-315	54.2	13.5	1.80	12.9	0.20	7.44	3.84	1.74	3.06	0.30	0.25	99.30	Ortley
SHER #6-342	55.1	13.5	1.96	13.1	0.19	6.91	3.37	1.99	3.11	0.35	0.13	99.78	Ortley
SHER #6-377	54.9	13.5	2.03	13.5	0.19	7.12	3.55	1.78	3.31	0.37	-0.19	100.06	Umtanum
SHER #6-426	54.9	13.7	1.91	13.0	0.19	7.31	3.76	1.75	3.30	0.34	-0.20	99.93	Grouse Creek
SHER #6-480	55.0	13.2	2.12	13.9	0.19	6.80	3.28	1.88	3.39	0.38	-0.28	99.85	Grouse Creek
SHER #6-506	54.6	13.5	2.35	13.4	0.19	6.98	3.45	2.06	3.17	0.38	0.16	100.14	Wapshilla Ridge
SHER #6-543	53.7	13.6	2.40	14.2	0.19	6.93	3.32	1.88	3.12	0.37	0.45	100.14	Wapshilla Ridge
SHER #6-573	54.8	13.6	2.37	13.6	0.18	6.84	3.29	1.89	3.41	0.38	-0.16	100.21	Wapshilla Ridge
SHER #6-625	54.7	13.4	2.36	13.7	0.18	6.86	3.36	2.03	3.25	0.38	-0.17	100.16	Wapshilla Ridge
SHER #6-758	54.0	13.34	2.30	13.98	0.19	6.97	3.42	1.81	3.21	0.38	0.28	99.90	Wapshilla Ridge
SHER #6-797	54.1	13.37	2.32	13.72	0.19	6.98	3.43	1.97	3.12	0.38	0.05	99.65	Wapshilla Ridge
SHER #6-921	53.4	13.3	2.25	14.2	0.20	7.13	3.56	1.67	3.18	0.38	0.69	100.01	Downey Gulch ?

Laboratory Manager: ______________________
M. L. G. (Gary) Gerritse

CONCORD ANALYTICAL SERVICES LIMITED

SHERWOOD #6 WELL

8540 Keele Street, Unit 38, Concord, Ont. L4K 2N2 CANADA
phone: (905) 660-5171, fax: (905) 660-9474
email: chemtest@pathcom.com, web: www.pathcom.com/~chemtest

Marvin H. Beeson
7264 SE Wilshire Court
Milwaukie, OR 97267

April 5,1999

CASL WO# MHB7016

SAMPLE ID	Ag ppm	As ppm	Ba ppm	Be ppm	Cd ppm	Cr ppm	Cu ppm	Ni ppm	Pb ppm	Sb ppm	Se ppm	Sn ppm	V ppm	Zn ppm
SHER #6-120	<1	<4	571	<1	<1	12	18	28	<4	<4	<20	<2	323	123
SHER #6-225	<1	<4	567	<1	<1	10	3	12	<4	<4	<20	3	353	122
SHER #6-296	<1	<4	612	<1	<1	6	5	12	<4	<4	<20	<2	291	110
SHER #6-315	<1	<4	542	<1	<1	14	7	13	<4	<4	<20	<2	292	104
SHER #6-342	<1	<4	622	<1	<1	12	7	13	<4	<4	<20	2	308	113
SHER #6-377	<1	<4	612	<1	<1	15	8	17	<4	<4	<20	2	324	114
SHER #6-426	<1	<4	585	<1	<1	13	14	40	<4	<4	<20	3	309	108
SHER #6-480	<1	8	642	<1	<1	7	12	14	<4	<4	<20	<2	333	114
SHER #6-506	<1	<4	655	<1	<1	10	16	15	<4	<4	<20	2	353	119
SHER #6-543	<1	<4	622	<1	<1	9	10	14	<4	<4	<20	3	350	117
SHER #6-573	<1	<4	633	<1	<1	9	19	12	<4	<4	<20	5	352	123
SHER #6-625	<1	<4	645	<1	<1	9	9	14	<4	<4	<20	<2	352	117
SHER #6-758	<1	<4	618	<1	<1	10	12	15	<4	<4	<20	4	348	114
SHER #6-797	<1	<4	637	<1	<1	10	11	15	<4	<4	<20	4	363	117
SHER #6-921	<1	<4	578	<1	<1	10	15	15	<4	<4	<20	5	383	114

Laboratory Manager: ______________________
M. L. G. (Gary) Gerritse

WSU GeoAnalytical Laboratory
(old analysis, 1985)

SHERWOOD #5 WELL

Sample ID	Depth	SiO2	Al2O3	TiO2	Fe2O3	FeO	MnO	CaO	MgO	K2O	Na2O	P2O5	TOTAL	FLOW ID
SH #5-1	95-100	55.10	15.85	2.40	2.00	10.17	0.17	6.67	3.36	1.31	2.51	0.45	99.99	Winter Water
SH #5-2	105-110	55.05	15.76	2.38	2.00	10.30	0.18	7.13	3.24	1.01	2.61	0.36	100.02	Winter Water
SH #5-3	195-200	57.70	16.38	2.48	2.00	9.61	0.16	4.90	2.02	1.74	2.65	0.38	100.02	Winter Water
SH #5-4	250-255	55.19	14.89	2.25	2.00	10.60	0.20	7.03	3.57	1.47	2.48	0.33	100.01	Winter Water
SH #5-5	280-285	56.27	15.57	2.12	2.00	8.56	0.18	7.13	3.62	1.89	2.35	0.32	100.01	Ortley
SH #5-6	345-350	55.25	15.29	1.95	2.00	9.45	0.19	7.45	4.17	1.58	2.38	0.27	99.98	Ortley
SH #5-7	375-385	55.98	15.47	2.15	2.00	8.92	0.18	7.15	3.76	1.79	2.28	0.32	100.00	Ortley
SH #5-8	424-430	55.44	15.34	2.20	2.00	9.85	0.18	7.01	3.71	1.47	2.48	0.33	100.01	Umtanum
SH #5-9	455-460	54.89	15.13	2.04	2.00	9.89	0.19	7.37	4.13	1.46	2.61	0.30	100.01	Grouse Creek
SH #5-10	470-475	55.18	15.23	2.06	2.00	9.75	0.19	7.36	4.09	1.47	2.38	0.30	100.01	Grouse Creek
SH #5-11	510-515	54.75	15.08	2.42	2.00	10.37	0.20	7.01	3.74	1.67	2.44	0.33	100.01	Wapshilla Ridge
SH #5-12	550-555	55.22	15.08	2.37	2.00	10.15	0.19	7.03	3.70	1.68	2.26	0.33	100.01	Wapshilla Ridge
SH #5-13	625-630	54.88	15.10	2.46	2.00	10.35	0.19	6.91	3.66	1.66	2.44	0.34	99.99	Wapshilla Ridge
SH #5-14	675-680	55.10	15.02	2.47	2.00	10.35	0.19	6.90	3.59	1.66	2.38	0.33	99.99	Wapshilla Ridge
SH #5-15	740-745	54.74	15.24	2.44	2.00	10.25	0.19	6.98	3.82	1.62	2.38	0.34	100.00	Wapshilla Ridge
SH #5-16	775-780	54.94	14.98	2.46	2.00	10.32	0.20	7.08	3.66	1.63	2.40	0.34	100.01	Wapshilla Ridge
SH #5-17	795-800	54.89	14.99	2.44	2.00	10.39	0.19	6.98	3.73	1.70	2.36	0.33	100.00	Wapshilla Ridge

WSU GeoAnalytical Laboratory
(old analysis, 1986)

Cedar Creek well

SAMPLE ID	SiO2	Al203	TiO2	Fe2O3	FeO	MnO	CaO	MgO	K2O	Na2O	P2O5	TOTAL	FLOW ID
CW100	57.04	16.05	2.55	2.00	11.51	0.17	4.61	2.17	1.73	1.83	0.35	100.01	Winter Water
CW140	55.75	14.98	2.22	2.00	10.42	0.19	6.67	3.42	1.51	2.48	0.34	99.98	Winter Water
CW210	55.48	14.90	2.11	2.00	10.13	0.19	7.11	3.70	1.75	2.32	0.30	99.99	Ortley
CW250	55.26	15.11	2.15	2.00	9.89	0.20	7.20	3.78	1.78	2.31	0.33	100.01	Ortley
CW280	55.11	14.94	1.97	2.00	9.84	0.19	7.51	4.12	1.57	2.46	0.28	99.99	Ortley
CW320	54.54	14.95	2.19	2.00	10.58	0.20	7.31	4.00	1.50	2.39	0.33	99.99	Ortley
CW340	55.57	14.94	2.17	2.00	10.15	0.18	6.99	3.70	1.50	2.46	0.34	100.00	Ortley
CW410	55.27	14.74	2.25	2.00	10.67	0.20	6.83	3.52	1.73	2.42	0.34	99.97	Umtanum?
CW470	54.29	15.06	2.50	2.00	10.85	0.19	7.14	3.72	1.62	2.30	0.32	99.99	Wapshilla Ridge
CW480	54.05	15.21	2.51	2.00	11.25	0.19	7.16	3.70	1.35	2.25	0.32	99.99	Wapshilla Ridge

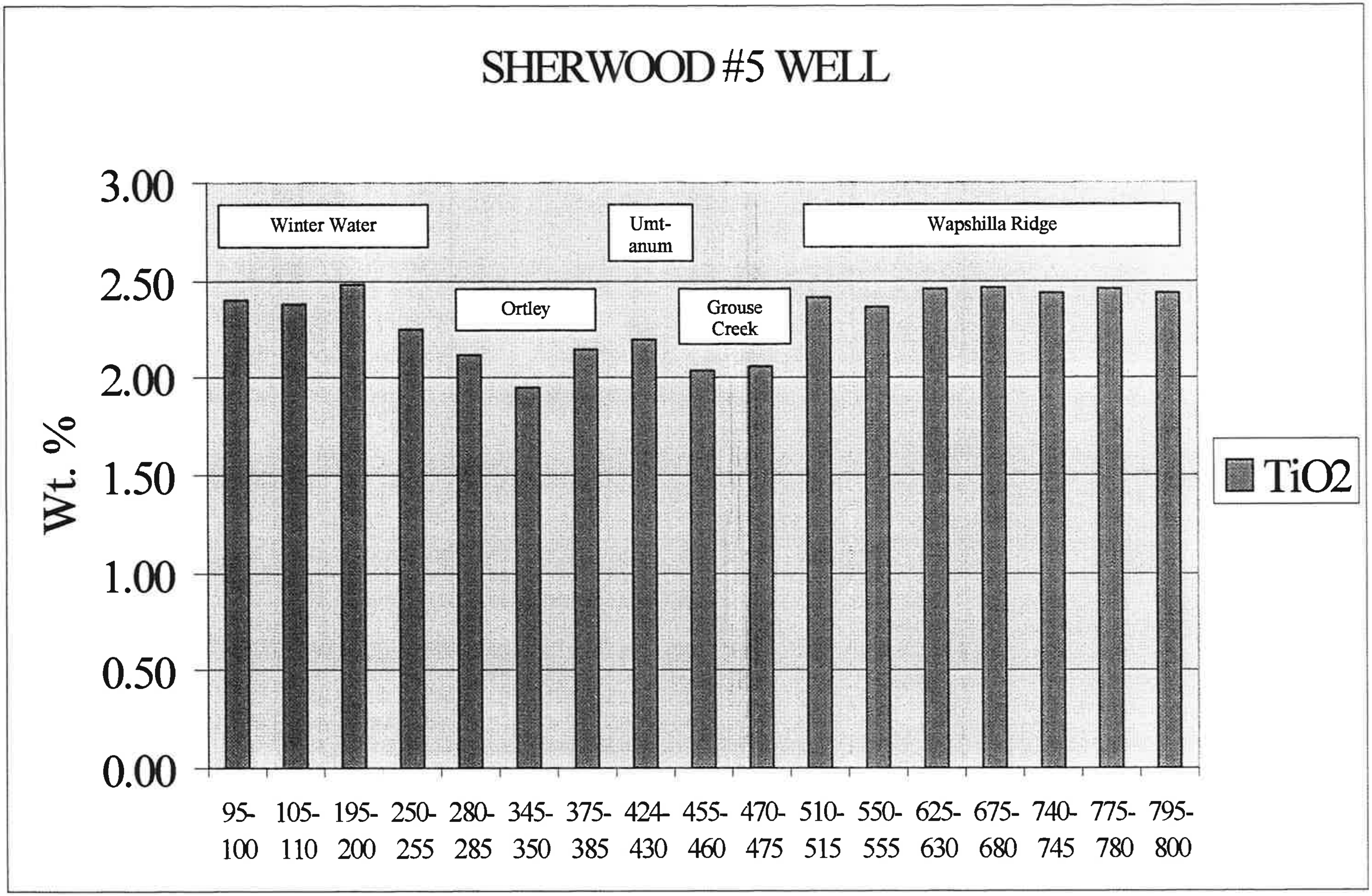
WSU GeoAnalytical Laboratory
(re-analysis, 1989)

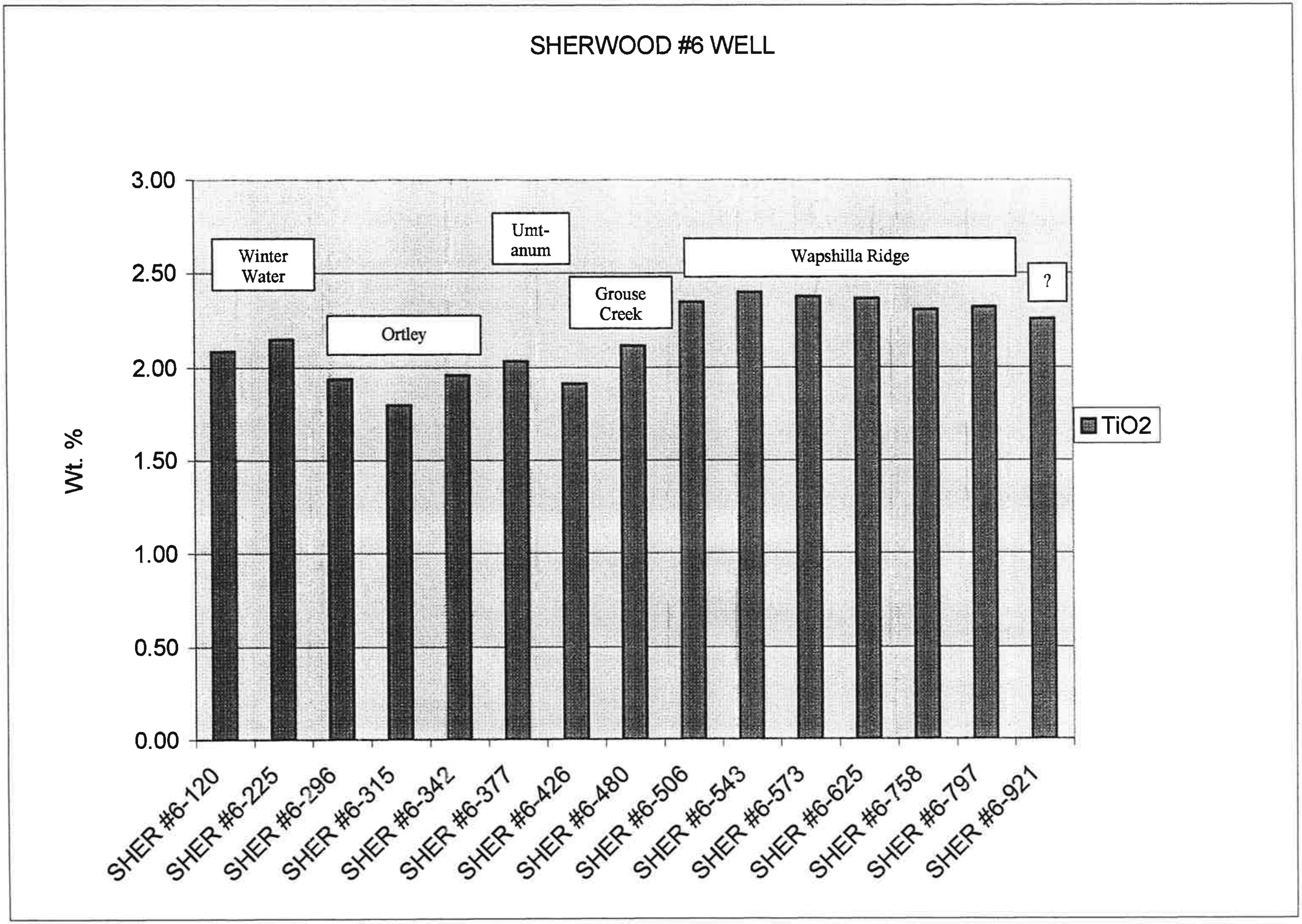
Cedar Creek well

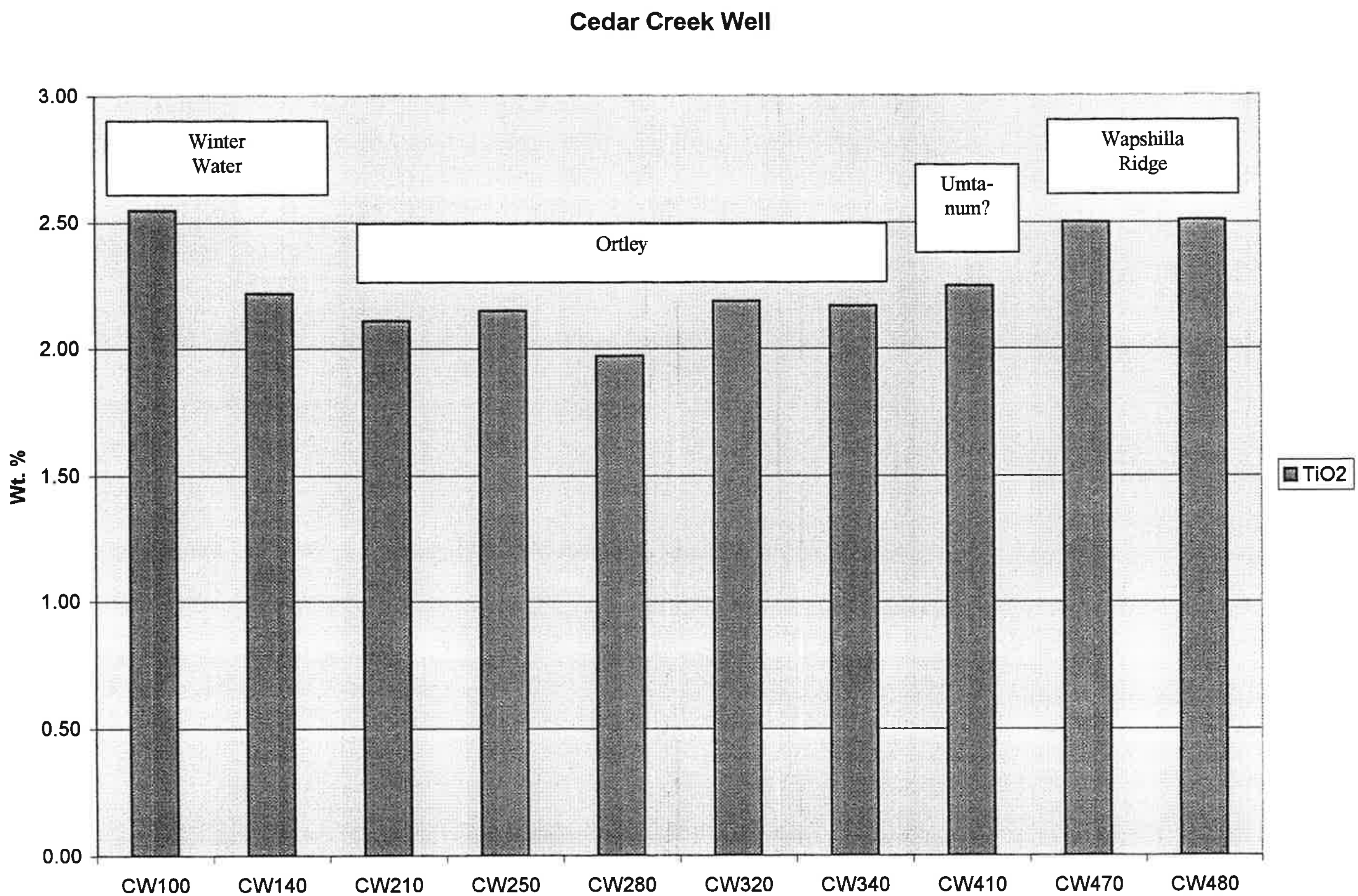
Weight Percent

SAMPLE	SiO2	Al203	TiO2	FeO	MnO	CaO	MgO	K2O	Na2O	P2O5	*TOTAL*	*FLOW ID*
CW100	56.77	15.15	2.363	13.34	0.166	4.93	2.03	1.84	3.00	0.408	100.00	Winter Water
CW140	56.62	13.73	2.046	12.00	0.192	6.73	3.30	1.65	3.37	0.361	100.00	Winter Water
CW210	56.16	13.88	1.953	11.79	0.192	7.15	3.64	1.87	3.03	0.324	99.99	Ortley
CW250	56.19	14.09	1.976	11.43	0.191	7.17	3.62	1.91	3.07	0.342	99.99	Ortley
CW280	55.79	14.11	1.803	11.36	0.191	7.63	4.11	1.72	3.00	0.295	100.01	Ortley
CW320	55.29	14.01	2.053	12.00	0.199	7.42	3.84	1.66	3.17	0.362	100.00	Ortley
CW340												
CW410	55.99	13.61	2.095	12.35	0.194	6.85	3.35	1.86	3.34	0.362	100.00	Umtanum?
CW470	54.84	14.25	2.358	12.64	0.190	7.23	3.58	1.78	2.78	0.355	100.00	Wapshilla Ridge
CW480	54.24	14.47	2.391	12.94	0.187	7.29	3.62	1.51	3.00	0.341	99.99	Wapshilla Ridge

	PPM															
SAMPLE	**Ni**	**Cr**	**Sc**	**V**	**Ba**	**Rb**	**Sr**	**Zr**	**Y**	**Nb**	**Ga**	**Cu**	**Zn**	**Pb**	**La**	**Ce**	**Th**
CW100	0	13	33	354	721	57	279	198	37	15.7	25	3	140	11	20	45	7
CW140	0	13	33	338	620	47	310	176	37	14.0	23	2	131	11	12	49	5
CW210	6	18	31	310	631	49	318	174	34	13.9	23	14	124	12	29	57	7
CW250	10	30	32	336	656	49	314	172	36	13.0	21	11	122	13	23	50	7
CW280	7	28	32	314	579	44	301	156	33	10.4	22	17	118	11	27	35	5
CW320	8	27	38	347	644	43	314	170	38	13.1	21	17	123	9	40	30	5
CW340																	
CW410	9	21	34	364	692	51	304	178	39	14.3	23	16	123	11	15	56	7
CW470	8	27	34	379	640	48	316	189	38	14.4	22	13	133	9	40	45	9
CW480	7	28	37	380	650	45	323	191	39	15.6	22	13	138	12	0	50	5


SHERWOOD #5 WELL
Wt. %
3.00
2.50
2.00
1.50
1.00
0.50
0.00
Winter Water
Ortley
Umt-anum
Grouse Creek
Wapshilla Ridge
TiO2
95-100
105-110
195-200
250-255
280-285
345-350
375-385
424-430
455-460
470-475
510-515
550-555
625-630
675-680
740-745
775-780
795-800


SHERWOOD #6 WELL
3.00
2.50
2.00
1.50
1.00
0.50
0.00
Wt. %
Winter Water
Ortley
Umt-anum
Grouse Creek
Wapshilla Ridge
?
TiO2
SHER #6-120
SHER #6-225
SHER #6-296
SHER #6-315
SHER #6-342
SHER #6-377
SHER #6-426
SHER #6-480
SHER #6-506
SHER #6-543
SHER #6-573
SHER #6-625
SHER #6-758
SHER #6-797
SHER #6-921


Cedar Creek Well
3.00
2.50
2.00
1.50
1.00
0.50
0.00
Wt. %
Winter Water
Ortley
Umta-num?
Wapshilla Ridge
TiO2
CW100
CW140
CW210
CW250
CW280
CW320
CW340
CW410
CW470
CW480

SQUIER ASSOCIATES