Tri City Water & Sanitary Authority

OREGON WATER RESOURCES DEPARTMENT

2020 SOLICITATION

WATER PROJECT GRANTS AND LOANS

GRANT APPLICATION

APPLICATION ATTACHMENTS

OREGON WATER RESOURCES DEPARTMENT

2020 SOLICITATION

WATER PROJECT GRANTS AND LOANS

GRANT APPLICATION

ATTACHMENT # 1

SITE MAP

Angus Lane Water

Storage Tank

PLAN VIEW - SITE PLAN (1/SITE)

SCALE: 1" = 150'

PRELIMINARY – NOT FOR CONSTRUCTION

REVISED	DESCRIPTION	SUBMIT.	APPR'D.	DATE

DESIGNED:	STM
DRAWN:	STM
CHECKED:	PW
APPROVED:	

midEa
8295 NW WYNOOCHEE DRIVE
CORVALLIS, OR 97330
TELEPHONE: (541) 404-3729

LINE IS 1 INCH AT FULL SCALE
IF NOT 1-INCH - SCALE ACCORDINGLY

TRI CITY, MYRTLE CREEK, WATER SYSTEM IMPROVEMENTS
TRI CITY WATER & SANITARY AUTHORITY

SITE MAP
ANGUS LANE WATER STORAGE TANK

PROJECT NO.	DRAWING NO.
TC - 20-03	C1
DATE	SHEET NO.
MAY 2020	1 OF 1

OREGON
WATER RESOURCES DEPARTMENT

2020 SOLICITATION

WATER PROJECT GRANTS AND LOANS

GRANT APPLICATION

ATTACHMENT # 2

PROPERTY ACCESS AUTHORIZATION

OREGON
WATER RESOURCES DEPARTMENT

Water Project Grants and Loans

Landowner Agreement

Instructions to Applicants: Work with landowners to complete this form for all properties on which the proposed project would occur. Submit this completed form as part of your grant/loan application. For questions contact WRD_DL_waterprojects@oregon.gov.

Project and Applicant Information

Project Name: Tri City, Myrtle Creek, Angus Land High-Level Above Ground Storage Project

Funding Applicant: Tri City Water & Sanitary Authority Co-Applicant (if applicable): ______

Funding Applicant Contact Information:
Name: Paul Wilborn
Phone Number: (541)863-5276
Email Address: Paul Wilborn

Co-Applicant Contact Information:
Name: ______
Phone Number: ______
Email Address: ______

Landowner Information

Landowner(s) Name: Bruce Allen & Roberta Faye Moore

Landowner Authorized Representative: ______

Landowner Contact Information (or Authorized Representative)
Address: 292 Angus Ln
(required) Myrtle Creek, OR 97457

(optional) Phone Number: ______
(optional) Email Address: ______

Property Information

List each property owned by the above-mentioned Landowner on which the project would occur:

County	Tax map	Lot number
Douglas	T30S R05W S08B	600

Landowner Acknowledgement

1. Bruce and Roberta Moore are the legal owner(s) (the Landowner) of the above described property (the Property).
2. I am authorized to act on behalf of the Landowner.
3. I am aware of and agree to the above-mentioned proposed project and grant permission for the Applicant, and the Applicant's agents, to conduct the following activities on the Property. (List activities below)
   a. Improve road to construct an above ground water storage tank and install 8" waterline to tie-in point
   b. Grant access for construction and geotechnical studies
   c. Allow construction of above ground storage tank and 8" waterline
   d. Permanent rights to access and maintain facilities after construction is completed
4. I am aware that monitoring information related to the Project is a matter of public record.
5. I certify that the above-mentioned information is true and accurate, I am aware of and agree to the proposed work, and I am authorized to sign as the Landowner or Authorized Representative.

Signature of Landowner or Authorized Representative: Bruce Allen Moore Roberta F Moore Date: 5/14/20

6. Print Name Bruce Allen Moore, Roberta Faye Moore

OREGON
WATER RESOURCES DEPARTMENT

Water Project Grants and Loans

Landowner Agreement

Instructions to Applicants: Work with landowners to complete this form for all properties on which the proposed project would occur. Submit this completed form as part of your grant/loan application. For questions contact WRD_DL_waterprojects@oregon.gov.

Project and Applicant Information

Project Name: Tri City, Myrtle Creek, Angus Land High-Level Above Ground Storage Project

Funding Applicant: Tri City Water & Sanitary Authority Co-Applicant (if applicable): ____

Funding Applicant Contact Information:
Name: Paul Wilborn
Phone Number: (541)863-5276
Email Address: Paul Wilborn

Co-Applicant Contact Information:
Name: ____
Phone Number: ____
Email Address: ____

Landowner Information

Landowner(s) Name: Betty E. Lawton Living Family Trust

Landowner Authorized Representation Gary and Sandy Lawton

Landowner Contact Information (or Authorized Representative)
Address: 1118 Flournoy Valley Road
(required) Roseburg, OR 97471

(optional) Phone Number: ____
(optional) Email Address: ____

Property Information

List each property owned by the above-mentioned Landowner on which the project would occur:

County	Tax map	Lot number
Douglas	T30S R05W S08B	506

Landowner Acknowledgement

1. Insert Landowner's Name is/are the legal owner(s) (the Landowner) of the above described property (the Property).
2. I am authorized to act on behalf of the Landowner.
3. I am aware of and agree to the above-mentioned proposed project and grant permission for the Applicant, and the Applicant's agents, to conduct the following activities on the Property. (List activities below)
   a. Improve road to construct an above ground storage tank and install 8" waterline to tie-in point
   b. Permanent rights to access and maintain facilities during construction and after construction is completed
   c. Allow construction of 8" waterline
   d. ATTACHED CONSTRUCTION REQUIREMENTS

   10 ITEMS TOTAL
4. I am aware that monitoring information related to the Project is a matter of public record.
5. I certify that the above-mentioned information is true and accurate, I am aware of and agree to the proposed work, and I am authorized to sign as the Landowner or Authorized Representative.

Signature of Landowner or Authorized Representative: [signature]
Date 5-27-20 Print Name: GARY LAWTON

Gary Lawton
1118 Flournoy Valley Rd
Roseburg, OR 97471
05/27/2020

Paul Wilborn
General Manager
Tri City Water & Sanitary Authority
215 N Old Pacific Hwy
Myrtle Creek, OR 97457

Dear Paul Wilborn:

Please see following waterline easement requirements;

1. General contractor and any subcontractor shall make available to land owner of Angus Lane Certificates of Insurance in amounts as required for Oregon Public Works projects.
2. No excess delays to land owner or people residing on Angus Land for their traveling egress shall be allowed. The general contractor will coordinate and communicate with land owners and accommodate their schedules.
3. Depending of time of year, excess mud or road slime will be removed and hauled away and replaced with one inch minus state approved gravel. If project creates dust for summer schedule, then water shall be used.
4. All excavated materials to be hauled away or dumped in narrow road areas if land owner requests.
5. All ditch trenches to be filled to minimum of 95% compaction with state approved ¾" or 1" minus.
6. Private testing services to field test all compaction.
7. Following project, Angus Lane is to be top coated with 1" minus state approved gravel; 3" thick and 14' wide minimum. Ditches are to be created on bank side, shoulders are to be tapered and feathered in.
8. All new hookups for water meters will have no charge to landowners affected. In addition, existing sewer drains, if interfere with project, will be modified or moved at no expense to affected landowner.
9. Tri City Water & Sanitary Authority will repair any sinkholes or bad road areas which appear as a result of improper compaction. This serves as a warranty after contractor is finished and time has passed. This warranty of obligation shall be in effect for 3 full winters after project is completed.
10. Contractor is to be notified and explained the private road status.

Sincerely,
Gary Lawton

Gary Lawton

OREGON WATER RESOURCES DEPARTMENT

2020 SOLICITATION

WATER PROJECT GRANTS AND LOANS

GRANT APPLICATION

ATTACHMENT # 3

DOCUMENTATION OF MATCHING FUNDS AUTHORIZATION

Tri City Water and Sanitary Authority

Regular Board Meeting

May 13, 2020

The Board of Directors of Tri City Water & Sanitary Authority met on Wednesday, May 13, 2020.

Pledge of Allegiance

Roll Call:

The following board members were present Lillian Elder, Chris DeWald, Bruce Stimpson and Carl White.

Diana Phillips was absent

Employees Present:

Paul Wilborn and Brooke Rainwater.

The meeting was called to order by Bruce at 6:30pm.

Minutes Approval:

Lillian made a motion to accept the March 11, 2020 Board Meeting Minutes. Chris seconded the motion. All in favor. Motion carried.

Wastewater Treatment Plant:

Steve Ledbetter was absent. Paul went over Steve's report with the board. The report will be a permanent part of the minutes on file.

Visitors:

Manager Report:

- **Financial Report** – Paul reported to the board that we are 83% of the way through the fiscal year and our revenues are above projections. We have worked hard this year to keep expenses down and that is reflected in the financial report as well. Office expenses are about 82% and plant expenses are close to 70% of projected.
- **Henry Street project update** – Paul stated that he and Sean have been working on getting good estimates for this project. They have both talked to several contractors, getting ideas how to tackle this project. It is a difficult project because of the depth of the proposed sewerline, but we should have a good estimate by the end of the week. After getting an estimate, we will contact the Industrial Board and see if we can get a grant from them.
- **Angus Lane Water Reservoir - Authorization of up to $550,000 in match funding**. – Paul informed the board that the application for a grant from the Oregon Water Resource Department

is due at the end of the week. We have spent a lot of time working through the application and are in the process of getting Letters of Support from the community and Landowner Agreements to accompany the grant. We also need approval from the Board to spend up to $550,000 in matching funds (25%) if we receive the grant. Chris made a motion that we spend $550,000 in matching funds (25%) if we receive the grant. Carl seconded the motion. All in favor. Motion carried.

- **District Mapping Update** – Paul reported that we have a new intern working on the project. The last one didn't work out very well. We now have a young man, Justin Propps that spends a few hours a week here working on the project. He seems to be making good progress.
- **Meter Reading Discrepancies** – Paul stated that we found 14 meters that were in the district that were reading at about 180%. We sent one back to Sensus and they determined that the meter was a ⅝ X ¾ meter with a ¾ register. That was the cause of the inaccuracy. All of the meters but one has been installed in the original location since installation and most have the same property owners. Paul wanted to know if we should issue refund checks, credit accounts, or give the customers a choice. The board discussed the options and all were in agreement that we should issue the customers refunds checks to make the process as clean as possible. Paul stated that he will make an attempt to contact the affected parties this week by phone if possible.
- **Audit Contract** – Paul stated that the Audit Contact is available for review and approval. The maximum fee for their services is $16,400, up from $15,900 last year. Carl made a motion that we accept the audit contract. Chris seconded the motion. All in favor. Motion carried.
- **Health Insurance Renewal** – Paul reported that our health insurance renewal through Special Districts was considerable higher than last year's rates. After shopping around, we found rates that were substantially less expensive that through SDAO but still through the same provider. The rates for our employees were $2,196 less than Special District renewal rates. We will be leaving Special Districts Insurance Services and going with the Regence stand-alone policy.

Operations:

- **Brian Kelly**: Brian was absent. Paul reported to the board that the Ozone is up and running and that he thinks it will be very valuable this year due to the drought-like conditions.
- **Cody Hammond:** Cody was absent. Paul reported that they have been having a lot of locates lately and that the team is working really hard to keep things running well.

Board Discussion:

- **Covid-19 relief –** Paul reported that we haven't noticed an impact from the Covid-19, but haven't done non-pay shut offs since February. We are still sending out delinquent notices and following up with phone calls. Paul wondered if the board had any thoughts or suggestions as to what we could/should do differently. After board discussion, the board would like us to start shut offs again in June.
- **COLA** – Paul asked for recommendations from Board for COLA. Chris made a motion that we give the employees 2.5% COLA. Carl seconded the motion. All in favor. Motion Carried.

Old Business:

- **Lift Station #7 Pumps –** Chris inquired if the replacement pump controls were installed at Lift Station #7. Paul stated that we are still waiting on a couple more parts and then he will be installing them.

New Business:

- **Charlie Swan** – Paul reported to the board that Charlie Swan (The owner of the property at Knoll Terrace) thinks our SDC fees are too high, which led into the discussion regarding the Wastewater SDC revision.
- **Wastewater SDC revision –** Paul stated to the board that we have had several inquiries regarding SDC fee's in the last couple of weeks. He also stated that while doing rate changes in the system that we have come across that a lot of accounts don't follow our current resolutions for SDC fees. Paul requested that he ask Sean Moran to reevaluate our current SDC Resolutions and update them to eliminate any inaccuracies. Carl made the motion to allow Paul to have Sean reevaluate our resolutions. Lillian seconded the motion.

Items not on Agenda:

Review Monthly Bills: Lillian made a motion to review and pay the bills. Chris seconded the motion. All in favor. Motion carried.

Adjournment: The meeting was adjourned at 7:21pm

Attested to by: Brooke Rainwater

2020 SOLICITATION

WATER PROJECT GRANTS AND LOANS

GRANT APPLICATION

ATTACHMENT # 4

PROJECT FEASIBILTY DOCUMENTATION

Applied
Geotechnical
Engineering
& Geologic Consulting

April 11, 2020

Tri-City Water & Sanitary Authority
215 N Old Pacific Highway
Myrtle Creek, OR 97457

SUBJECT: PRELIMINARY GEOTECHNICAL EVALUATION, PROPOSED NEW RESERVOIR SITING, TRI-CITY WATER & SANITARY AUTHORITY, MYRTLE CREEK, OREGON

At your request, Applied Geotechnical Engineering and Geologic Consulting LLC (AGEGC) has conducted a preliminary geotechnical evaluation for the siting of your proposed new water reservoir. The proposed reservoir location is east (uphill) of Angus Lane in Tri-City, Oregon. Our preliminary work included a review of geologic information for the site and vicinity, a ground-level reconnaissance of the proposed site, and engineering analyses. This report summarizes our work and provides our conclusions and recommendations for the suitability of the proposed site and recommendations for additional geotechnical work for design of the reservoir.

SITE DESCRIPTION

The proposed reservoir site is located on the hillsides east of Tri-City. A senior geotechnical engineer/geologist completed a site visit to the property with you. The site is moderately sloped down to the west. The site is undeveloped and is wooded with oak trees and scattered areas of grass vegetation. A deep ravine is located south of the site. The ravine has steep side-slopes. We did not observe any large-scale, deep-seated slope instability along the ravine. Minor sloughing was observed in the cut and fill slopes along the access road to the site.

PROJECT DESCRIPTION

We understand that the new reservoir will be a steel tank. The site is located on a moderate steep slope. Some cut will be required to level the building pad. Typically, steel reservoirs are founded on ring foundations to support the exterior wall of the tank.

GEOLOGY

Based on our experience in this area and our observations at the site, the site is underlain by sedimentary rock. The rock is locally weathered to a stiff silt soil. The stroke and dip of the sedimentary rock could not be determined in the field. A deep ravine located south of the site has steep side-slopes, indicating the weathered sedimentary rock is relatively strong. The hillside slope at the site is moderately steep and uniform with no indications of deep-seated slope instability. Groundwater seepage or springs were not observed at the proposed site. We anticipate that perched groundwater conditions occur at the top of the weathered rock during periods of heavy and/or continuous rainfall. There are no mapped faults near the site.

PRELIMINARY CONCLUSIONS AND RECOMMENDATIONS

Based on our observations and the site and our discussions with you, in our opinion, the site is suitable for development with the proposed steel reservoir. The main geotechnical and geologic concerns at the site include local zones of deep surficial silt soils, moderately steep slopes and seasonal perched groundwater conditions. These may impact the cost of construction of the reservoir, but do not preclude development of the site.

The following contains our recommended scope of work and budget for additional geotechnical work needed for design of the reservoir.

SCOPE OF WORK

Our proposal is based on our understanding of geologic conditions at the site, our knowledge of the project, and our experience with similar projects.

Our proposed scope of work includes the following items.

1) After the tank location is finalized, a second ground-level reconnaissance will be completed by a licensed geotechnical engineer/geologist to stake potential boring locations for one-call utility locates and to show locations that will require some grading for access of the drill rig.

2) Subsurface conditions for the proposed reservoir will be evaluated with three geotechnical borings completed to a depth of about 50 ft using a track-mounted drill rig. If hard rock is encountered, a minimum of 15 ft of rock will be cored. The locations of the borings will be estimated in the field using existing site conditions and landmarks. Representative soil samples will be obtained from the boring. A licensed geotechnical engineer/geologist, provided by our firm, will log the boring and obtain representative samples of the soils encountered. The spoils from the borings will be left on-site.

3) A site-specific seismic hazard study will be completed based on the subsurface conditions beneath the site and our estimate of potential regional and local seismic activity that might affect the site. Our report will include a full description of the regional seismic environment, the results of our literature review, and the expected ground response (soil classification and peak ground accelerations) and will contain conclusions and recommendations relating to the project. In addition, the report will contain an evaluation of the earthquake-induced landslide, liquefaction, settlement, fault rupture, and tsunami hazards at the site, including the effects of local geology and topography.

4) Laboratory tests will be conducted to provide data on the important physical characteristics of the subsoils, essential for engineering studies and analyses. The

laboratory tests may include standard classification tests, such as natural water content, dry density, and grain-size distribution.

5) Engineering studies and analyses will be accomplished that will lead to the preparation of conclusions and recommendations concerning (1) site grading including cut and fill slopes; (2) types of foundations; (3) allowable bearing pressures; (4) bearing strata; (5) estimated settlements (total and differential); (6) seismic design criteria; and (7) other design or construction considerations that may arise during the course of the study.

6) A report will be prepared discussing the work accomplished and presenting the results of the various tests and office studies. An electronic (PDF) copy of the report will be provided for your use and distribution.

FEE

The fee for the above work will be computed on a time and material basis. The total estimated costs for the investigation are summarized below.

1) Field Program and Review of Information	
Drilling Subcontractor	$ 9,800
AGEGC Labor and Equipment	$ 2,800
2) Laboratory Testing	$ 500
3) Engineering and Report	$ 1,600
4) Seismic Study	$ 1,900
Total :	$ 16,600

Sincerely,

Applied Geotechnical Engineering and Geologic Consulting, LLC

Robin L. Warren, P.E., G.E., R.G.
Principal

REGISTERED PROFESSIONAL ENGINEER
15045PE
OREGON
JULY 25, 1990
ROBIN L. WARREN

Renewal: June 2020

TRI-CITY
JOINT WATER & SANITARY AUTHORITY

DOUGLAS COUNTY, OREGON

Water System Master Plan

May 2006

HBH
Consulting Engineers

Tri-City
Joint Water & Sanitary Authority
Douglas County, Oregon

Water System Master Plan

May 2006

REGISTERED PROFESSIONAL ENGINEER
17,804
OREGON
JULY 25, 1995
ROBERT E. BYNUM
RENEWAL DATE: 12/31/2007

HBH
Consulting Engineers

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Executive Summary

Section ES

ES.1 Background

The Tri-City Joint Water & Sanitary Authority is located in southern Douglas County Oregon approximately 20 miles south of Roseburg along Interstate Highway 5 and the South Umpqua River. Tri-City is an unincorporated urban area immediately south of the City of Myrtle Creek. The Authority owns, operates, and maintains a community water system identified by Public Water System ID number OR4100549. The water system includes a river intake on the South Umpqua River, a conventional water treatment plant, 4 finished water storage tanks, 2 distribution pump stations, and approximately 30 miles of piping.

Rapid new growth, storage deficiency concerns, river intake problems, and treatment plant run times approaching maximum in summer months lead to the need for an updated Water System Master Plan. The Master Plan planning period is 20 years ending in 2025.

ES.2 Population, EDUs, and Growth

ES.2.1 Current Population and EDUs

The 2000 Census Data for the Tri-City Census Designated Place (CDP) lists a population of 3519 with 1348 occupied homes (1409 total homes) and an average of 2.6 people per household.

Based on Tri-City service records there are 1340 single-family dwellings, 50 apartment units, and 204 mobile homes currently being served by the water system. An average of 2.6 people per single-family dwellings is used. For apartments and mobile homes, the persons/unit figure is 1.38 and 2.48 persons per unit respectively.

Based on water sales records (9/2004 to 8/2005), the average water use per single-family dwelling is 7075 gallons per month. The 1340 single-family dwellings are equal to 1340 EDUs. The 50 apartment units are equal to 26.5 EDUs, and the 204 mobile homes are equal to 194.7 EDUs. The total number of residential EDUs is therefore 1561. Using the Census Data figure of 2.6 people per EDU results in a current estimated residential population of 4059 persons. This estimate is believed to be more accurate than the Census Data and a current population of 4059 persons is assumed in this Plan.

Based the same 12-month water sales period as above, the non-residential water users (commercial, public, industrial, schools) used an amount of water equivalent to 170 single-family dwellings (170 EDU). The total system EDU is therefore 1731. The equivalent service population is 4501.

ES.2.2 Projected Population and EDUs

The Tri-City Board has adopted a 2.5% average annual growth rate. Census Data from 1970 to 2000 shows an average growth of 5% however the rate slowed to approximately 1% over the last decade. Recent development has been significantly greater than in previous years and the potential for additional growth is great. The Myrtle Creek Comprehensive Plan indicates between 1.3 and 2.3% growth in Tri-City. Past planning efforts in Tri-City have used 2.5%. For the recent Tri-City/Myrtle Creek sewer treatment plant, a growth of 2.5% was adopted.

Based on a 2.5% average annual growth rate, Tri-City population will increase from 4059 to 6651 over a 20-year period. The number of EDUs is projected to increase from 1731 to 2837. To accommodate this level of growth, approximately 1000 new homes are required over a 20-year period with an average of 2.6 people per household. With even growth in commercial and industrial use, the commercial EDUs are projected to increase from 101 to 166 and the industrial EDUs are projected to increase from 19 to 31 EDUs.

ES.3 Water Demand

ES.3.1 Past and Current Water Demand

Plant records from January 2001 through August 2005 were used to determine the current water demand values. The average daily demand (ADD) is 540,000 gallons per day (gpd) or 133 gallons per person per day (gpcd). During summer peak days, the maximum daily demand (MDD) is 1,405,000 gpd or 346 gpcd. Approximately 200 million gallons of water per year is required by the system users.

Year	AAD (gallons)	Max Month (gal.)	MMD (gpd)	ADD (gpd)	MDD (gpd)	P.F. MDD/ADD	P.F. MMD/ADD	Days Over 1 MGD
2001	205,583,800	23,238,200	749,619	563,243	1,302,900	2.31	1.33	1
2002	183,322,900	25,123,300	810,429	502,255	1,164,100	2.32	1.61	3
2003	195,308,600	26,891,300	867,461	535,092	1,361,800	2.54	1.62	55
2004	197,397,000	26,702,100	861,358	539,336	1,484,100	2.75	1.60	39
2005*		25,406,300	819,558	508,945	1,458,720	2.87	1.61	22
Average	*195,403,075*	*25,472,240*	*821,685*	*534,981*	*1,354,324*	*2.56*	*1.55*	

* January through August

Current Design Values, End of Year 2005									
AAD	197,100,000	gal. per year							
MMD	26,815,000	gal. per month							
MMD	865,000	gpd	601	gpm	213	gpcd	500	gpd/EDU
ADD	540,000	gpd	375	gpm	133	gpcd	312	gpd/EDU
MDD	1,405,000	gpd	976	gpm	346	gpcd	812	gpd/EDU
PHD	2,700,000	gpd	1,875	gpm	665	gpcd	1,560	gpd/EDU
MDD P.F.	2.60							
MMD P.F.	1.60							
PHD P.F.	5.00							

About 9% of the water produced is used at the plant for backwashing the filters and other plant needs. An average of 16% is unaccounted for and is lost due to leaks, inaccurate meters, system flushing, and other unmetered use. 90% of the water sold goes to residential customers.

ES.3.2 Projected Water Demand

The projected population and EDU figures are multiplied by the current design values in order to estimate future water needs. The average demand is expected to increase to 885,000 gpd and the maximum day up to 2.3 million gallons per day. Since the treatment plant cannot run 24 hours per day, a plant capacity of 2.5 mgd is needed for the planning period.

Year	Population	EDU	ADD (gpd)	MMD (gpd)	MDD (gpd)	PHD (gpd)
2005	4,059	1,731	540,000	865,000	1,405,000	2,700,000
2006	4,160	1,774	553,500	886,625	1,440,125	2,767,500
2007	4,264	1,819	567,338	908,791	1,476,128	2,836,688
2008	4,371	1,864	581,521	931,510	1,513,031	2,907,605
2009	4,480	1,911	596,059	954,798	1,550,857	2,980,295
2010	4,592	1,958	610,960	978,668	1,589,629	3,054,802
2011	4,707	2,007	626,234	1,003,135	1,629,369	3,131,172
2012	4,825	2,058	641,890	1,028,213	1,670,103	3,209,452
2013	4,945	2,109	657,938	1,053,919	1,711,856	3,289,688
2014	5,069	2,162	674,386	1,080,266	1,754,652	3,371,930
2015	5,196	2,216	691,246	1,107,273	1,798,519	3,456,228
2016	5,326	2,271	708,527	1,134,955	1,843,482	3,542,634
2017	5,459	2,328	726,240	1,163,329	1,889,569	3,631,200
2018	5,595	2,386	744,396	1,192,412	1,936,808	3,721,980
2019	5,735	2,446	763,006	1,222,222	1,985,228	3,815,029
2020	5,879	2,507	782,081	1,252,778	2,034,859	3,910,405
2021	6,026	2,570	801,633	1,284,097	2,085,730	4,008,165
2022	6,176	2,634	821,674	1,316,200	2,137,874	4,108,369
2023	6,331	2,700	842,216	1,349,105	2,191,320	4,211,079
2024	6,489	2,767	863,271	1,382,832	2,246,104	4,316,356
2025	**6,651**	**2,836**	**884,853**	**1,417,403**	**2,302,256**	**4,424,264**

ES.4 Improvement Needs

ES.4.1 Water Supply

Tri-City has 6 different water use permits allowing withdrawal of surface water from the South Umpqua River. The total maximum withdrawal is limited to 4.87 cubic feet per second (cfs) or 2185 gpm. At this point, only two of the permits have been certificated. The Authority should continue efforts to file a Claim of Beneficial Use for each non-certificated permit as the ability to properly treat the water is realized.

During some low-river flow years the Water Master restricts use to the pre-1958 water rights in order to protect the instream water rights held by the State. Tri-City's pre-1958 water rights total only 1.445 cfs or 648 gpm. This flow is insufficient to meet the community's needs. During these times, Tri-City must also use water from the Galesville Reservoir. Tri-City holds a Storage Use Contract which allows up to 95 acre-feet (30 million gallons) to be taken from the Galesville Reservoir. With summer water demand sometimes exceeding 25 million gallons per month, the length that a water restriction can be endured is limited. Under current conditions, the pre-1958 water rights plus the Galesville water is sufficient to make it through 60 to 90 days of a water restriction period.

As demand increases in the system the length of water restriction that can be endured is shortened. By the end of the planning period it is estimated that the current 95 acre-feet permitted will last only about 33 days. The water restriction period can last longer than 33 days and it is evident that additional water from Galesville may be required in the future.

Fortunately, the large majority of municipal water allowed from the Galesville Reservoir is currently unallocated. Tri-City should continue to renew the current 95 acre-feet permit as required. In addition, the allocation of water from the Reservoir should be monitored. If other municipalities begin to purchase the remaining water to a point where unallocated water is low, Tri-City should purchase additional water. Based on worst case conditions, an additional 110 acre-feet could be required in 20 years.

ES.5.1 Water Intake and Pump Station

The existing intake is limited in capacity during low-water conditions. Exact configuration of the intake in the under water sections is not known at this time however all evidence suggests a maximum capacity of 900 to 1200 gpm under extreme low water conditions. In addition the intake lacks proper fish screening equipment as required by law. One or more of the water use permits state that Tri-City must install fish screening equipment.

Several options for modifying the existing intake or constructing a new intake are evaluated in the Plan. The lowest cost feasible option is to modify the existing intake with stainless steel tee fish screens and add an air compressor/control building to allow cleaning of the screens. Additionally, a portable emergency pump is recommended that can supply the plant with water in the event of an intake failure or malfunction. This improvement is needed immediately with construction scheduled for the summer of 2007. The time permitted for instream work is July 1 through August 31. A permit will be needed from the Division of State Lands and the design must be meet ODFW requirements. A $75,000 grant can potentially be obtained from the ODFW Cost Share Grant Program. Preliminary design, permit application, and grant application should occur in fall of 2006 in order to allow construction in the summer of 2007. Design should be completed by March or April with project bidding no later than May 2006. The estimated project cost for this improvement is $221,000.

The existing pump station near the intake is sufficient for near future needs. By 2010, both pumps may need to operate simultaneously to meet demand. To correct this deficiency and allow a lead/lag pump configuration for reliability, larger pumps will be required. According to Flgyt (pump manufacturer) the existing wetwell is large enough to allow installation of the required pumps which will each produce at least 1740 gpm. Modification of the wetwell and pump discharge piping will be required in order to fit the larger pumps. New variable frequency drives will also be required. The estimated cost for this project is $152,000.

ES.6.1 Water Treatment

A plant flow of 1740 gpm is needed for the planning period. The existing plant has been producing approximately 1000 gpm. Sedimentation basin proper design parameters will be exceeded at flows above 1150 gpm. The filters can handle flows of approximately 1430 gpm with normal conservative loading rates. Recent modifications with the plant discharge pump drives will allow flows above 1000 gpm. Since typical design parameters will be exceeded at higher flows, it is not yet known what flow the existing plant will be able to successfully treat. It is suspected that a flow of 1250 gpm will be easily achieved and flows as high as 1400 gpm may be possible. The length of time that plant capacity improvements are delayed is dependant on the flow that can be achieved by the operator.

ES.6.1.1 Disinfection System Improvements (Priority 1)

Before plant flows are increased, an immediate improvement to disinfection facilities are recommended. The existing chlorine gas equipment has reached the end of its expected design life and lacks required safety features. Chlorine gas can be fatal if inhaled. Personnel delivering chlorine have threatened to stop delivering to Tri-City due to the lack of safety provisions. The cost of safety provisions, including alarms, forced ventilation, and a gas scrubber would equal or exceed the cost of new alternative equipment. To eliminate the safety hazards associated with chlorine gas, it is recommended that Tri-City install equipment to generate a weak solution of sodium hypochlorite on site.

Improvements to increase chlorine contact time are also recommended. A previous tracer study showed that only 34 minutes of contact time was provided in the clearwells at a flow of 1040 gpm. A contact time of 30 minutes is required under expected worst case conditions. Even by increasing the minimum depth of water in the clearwell, violations could occur at higher flowrates. The newer clearwell constructed in 2000 has excellent baffling and efficiency however the original clearwell remains unbaffled and provides almost no contact time. It is recommended that baffling be added to the original clearwell.

The estimated project cost for recommended disinfection system improvements is $261,000. It is recommended that these disinfection system improvements be considered a high priority project since they should be completed before attempts at increasing plant flows are made. Ideally, this work would be constructed during the winter months of 2007.

ES.6.1.2 Backwash Pond Improvements (Priority 1)

The existing backwash pond is difficult to clean and does not allow proper settling of the backwash water. To improve the pond, a center dike or wall should be added to provide two cells. This will allow one side to be dewatered and cleaned while the other side is in service. Additionally, the discharge and outlet piping should be separated by the greatest distance possible to allow only clarified effluent to be discharged and sediment/sludge to remain in the pond. Estimated approximate budget is $65,000.

ES.6.1.3 Plant Capacity Improvements (Priority 2)

To allow the plant to reliably produce the 1740 gpm projected for the planning period, improvements to the treatment process are recommended. If the existing plant is capable of being pushed to 1400 gpm, the plant capacity improvements could possibly be delayed until 2015. If a flow of only 1250 gpm can be reliably treated, improvements will be needed in 2010. In either case, the Authority should begin planning now to ensure that revenue is being generated in anticipation of future improvements.

Alternatives include mixed-media granular filtration and membrane filtration equipment. If conventional granular filters are used, two additional gravity filters are recommended to avoid excessive loading rates. This will become more important in the future as the Stage 2 Long-Term Enhanced Surface Water Treatment Rule is enacted. Additionally, a second sedimentation basin would be recommended due to the fact that filter performance relies heavily on proper sedimentation upstream. Installation of additional gravity filters would be difficult since the plant layout does not facilitate expansion.

Membrane filters would eliminate the need for a second future sedimentation basin and would provide higher water quality making it easier to meet future treatment standards. In addition, future additional expansion beyond the planning period would be much easier to implement. Operator time required and chemical costs would also be reduced. Membrane filtration has a higher capital cost than conventional filtration but approximately equal present worth value when 20-years of operation and maintenance are included.

The project budget for the membrane filtration plant upgrade is $2.4 million at today’s prices. At the current rate of inflation in construction costs, the project would cost $2.8 million in 2010 and $3.5 million in 2015. The project will likely cost even more than these future estimates since the capacity would also be greater than planned for now to provide a 20-year design at the time of construction.

ES.5 Storage Needs

At current water demands in the community, Tri-City has an existing storage deficiency of 279,000 gallons. For the planning period an additional storage capacity of 1.4 million gallons is needed. The existing storage deficiency is primarily in the high-level service area served by the Valley Drive Pump Station and the Back Acres Storage Tank.

ES.5.1 High-Level Service Area Storage Need

With only 87,000 gallons of storage in the Back Acres Storage Tank, the high-level area is extremely deficient in storage. Storage requirements are the sum of equalization storage, emergency storage and fire storage. The existing storage tank provides only one-half of the recommended fire storage and no equalization or emergency storage.

A storage capacity of 430,000 gallons is recommended for the high-level service area. If the Back Acres Tank remains and a second tank is constructed, the new tank should have a capacity of 340,000 gallons. If the existing tank is replaced with a new tank, the new tank should have a capacity of 430,000 gallons.

Due to the steep slopes at suitable elevations, site alternatives are limited. A second tank at a new site is preferred to provide redundancy and allow continued service while one tank or the other is shut-down for cleaning or repairs. A second tank will require land acquisition, significant excavation, and lengthy piping runs to tie into the system. Removing the existing tank and constructing a new tank has a lower capital cost ($50,000 less) but will result in a period of virtually no storage during construction while a small temporary tank is used.

If land can be acquired for a new tank site then this option should be used and the project budget is $750,000. If land cannot be obtained, the existing tank site should be used and the budget is $700,000. The project cost is based on a glass-fused steel tank. If post-tensioned concrete is desired for added durability the cost would be significantly higher.

ES.5.2 Main-Level Service Area Storage Need

With 430,000 gallons of storage in the high-level area, the main service area needs an additional 1.1 million gallons storage tank for the planning period. If the tank construction is delayed until 2010, the size should be increased to 1.5 million gallons. Many potential locations are possible for this new tank. Seven potential sites are presented in the Plan. The average length of connecting piping is used to estimate the cost. The estimated budget cost for a 1.1 MG tank in 2005 dollars in $1.2 million.

ES.5.3 Need for Improvements at Existing Tanks

The 3 existing steel tanks will require painting during the planning period. The Walnut St. and Aker Drive tanks are in the worst condition and need exterior repainting as a Priority 1 improvement. If a suitable new tank site for a high-level tank cannot be obtained, the existing Back Acres Tank will need to be removed and as such should not be refurbished until the tank location is finalized.

The 3 existing steel tanks also do not have separate inlet and outlet pipes as now required in tank construction. As a result, very poor mixing is achieved in these tanks and low chlorine residuals occur. To provide at least a detectable chlorine residual in the tanks, the water levels must now be allowed to drop to approximately half full on a regular basis. During these times, storage in the system is extremely deficient. To improve mixing and water quality and to provide more efficient use of the storage changes at the tank piping are recommended. It is possible to add some minor piping and check valves inside the tanks at a relatively low cost. Alternatively, exterior piping and valve vaults could be added at a higher cost. The improvements in the Plan are based on the lower cost interior piping modifications.

ES.6 Distribution System Needs

Existing deficiencies include two large areas with inadequate fire protection, lack of fire protection on Ridgewood Place, lack of proper pipe looping, and several areas where additional fire hydrants are needed to provide coverage. A map showing the fire protection deficient areas is shown in Figure 7.5-1.

The south problem area is around Esther Ct, Arnold Ln., Celestial Ln., Matthews Ln., and Briggs Dr. Fire flows are half of the minimum needed flows due to the lack of looping and the large amount of 6-inch pipe. Options possible include replacing the main along Bills Road with larger piping or connecting the dead ends to provide looping. Connecting the east-side dead end pipes to provide looping has a significantly lower cost and is the recommended Priority 1 improvement. Easements will be required. With the east-side looping, minimum fire flows will be adequately provided. As a Priority 2 project, looping the west side of the dead-ends in this area is recommended.

The north problem area is just south of Walnut Street (Walnut Street Area) and includes the hydrants on portions of Cook St., Arburnia St., Mona St., Rollin Ct., Conrad St., Allan St., and Luke Ct. Priority 1 improvements estimated at $249,000 correct all deficiencies except at the higher elevations around Luke Ct. This involves upsizing 800 feet of piping on Arburnia, 570 feet on Cook St., and 375 feet on Clark and Mona St. The project will also eliminate 3 dead-end pipe sections.

As a Priority 2 project in this north area, the 12-inch piping running from Woodcrest to NE Donald Terrace can be extended to the intersection of Clark St. and Cook St. to eliminate existing 6-inch piping restrictions. To replace some 2-inch and 6-inch pipe and eliminate 2 dead-end runs, this new piping should be routed through easements along Fred Way to Allan St. and then up Allan St. to Clark St. To finalize the Priority 2 improvements in this area and solve the fire flow problem around Luke Ct., an additional 365 feet of 12-inch piping is needed from Allan St. to the fire hydrant on Luke Ct. These two piping projects are listed as “Walnut Street Area Priority 2 Piping”.

No fire hydrant exists at the cul-de-sac at Ridgewood Place and the existing 4-inch piping cannot support fire flows even if a hydrant was provided. To correct the complete lack of fire protection in this area, the existing 4-inch piping should be replaced with 8-inch and a fire hydrant placed near the end of the street. This is considered a Priority 1 improvement.

Various other lower priority piping improvements are suggested in the Plan (Priority 3) to replace undersized piping and potentially reduce leakage. These projects can be undertaken as the need arises due to future maintenance issues or development needs.

The last distribution system improvement discussed in the potential replacement of piping along Highway 99 due to future phases of Douglas County’s road widening project. These projects may be forced upon the Authority when the County begins this work. The time frame at which these projects may be needed is unknown. Phase 1 has already been completed which required piping replacement from just north of Chadwick to just north of Woodcrest Dr. Subsequent phases are shown to continue north to the Authority service boundary.

ES.7 Priority 1 Needs

Priority 1 Improvements - Immediate Need				
Item	**Description**	**Project Cost**	**% Capacity for Growth**	**Eligible SDC Cost**
1	Intake Screening Modifications	$185,735	43%	$79,601
2	Portable Emergency Supply Pump	$35,000	43%	$15,000
3	Plant Disinfection System Improvements	$260,625	43%	$111,696
4	Plant Backwash Pond Improvements	$65,000	43%	$27,857
5	High Level Storage, 0.43 MG Tank	$697,433	79%	$551,458
6	Walnut St. Tank Refurbishment	$44,758	0%	$0
7	Aker Dr. Tank Refurbishment	$43,420	0%	$0
8	Tank Mixing Improvements (Aker and Walnut)	$23,630	0%	$0
9	South Area Piping Improvements, Option B	$211,681	50%	$105,841
10	Walnut St. Area Priority 1 Piping	$248,945	50%	$124,473
11	Ridgwood Place Piping	$105,814	0%	$0
Total		**$1,922,040**		**$1,015,925**

Priority 1 projects address immediate needs and deficiencies. Upon completion, all fire flow problem areas requiring piping improvements will be corrected with the exception of Luke Court. Various other areas can be corrected by merely adding fire hydrants as available as shown in Figure 7.5-2. With completion of the Priority 1 plant and intake improvements (items 1-3), plant flows can be increased to allow a major plant upgrade project to be delayed. With the construction of additional storage in the high-level service area, the existing overall storage deficiency will be corrected and adequate storage should then exist in the high-level area. A larger tank should never be required in the future for the high-level system unless the boundary changes significantly.

USDA Rural Utilities Service (RUS) has stated that Tri-City could be given funding package with 20% grant and 80% loan at 4.375%. If all Priority 1 improvements were undertaken with this funding package, the annual loan payment would be $116,930 for 20 years. To provide a 10% cushion, a monthly revenue of $10,700 would be required. With 1731 current EDUs, the monthly cost per EDU would be $6.20.

If all Priority 1 project except the storage tank were undertaken the annual loan payment would be $74,500 with the same grant/loan package. A monthly revenue of $6,830 dollars or $3.95 per EDU would be required over that needed to cover current operational costs and debt payments.

The above costs per EDU do not take into account SDC funds that might be collected to help pay for debt. Overall, Priority 1 is 53% SDC eligible.

It is important to remember that if projects are delayed until future years, the amount budgeted should be increased to account for inflation and rising construction costs. The method discussed in Section 5.3.1 should be used as a rough estimate for future year costs. The sizing of improvements such as storage tanks, pumps, and treatment capacity should also be updated if delayed until future years so that any project undertaken is sized for a 20-year period beginning at the time of construction.

It is recommended that the Priority 1 project be undertaken immediately with a loan/grant package. User rates would then be used to pay the debt service and incoming SDCs used to reimburse the capital improvement fund. It appears that a new rate structure as proposed in Section 9.2.2 will be sufficient to fund all of the Priority 1 improvements with an average rate of $43 per EDU. Water sales revenue should increase by approximately $290,000 per year.

ES.8 Priority 2 Needs

Priority 2A Improvements - Needed in 2010				
Item	**Description**	**Project Cost**	**% Capacity for Growth**	**Eligible SDC Cost**
1	Raw Water Pump Replacement	$151,336	43%	$65,075
2	Membrane Filtration Plant Upgrade	$2,376,900	43%	$1,022,067
3	Main Level Storage, 1.1 MG Tank	$1,178,456	100%	$1,178,456
Total		**$3,706,692**		**$2,265,597**

Priority 2B Piping Improvements - Desired by 2010				
Item	**Description**	**Project Cost**	**% Capacity for Growth**	**Eligible SDC Cost**
1	South Area West Loop	$208,071	25%	$52,018
2	Walnut St. Area Priority 2 Piping	$321,503	75%	$241,127
Total		**$529,574**		**$293,145**

Priority 2 projects involve items needed for future deficiencies that will occur with growth except the Walnut Street Area Priority 2 Piping. This Priority 2 piping is needed to correct a current deficiency with fire flows at Luke Court. Due to the cost, this project is deferred to Priority 2.

Based upon the adopted growth projections, the additional storage tank will be needed by 2010. If the tank project is delayed until 2010, the size and cost should be reevaluated but will probably need to be around 1.5 million gallons in capacity. The storage tank is needed entirely to provide for growth and can be funded entirely with SDC money.

With recent changes in the water treatment plant equipment installed near the completion date of this Plan, it appears possible to push plant flows above that provided today (1000 gpm). Once the Priority 1 plant improvements are constructed, plant flows can be increased safely. The point at which successful treatment is no longer possible is not yet known however the sedimentation process will be overloaded at flows above 1150 gpm. It may be possible to increase treatment flows up to 1400 gpm with careful attention but this is uncertain. The flow that can be treated properly will dictate when the Priority 2 plant improvements are required. Replacement of the raw water pumps can be delayed until the plant improvements are required however it may be necessary to purchase one new pump identical to the Flygt pump recently purchased. It is unlikely that plant improvements can be delayed for the entire planning period and as such budget plans should be started now.

If a new rate structure results in capital improvement reserve funds quickly, it is recommended that the Priority 2B piping be initiated as funds are accumulated. If a rate structure is adopted which results in an average of $43 per month per EDU $150,000 to $175,000 per year could be available after payments for Priority 1 projects. This would allow the Priority 2B projects to be funded within 3 to 4 years. After this, all reserve funds should be saved for future remaining Priority 2 improvements.

ES.9 Priority 3 Needs

Priority 3 Piping Improvements - Desired by 2020				
Item	**Description**	**Project Cost**	**% Capacity for Growth**	**Eligible SDC Cost**
1	Back Acres Piping	$114,868	50%	$57,434
2	Peacock Lane Piping	$54,141	0%	$0
3	Taylor to Corwin Loop	$74,018	25%	$18,504
4	Taylor to Susan Loop	$108,999	75%	$81,749
5	Jack Court	$31,136	0%	$0
6	Carriage Place	$56,260	0%	$0
7	Irving Drive	$167,356	50%	$83,678
8	Klimback to Carte	$80,284	75%	$60,213
9	Old Pacific Hwy. Piping due to Road Widening	$900,000	50%	$450,000
10				
11				
Total		**$1,587,061**		**$751,579**

The Priority 3 projects can be undertaken as the need arises due to piping deterioration or development. The projects are desired for the planning period but could be delayed if funding is not available. The exception is the piping replacement that will be required associated with widening of Old Pacific Highway by Douglas County. When the County undertakes the subsequent phases of this work, Tri-City will be required to replace the piping.

Introduction

Section 1

1.1 Background and Need

The Tri-City Joint Water & Sanitary Authority is located in southern Douglas County Oregon approximately 20 miles south of Roseburg along Interstate Highway 5 and the South Umpqua River (see Figure 1-1 *"Location Map"*). Tri-City is an unincorporated urban area immediately south of the City of Myrtle Creek and located within the Urban Growth Boundary of the City of Myrtle Creek. The Tri-City urban area is defined in the Douglas County Comprehensive Plan as an area "irrevocably committed to urban uses". The area was previously served by the Tri-City Water District and the Tri-City Sanitary District. In efforts to reduce administrative costs and streamline operations, the Districts were combined in 2005 to form the Tri-City Joint Water & Sanitary Authority.

Today the water system of the "Authority" is classified as a "community water system" and is identified on the Oregon Department of Human Services, Drinking Water Program (DWP) public water system inventory by Public Water System (PWS) Identification Number OR4100549. The State currently lists the service population as 3500 persons with 1500 water service connections.

Past planning efforts for the Tri-City water system include "A Master Development Plan for Water System Improvements to 1995" completed in 1974, and a "Water System Master Plan" completed in 1994. Various improvements to the system have been completed over time including many of the improvements recommended in the 1994 Plan; however some of the improvements recommended have yet to be initiated.

Since completion of the last Master Plan in 1994, significant changes in Drinking Water Standards required by the Safe Drinking Water Act have occurred including the Long-Term Stage 1 Enhanced Surface Water Treatment Rule (LT1ESWTR), and the Stage 1 Disinfectants and Disinfection By-Products Rule (Stage 1 D/DBP). In addition, Stage 2 of both rules will occur within the planning period of this Plan. None of these requirements have been addressed in previous planning efforts. A new Water System Master Plan is needed to ensure the Authority is properly prepared for these rules. Further, it is now apparent that the current treatment capacity of the water treatment facility will be exceeded in a relatively short period of time. The 1994 recommendations and subsequent plant improvements will not suffice to the design life date of 2018 and additional improvements will be required which have not yet been addressed in past Plans.

Growth in the community has been slower than predicted in the last Plan however recent development has increased dramatically. It is estimated that the current population is approximately 1000 persons less than projected in the last Plan; even so, growth patterns have resulted in pipeline and pump station improvements being required that were not originally planned for. An update is needed on growth projections and patterns to ensure prudent and economical planning. Updated guidance is needed to ensure that public infrastructure improvements provided for development are consistent with the long-term community needs and do not create problems for the Authority.

Finally, system mapping currently available is incomplete and does not reflect all of the changes that have occurred in the system. Updated mapping and hydraulic modeling is needed for the current system along with new analysis reflecting updated growth patterns and updated recommendations for needed system improvements.

1.2 Study Objective

The purpose of this Water System Master Plan is to furnish the Tri-City Joint Water & Sanitary Authority with a comprehensive planning document that provides engineering assessment of system components and guidance for future planning and management of the water system over the next 20 years. This document satisfies the Oregon Drinking Water Program (DWP) requirements for water master plans. Principal Plan objectives include:

- Description and mapping of existing water system
- Evaluation of existing water system components
- Prediction of future water demands
- Evaluation of the capability of the existing system to meet future needs and regulations
- Recommendations for improvements needed to meet future needs and/or address deficiencies
- Discussion of financing options and impacts to water rates
- Description of water management and conservation measures

This Plan details infrastructure improvements required to maintain compliance with State and Federal standards as well as provide for anticipated growth. Capital improvements are presented as projects with estimated costs to allow the Authority to plan and budget as needed. Supporting technical documentation is included to aid in grant and loan funding applications and meet the requirements of the Oregon Economic and Community Development Department (OECDD), the Oregon Water Resource Department (WRD), the Rural Utilities Service (RUS), as well as the DWP.

1.3 Scope of Study

1.3.1 Planning Period

The planning period for this Water System Master Plan is 20 years, ending in the year 2026, in accordance with OAR 333-061-0060(5)(b). The period must be short enough for current users to benefit from system improvements, yet long enough to provide reserve capacity for future growth and increased demand. Existing residents should not pay an unfair portion for improvements sized for future growth, yet it is not economical to build improvements that will be undersized in a relatively short period of time. OAR 690-086-0170 suggests that demands be projected over 20 years, which is a typical planning period for water master plans.

1.3.2 Planning Area

The Master Plan planning area is that contained within the Authority Boundary. The area can be seen in Figure 1-2 *"Area Map"*.

1.3.3 Work Tasks

In compliance with Drinking Water Program and Water Resource Department plan elements and standards, this study provides descriptions, analyses, projections, and recommendations for the water system over the planning period. The following elements are included:

- Study area characteristics including land use and population trends and projections
- Description of the existing water system including supply, treatment, storage and distribution
- Existing regulatory environment including regulations, rules and plan requirements
- Current water usage quantities and allocations
- Projected water demands

- Existing system capacity analysis and evaluation
- Improvement alternatives and recommendations with associated costs
- Recommendations for water management planning and water usage curtailment
- A summary of recommendations with a Capital Improvement Plan
- Funding options
- Maps of the existing system and recommended improvements

Study Area

Section 2

2.1 Physical Environment

2.1.1 Location

The Tri-City Joint Water & Sanitary Authority is located in southern Douglas County Oregon approximately 20 miles south of Roseburg along Interstate Highway 5 and the South Umpqua River, and immediately south of the city of Myrtle Creek (see Figure 1-1 *"Location Map"*). The Tri-City urban area is defined in the Douglas County Comprehensive Plan as an area "irrevocably committed to urban uses".

The Tri-City JW&SA Boundary includes the town site along old Highway 99, the surrounding area to the east, and some industrial land west of I-5 at the south end of the boundary. The Authority Boundary encompasses an area of 1952 acres or 3.05 square miles. The entire water system is located within the Boundary with the exception two storage tanks located slightly outside the boundary to the east.

The town is in Township 29 and 30 South, Range 5 West, W.M. The Authority and surrounding area is shown in Figure 1-2 *"Area Map"*.

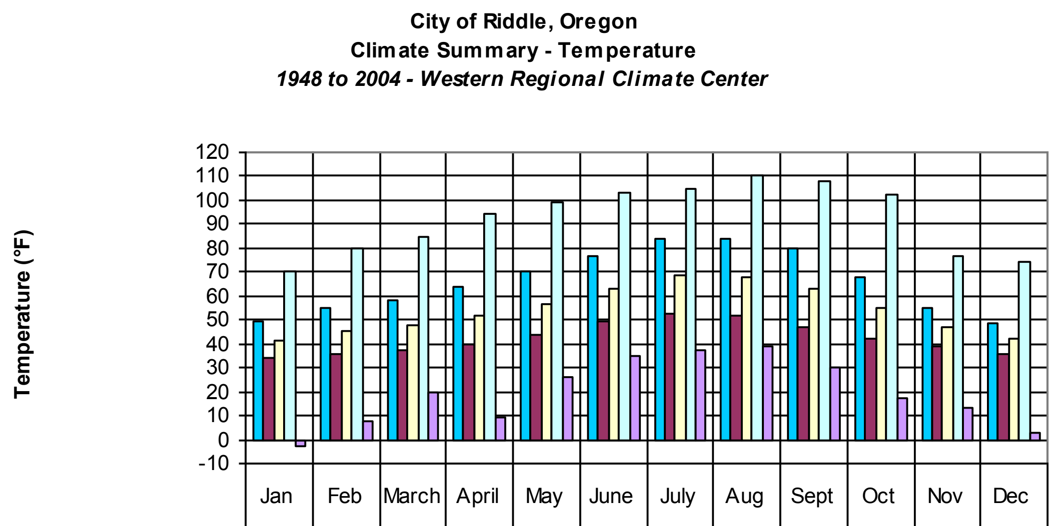
2.1.2 Climate

Climate information for Tri-City was obtained using long-term records collected at nearby weather stations in Myrtle Creek and Riddle. Temperature data is not available for the closest station in Myrtle Creek so the Riddle data (Station 357169) is shown. Precipitation data is from the Myrtle Creek station (Station 355891).

The average temperature ranges from the low 40s to the high 60s with an annual average of 54°F. Extreme temperature recordings range from -3°F to 110°F and summer days approaching and exceeding 100°F are not uncommon. The temperature exceeds 90°F an average of 24 days per year. The warmest months are generally July and August with an average mean of slightly above 68°F. Mean monthly average winter temperatures are always above freezing however the temperature drops below freezing an average of 43 days per year.

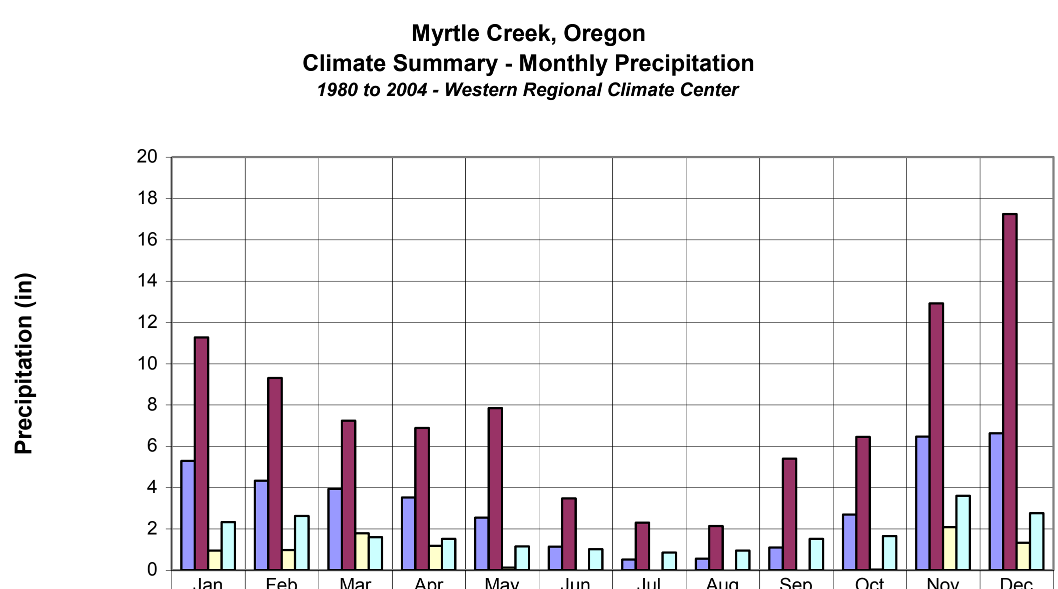
The average annual precipitation in the area is 38.7-inches. Most precipitation is in the form of rainfall as the average annual snowfall is only 2.4 inches. Monthly precipitation means range from a low of 0.52 inches in July to a high of 6.62 inches in December. The average number of days per year that rainfall exceeds 1-inch is 5 days. Approximately 42% of yearly precipitation occurs during the winter months (Dec.-Feb.). On average, about 6% of the annual precipitation occurs during summer months (Jun.-Aug.). Roughly equal amounts of rainfall occur in the Fall and Spring. The highest 1-day precipitation total recorded was 3.6 inches in November of 1998. The highest annual snowfall of 18 inches was recorded in 1990.

Based on the NOAA Atlas 2, Volume X Isopluvial maps, the 5-year, 24-hour precipitation is 3.1 inches.



	Jan	Feb	March	April	May	June	July	Aug	Sept	Oct	Nov	Dec
Avg. Max.	49	54.6	58.6	63.7	70.1	76.8	84.1	84	79.5	68.1	55.1	48.5
Avg. Min.	34.4	35.9	37.1	39.4	44	49.3	52.7	52.1	47.1	42.3	39.2	35.5
Avg. Mean	41.7	45.2	47.8	51.6	57	63	68.4	68.1	63.3	55.2	47.1	42
Extreme Day High	70	80	85	94	99	103	105	110	108	102	77	74
Extreme Day Low	-3	8	20	9	26	35	37	39	30	17	13	3

Figure 2-1 – Mean Monthly Temperature



	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Avg. Mean	5.29	4.33	3.93	3.52	2.54	1.13	0.52	0.56	1.1	2.69	6.47	6.62
Avg. High	11.27	9.31	7.23	6.88	7.84	3.48	2.3	2.13	5.4	6.45	12.92	17.24
Avg. Low	0.95	0.97	1.78	1.17	0.12	0	0	0	0	0.03	2.08	1.32
1-Day Max.	2.32	2.63	1.59	1.51	1.15	1.01	0.85	0.95	1.52	1.65	3.6	2.76

Figure 2-2 – Mean Monthly Precipitation

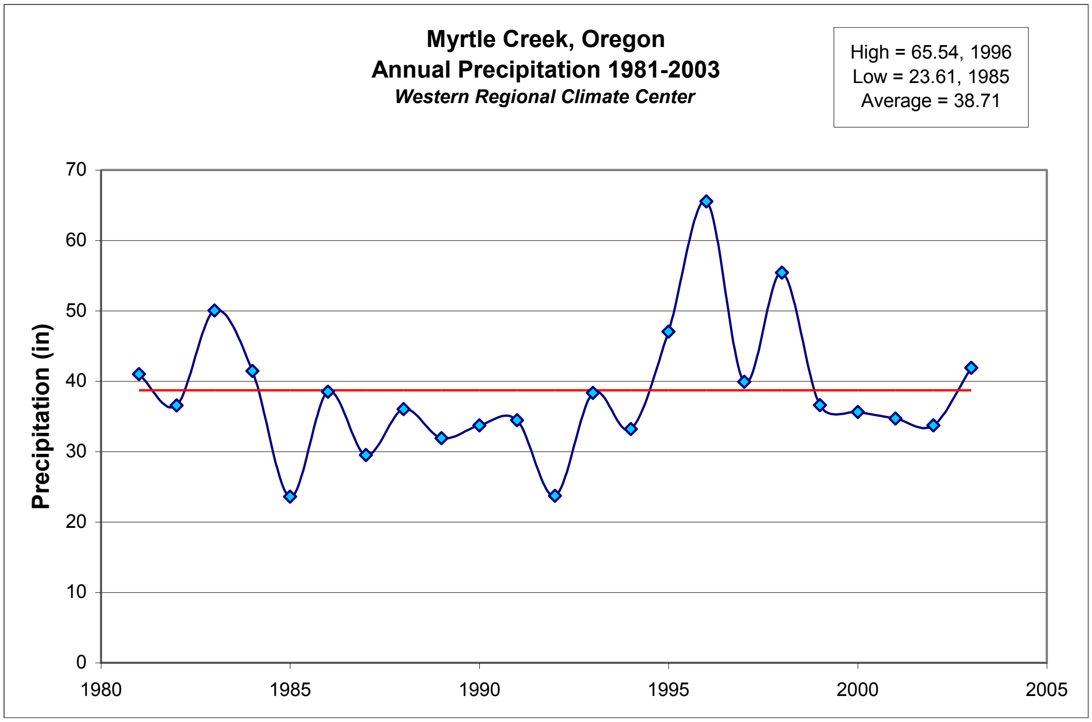


Figure 2-3 – Annual Precipitation

2.1.3 Geology

Area geology in this discussion was derived using the 1991 Geologic Map of Oregon (GIS version, 1:500,000 scale) by Walker and MacLeod. A map of the area geology is shown in Figure 2-4.

The low lying areas along the South Umpqua River are underlain by “Qal” formations consisting of alluvial deposits of sand, gravel, and silt. In places these formations can include talus and slope wash and local soils containing abundant organic material and thin peat beds. East of the “Qal” formations, in the majority of the area with slopes rising to the east, lies a sandstone, siltstone, and limestone conglomerate called the Myrtle Group and designated “Kjm”. A small portion of the lower elevation areas at the south end of the Authority Boundary are underlain by landslide and debris flow deposits designated “Qls”.

The water treatment facility is located near the intersection of “Qal” and the “Kjm” formations. All storage tanks are located in areas shown as “Kjm” stone conglomerate.

There is a fault line shown running north-south to the west of the area. An additional fault line is shown along the east side of the area approaching the Authority Boundary at the southeast corner near one of the storage tanks. The accuracy of the fault locations in this area is considered poor, being developed at 1:500,000 scale. Records do not show any activity at these faults.

2.1.4 Soils

The area soils are shown graphically in Figure 2-5. The area is predominantly covered with soil types consisting of gravelly sandy loam, banning loam, silty clay loam, gravelly loam, and silt loam. As the elevation rises to the east, some other soil types are encountered such as Beekman-Vermisa Complex, Josephine-Speaker Complex, and Debenger-Brader Complex.

Soils within the Tri City Urban Area are of three general types: fertile alluvial soil adjacent to the South Umpqua River and in areas subject to flooding; old terrace soils which were deposited on ancient floodplains and are less fertile than alluvial soils; and the upland soils which exist on steep slopes in the area.

2.1.5 Topography

The topography of the area is shown on the USGS quadrangle maps presented in Figure 1-2. More detailed (10-foot contours) topography based on aerial surveying is shown as Figure 2.1.5-1. Tri-City lies in a narrow valley formed by the South Umpqua River on the west and relatively steep mountains on the east. The area generally has topography rising gently from west to east with slopes less than 10%. Much of the area is within the “Missouri Bottom” being relatively flat and slightly above the water surface of the South Umpqua River. Some areas along the eastern boundary of the area have steep slopes on the order of 45% to 60% that hinder development. According to the Myrtle Creek Comprehensive Plan, approximately 72 acres of land within the Urban Area consist of land with slopes in excess of 25%. This land is often subject to slow earth flows and erosion, requiring additional engineering and construction techniques when developed for residential use. Forty-eight of these acres have been designed for low density residential use while 24 are in public usage.

2.1.6 Vegetation and Wildlife

A large portion of the Tri-City area is developed in an urban manner with a mixture of commercial, industrial, and residential development. The lower elevations are mostly disturbed with significant housing, roadways, and pastures. On the hills to the east small stands of Oak and miscellaneous shrublands and grasslands occur in between residential developments. East of the area and outside the study area is largely forested with evergreen forests (primarily Douglas fir) and mixed forests. The South Umpqua River and riparian strips along its banks serve as a major habitat area for quail, water fowl, non-game species and fish. Fish species include Chinook salmon, coho salmon, winter steelhead, sea-run cutthroat trout, smallmouth bass, American shad, native cutthroat, largescale sucker, Umpqua dace, Umpqua squawfish, Umpqua chub, redside shiner, Pacific lamprey, and speckled dace. The riparian habitat along the river channel includes Oregon ash, bigleaf maple, cottonwood, snowberry, willows, and a variety of other forbs and grasses.

The Myrtle Creek Comprehensive plans states that no endangered species are known to exist in the study area. An environmental assessment (EA) prepared in 2002 for the Tri-City/Myrtle Creek wastewater treatment plant includes documentation for listed species and species of concern that may occur in the area. At the time of the EA the only endangered species possibly occurring in the area was the Columbian white-tailed deer which has since been removed from the federal list of threatened and endangered species. Threatened species include the Bald eagle, Oregon coast Coho salmon, and Kincaid’s lupine. Candidate species include the Coastal cutthroat trout and the Steelhead. The South Umpqua River in the area is considered Essential Fish Habitat since the area is accessible to Chinook and Coho salmon, and no long-standing natural barriers are present downstream.

Land cover for the area from the National Land Cover Dataset (NLCD) for Oregon, 2-24-2000 version is shown as Figure 2-6.

2.1.7 Cultural Resources

Cultural resources are prehistoric and historic archaeological sites; historic buildings, structures, and records; certain types of museum collections; and traditional cultural or sacred properties that are important to Native Americans and other ethnic groups. There are no places within the study area listed on the National Register of Historic Places. No known structures are being considered for inclusion in the Register.

The State Historic Preservation Office (SHPO) has not conducted any previous cultural resource surveys within or near the project area and thus has no records of archaeological or other cultural resource sites. The area does however lie within an area generally perceived to have a high probability for possessing archaeological sites and/or buried human remains. SHPO does recommend that caution be taken during any ground disturbing projects. If any cultural material is discovered during construction activities, all work should cease until a professional archaeologist can assess the discovery.

2.1.8 Floodplains

The TCJWSA contains several areas within the 100-year floodplain generally along the South Umpqua River. The FIRM floodplain map for the area is shown in Figure 2-7.

The South Umpqua River has significant runoff and experiences relatively frequent flooding. According to the Myrtle Creek Comprehensive Plan, the Tri City Urban Area contains approximately 156 acres of land within the 100 year floodplain. Of this floodplain area, 74 acres are classified as being located within the floodway while 82 acres are within the flood fringe. Development within these areas is regulated by County ordinance to ensure certain minimum safety standards are met. Based on the FIRM floodplain map and 2-foot elevation contours of the TCJWSA generated through aerial surveying, there is a total of 364 acres of the study area located within the 100-year floodplain. The water treatment plant area is located within the floodplain. According to the FEMA FIRM Map, the 100-year flood elevation in the vicinity of the plant is approximately 623 feet above sea level based on NGVD 1929. According to the original plant construction plans the plant concrete floor slab was 623.0 feet. According to the 1999 plant upgrade plans, the original plant building floor is at 622.75 feet (*note 624.25 based on recent survey*) and the new office/clearwell floor slab is at 623.5 feet. It is not clear which vertical datum was used for the plant plans. Based on recent aerial surveying, the plant site appears to be elevated approximately 1 foot above the floodplain (624 feet).

2.1.9 Wetlands

A map of the area showing the NWI designated wetlands is shown in Figure 2-8. Printed maps for the Myrtle Creek and Canyonville Quads were obtained from the Oregon Department of State Lands. The data available has not been updated for some time and digital NWI data for the area is not yet available. Wetlands occur in the study area in riparian zones along the South Umpqua River, generally limited to the small immediate river bank area. A small Riverine wetland area is shown in the floodplain along the river bank to the west of Wecks Road. Three small Palustrine wetland areas are shown in low lying areas north of Woodcrest Drive. The backwash ponds at the water treatment plant are classified as wetlands as well. Two small stream/drainage routes in the south end of the area are designated wetlands. The larger of these flows from east to west running just north of Crest Ave., crossing the freeway, and terminating at the river. A portion of this larger drainage, just north of Weaver Ave., expands in width and is designated a Palustrine wetland according to the NWI map. A fairly new subdivision exists in this area now and the County may have determined that this area was not actually a wetland.

2.2 Land Use

Land in Tri-City is used for residential, commercial, industrial, farm, and public purposes. The Land Use Zoning Map is shown in Figure 2-9. The Tri-City urban area is defined in the Douglas County Comprehensive Plan as an area “irrevocably committed to urban uses". Approximately 1142 acres (58%) of the land is zoned for residential use. For R1 and R2 residential areas, Douglas County development code standards require a 6,500 square foot minimum lot size for a single family dwellings, 10,000 square foot minimum lot size for a duplexes and multi-family dwellings with 2,000 square feet per dwelling unit in multi-family complexes. Suburban Residential (RS) areas require a 15,000 square foot lot. Land outside the boundary is primarily zoned Farm Forest to the east and for cropland to the west.

Table 2.2-1
TCJWSA Land Use Zoning

Zone	Description	Area (acres)	% Area
R1	Single Family Residential	832	42.6%
R2	Multi-Family Residential	47	2.4%
RS	Suburban Residential	263	13.5%
CT	Tourist Commercial	37	1.9%
C2	Commercial	33	1.7%
C3	Commercial	35	1.8%
M1	Industrial	17	0.9%
M2	Industrial	40	2.0%
M3	Industrial	87	4.5%
F1	Exclusive Farm Use - Crop	74	3.8%
F2	Exclusive Farm Use - Crop	79	4.0%
FF	Farm Forest	12	0.6%
FG	Exclusive Farm Use - Grazing	50	2.6%
PR	Public Reserve	89	4.6%
R/W	Road Right-Of-Way	259	13.3%
		1954	

Within the last 2 years, over 100 new lots have been developed in R1 zoned areas with slopes ranging up to 30%. Housing has not yet been constructed on most of these lots but is expected soon. Based on the Census Data figure of an average of 2.6 people per household in Tri-City, 260 people could reside in these new lots. Additionally, over 165 acres of R1 land and 85 acres of RS land exists which has yet to be developed. Deleting areas with slopes in excess of 15% results in approximately 130 acres of R1 land and 85 acres of RS land. Based on these vacant buildable lands and development code standards, it appears that over 1,375 new single family dwelling units could easily be accommodated without zone changes. With a figure of 2.6 people per household, a population increase of 3,500 can easily be accommodated with the available land.

2.3 Population

2.3.1 Existing Population

Tri-City is an unincorporated community and long-term census data is not available for the Authority boundary. The State Drinking Water Program lists the current population of the TCJWSA at 3500 persons. A document prepared by the USDA Forest Service, "Delimiting Communities in the Pacific Northwest", April 2003, suggests a population of 3880 in Tri-City. Oregonprospector.com, lists the 2004 population of the Tri-City at 4082 persons.

A Tri-City Census Designated Place (CDP) exists with a boundary slightly larger than the Authority boundary. The CDP includes all of the TCJWSA plus some land to the east which is largely unpopulated. The 2000 census data lists 1348 occupied homes with 2.6 people per household and a population of 3519.

The most accurate estimate of current population can be obtained by looking at the actual number of dwellings in the boundary based on water accounts. For the period from 9/2004 to 8/2005, the water records show that 1340 single family dwellings exist. In addition, there are 204 mobile homes and 50 apartment units in 9 apartment complexes. Based on water use records, the 50 apartment units use an amount of water equal to 26 single family dwellings and the 204 mobile homes use an amount of water equal to 195 single family dwellings. Using the census figure of 2.6 people per household, there are 4059 people in the TCJWSA. These figures result in an average of 1.38 people per apartment unit and 2.48 people per mobile home unit.

Table 2.3-1 – Existing Residential Population

Dwelling Type	Units	EDU	People	Average People/Unit
Single Family Homes	1340	1340	3484	2.6
Apartment Units	50	26.5	69	1.38
Mobile Homes	204	194.7	506	2.48
Total	**1,594**	**1,561**	**4,059**	**2.546**

2.3.2 Projected Population

According to US Census data for Douglas County, the total population of the County increased from 14,565 in 1900 to a total of 100,399 in 2000. This growth is equivalent to an average annual growth rate of 1.9 % over the 100 year period. For the period from 1990 to 2000, growth in the County has average 0.6% per year.

Census data for the Tri-City CDP indicates that the number of housing units grew at an average annual rate of 5.0% from 1970 to 2000 while population grew at an annual average rate of 4.2% for the same 30 year period. Rapid growth occurred in the 1970's, slowed substantially in the 1980's, and actually declined in the 1990's according to the Census data.

Table 2.3-2 – Tri-City CDP Census Data

Year	Population	Total Housing Units	Occupied Housing Units	People/Household
1970	1039	325	?	?
1980	3439	1254	1230	2.8
1990	3585	1333	1280	2.8
2000	3519	1409	1348	2.6

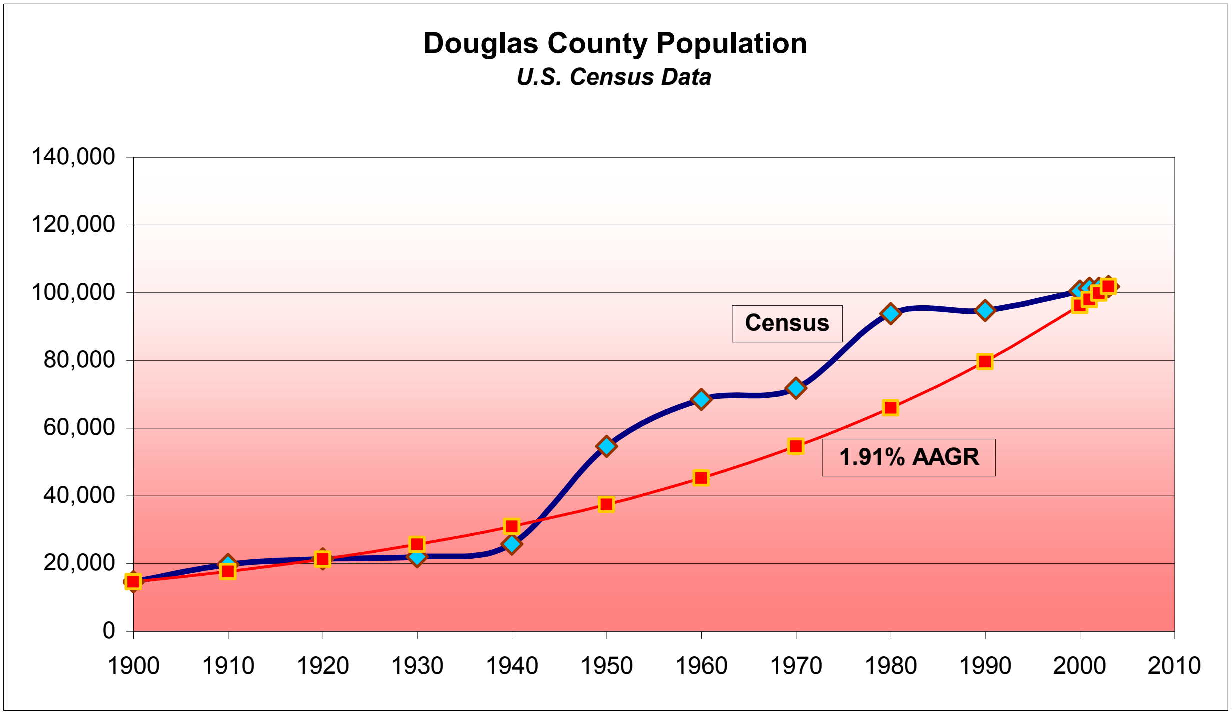


Figure 2.3-1 – Douglas County Historical Population and Growth

The Census Data for the Tri-City CDP is inaccurate and in fact population has continued to rise rather than decline as the Census Data suggests. In the 1994 Water System Master Plan prepared for Tri-City, actual residential customer accounts (year 1992 records) were used to estimate population resulting in an estimate of 3,775 persons in 1992 (0.56% AAGR to 2005). This estimate used 2.7 people per household and applied this same figure to mobile homes and apartments. If the current figures of 1.38 persons per apartment unit and 2.48 persons per mobile home unit are applied to the 1992 account records, the resulting 1992 population estimate is 3,673 persons (0.77% AAGR to 2005). An average growth of 0.8% annually is an accurate estimate of growth in Tri-City over the last decade. Development has increased significantly in recent years and growth in the near future is expected to be much greater than has occurred over the last decade.

A projected growth rate must be selected that ideally represents the next 20 years of growth in Tri-City as accurately as possible. If projected growth is lower than actually occurs, facilities that are upgraded may be undersized too soon. If projected growth is too high, facilities may be oversized. When improvements are required, it is more economical to be slightly conservative in growth projections to avoid spending money on facilities that are undersized in a relatively short time period. The Myrtle Creek Comprehensive Plan states that growth in Tri-City may be between 1.3% and 2.3%. Previous planning documents for Tri-City have used an AAGR of 2.5%. Recently, a growth rate of 2.5% was selected for the planning and design of the new Tri-City/Myrtle Creek wastewater treatment plant. For the purposes of this Water System Master Plan, the Tri-City Board has adopted a 2.5% average annual growth rate for the next 20 years.

Table 2.3-3 – Tri-City Projected Residential Population, 2.5% Growth Rate

Year	Dwelling Units	People	Average People/Unit
2005	1,594	4,059	2.546
2010	1,804	4,592	"
2015	2,041	5,196	"
2020	2,309	5,879	"
2025	2,612	6,651	"

Dwelling Units include homes, mobile homes, and apartments

Using a 2.5% average annual growth rate over the 20-year period results in a projected population of 6,651 at the end of the planning period, or an increase of 2,592 people. This increase is an average of 129.6 persons per year and 50.9 dwelling units per year. Approximately 1,018 new dwelling units in Tri-City will be required over the next 20 years to accommodate this growth. Sufficient buildable land exists within the current boundary to support this growth.

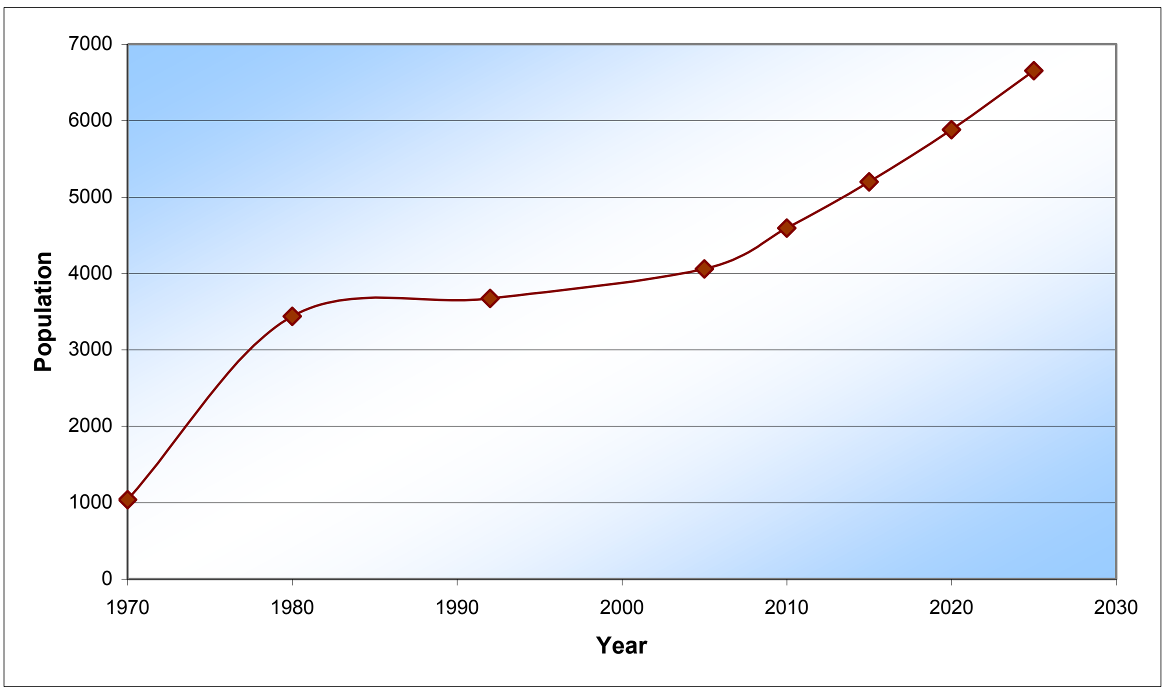


Figure 2.3-2 – Tri-City Historical and Projected Population

2.3.3 Equivalent Dwelling Units (EDU) and Service Population

Based on water sales records from the summer of 2004 to the summer of 2005, existing EDUs in Tri-City total 1,731. Residential customers account for 1,561 EDU. Each single-family dwelling in Tri-City is sold an average of 7,075 gallons of water per month, or 84,905 gallons per year. This amount of water is the amount a single EDU uses. Other users such as commercial and industrial can be described as a number of EDUs by dividing their average water use by the amount of water for a single EDU. For example, the schools in Tri-City used 3,712,108 gallons of water in one year. Dividing this amount by the amount a single residence uses (84,905 gal) shows that the schools are equivalent to 44 single family dwellings or 44 EDU based on water use.

Table 2.3-4 – Current System EDU

User Type	Accounts	Units	Annual Water Sales	Gal/Account	EDU
Single Family Res. Inside UGB	1,338	1,338	113,602,681	84,905	1,338
Single Family Res. Outside UGB	2	2	153,650	76,825	2
Multi-Family Res. Apartments	9	50	2,247,531	249,726	26.5
Mobile Home Parks	12	204	16,528,450	1,377,371	194.7
Restaurants (units = seats)	9	531	2,425,100	269,456	29
Small Business Commercial	53	53	3,370,783	63,600	40
Large Commercial	3	3	2,818,100	939,367	33
Public	3	3	493,270	164,423	6
Industrial	1	1	1,613,600	1,613,600	19
Schools	3	9	3,712,108	1,237,369	44
	1,433	2,194	146,965,273		1,731

Average Persons per Dwelling Unit = 2.546
Average Persons per Residential EDU = 2.60
Average Persons per Apartment Dwelling Unit = 1.38
Average Persons per Mobile Home Dwelling Unit = 2.48
Average Persons per Total System EDU = 2.345

Water Sales per EDU = 84,905 gal. per year = 7,075 gal. per month = 233 gallons per day
Residential Water Sales per Person = 32,656 gal. per year = 2,721 gal. per month = 90 gpcd

The current estimated residential population is 4,059 persons and the current total system EDUs is 1,731. The 170 non-residential EDUs are sold an amount of water equivalent to the use of 442 persons. The current estimated total equivalent service population is therefore 4,501.

Table 2.3-5 – Projected EDU and Equivalent Service Population

Year	Population	EDU Residential	EDU Commercial	EDU Industrial	EDU Public	EDU Schools	EDU Total	Equivalent Population
2005	4,059	1,561	101	19	6	44	1,731	4,501
2006	4,160	1,600	104	19	6	45	1,774	4,614
2007	4,264	1,640	106	20	6	46	1,819	4,729
2008	4,371	1,681	109	20	6	47	1,864	4,847
2009	4,480	1,723	111	21	7	49	1,911	4,968
2010	4,592	1,766	114	21	7	50	1,959	5,092
2011	4,707	1,810	117	22	7	51	2,008	5,220
2012	4,825	1,856	120	23	7	52	2,058	5,350
2013	4,945	1,902	123	23	7	54	2,109	5,484
2014	5,069	1,950	126	24	7	55	2,162	5,621
2015	5,196	1,998	129	24	8	56	2,216	5,762
2016	5,326	2,048	133	25	8	58	2,271	5,906
2017	5,459	2,100	136	26	8	59	2,328	6,053
2018	5,595	2,152	139	26	8	61	2,386	6,205
2019	5,735	2,206	143	27	8	62	2,446	6,360
2020	5,879	2,261	146	28	9	64	2,507	6,519
2021	6,026	2,318	150	28	9	65	2,570	6,682
2022	6,176	2,375	154	29	9	67	2,634	6,849
2023	6,331	2,435	158	30	9	69	2,700	7,020
2024	6,489	2,496	161	30	10	70	2,768	7,196
2025	6,651	2,558	166	31	10	72	2,837	7,375

Regulatory Requirements

Section 3

3.1 Responsibilities as a Water Supplier

Per OAR 333-061-0025, water suppliers are responsible for taking all reasonable precautions to assure that the water delivered to water users does not exceed maximum contaminant levels, to assure that water system facilities are free of public health hazards, and to assure that water system operation and maintenance are performed as required by these rules. This includes, but is not limited to, the following:

- Routinely collect and submit water samples for laboratory analyses at the frequencies and sampling points prescribed by OAR 333-061-0036 “Sampling and Analytical Requirements”;
- Take immediate corrective action when the results of analyses or measurements indicate that maximum contaminant levels have been exceeded and report the results of these analyses as prescribed by OAR 333-061-0040 “Reporting and Record Keeping”;
- Continue to report as prescribed by OAR 333-061-0040, the results of analyses or measurements which indicate that maximum contaminant levels (MCLs) have not been exceeded;
- Notify all customers of the system, as well as the general public in the service area, when the maximum contaminant levels have been exceeded;
- Notify all customers served by the system when the reporting requirements are not being met, or when public health hazards are found to exist in the system, or when the operation of the system is subject to a permit or a variance;
- Maintain monitoring and operating records and make these records available for review when the system is inspected;
- Maintain a pressure of at least 20 pounds per square inch (psi) at all service connections at all times (at the property line);
- Follow-up on complaints relating to water quality from users and maintain records and reports on actions undertaken;
- Conduct an active program for systematically identifying and controlling cross connections;
- Submit, to the DWP, plans prepared by a professional engineer registered in Oregon for review and approval before undertaking the construction of new water systems or major modifications to existing water systems, unless exempted from this requirement;
- Assure that the water system is in compliance with OAR 333-061-0205 “Water Personnel Certification Rules - Purpose” relating to certification of water system operators.
- Assure that Transient Non-Community water systems utilizing surface water sources or sources under the influence of surface water are in compliance with OAR 333-061-0065 “Operation and Maintenance” (2)(c) relating to required special training.

3.2 Public Water System Regulations

Water providers should always be informed of current standards, which can change over time, and should also be aware of pending future regulations. As of this writing, OAR Chapter 333, Division 61 covering Public Water Systems is over 280 pages in length. This Section is not meant to be a comprehensive list of all requirements but a summary of the general requirements.

Specific information on the regulations concerning public water systems may be found in the Oregon Administrative Rules (OAR), Chapter 333, Division 61. The rules can be found on the Internet at http://oregon.gov/DHS/ph/dwp/rules.shtml where copies of all the rules and regulations can be printed out or downloaded for reference. A summary of Oregon drinking water quality standards published in "*Pipeline*" (Volume 19, Issue 4, Fall 2004) by the State Drinking Water Program is included in Appendix A.

Drinking water regulations were established in 1974 with the signing of the Safe Drinking Water Act (SDWA). This act and subsequent regulations were the first to apply to all public water systems in the United States. The Environmental Protection Agency (EPA) was authorized to set standards and implement the Act. With the enactment of the Oregon Drinking Water Quality Act in 1981, the State of Oregon accepted primary enforcement responsibility for all drinking water regulations within the State. Requirements are detailed in OAR Chapter 333, Division 61. The SDWA and associated regulations have been amended several times since inception with the goal of further protection public health.

SDWA requires EPA to regulate contaminants which present health risks and are known, or are likely, to occur in public drinking water supplies. For each contaminant requiring federal regulation, EPA sets a non-enforceable health goal, or maximum contaminant level goal (MCLG). This is the level of a contaminant in drinking water below which there is no known or expected risk to health. EPA is then required to establish an enforceable limit, or maximum contaminant level (MCL), which is as close to the MCLG as is technologically feasible, taking cost into consideration. Where analytical methods are not sufficiently developed to measure the concentrations of certain contaminants in drinking water, EPA specifies a treatment technique, instead of an MCL, to protect against these contaminants.

Water systems are required to collect water samples at designated intervals and locations. The samples must be tested in state approved laboratories. The test results are then reported to the state, which determines whether the water system is in compliance or violation with the regulations. There are three main types of violations:

> (1) MCL violation — occurs when tests indicate that the level of a contaminant in treated water is above EPA or the state's legal limit (states may set standards equal to, or more protective than, EPA's). These violations indicate a potential health risk, which may be immediate or long-term.
>
> (2) Treatment technique violation — occurs when a water system fails to treat its water in the way prescribed by EPA (for example, by not disinfecting). Similar to MCL violations, treatment technique violations indicate a potential health risk to consumers.
>
> (3) Monitoring and reporting violation — occurs when a system fails to test its water for certain contaminants, or fails to report test results in a timely fashion. If a water system does not monitor its water properly, no one can know whether or not its water poses a health risk to consumers.

If a system violates EPA/state rules, it is required to notify the state and the public. States are primarily responsible for taking appropriate enforcement actions if systems with violations do not return to compliance. States are also responsible for reporting violation and enforcement information to EPA quarterly.

There are now EPA-established drinking water quality standards for 91 contaminants, including seven microbials and turbidity, seven disinfection byproducts and residuals, 16 inorganics (including lead and copper), 56 organics, and five radiologic contaminants. These standards either have established MCLs or treatment techniques.

New rules in effect since the year 2000 include the Interim Enhanced Surface Water Treatment Rule, the Filter Backwash Recycling Rule, the Long-Term Stage 1 Enhanced Surface Water Treatment Rule, the Stage 1 Disinfectant/Disinfection Byproducts Rule, and revisions to the Lead and Copper Rule.

A general summary of current rules is included in the Section for a surface water system using conventional filtration treatment and serving less than 10,000 persons.

Total Coliform Rule

Routine samples collected by Oregon public water suppliers are analyzed for total coliform bacteria. All community systems, and noncommunity systems using surface water sources or serving over 1,000 people, must sample monthly. All other systems must test for coliform bacteria once per quarter. For systems serving between 1,001 and 2,500 people 2 samples per month are required. Systems serving between 2,501 and 3,300 people are required to take 3 samples per month. Systems serving between 3,301 and 4,100 people are required to take 4 samples per month. Systems serving between 4,101 and 4,900 people are required to take 5 samples per month. Systems greater than 4,900 persons have additional sampling requirements.

Compliance is based on the presence or absence of total coliforms in any calendar month (or quarter). Sample results are reported as “coliform-absent” or “coliform-present”. If any sample is coliform-present, a set of at least three repeat samples must be collected within 24 hours. Small water systems that collect one routine sample per month or fewer must collect a fourth repeat sample. Repeat sampling continues until the maximum contaminant level is exceeded or a set of repeat samples with coliform-absent results is obtained.

Small systems (fewer than 40 samples/month) are allowed no more than one coliform-present sample per month, including any repeat sample results. Larger systems (40 or more samples/ month) are allowed no more than five percent coliform-present samples in any month, including any repeat sample results. Confirmed presence of fecal coliform or *E. coli* presents an acute health risk and requires immediate notification of the public to take protective actions such as boiling or using bottled water.

Surface Water Treatment Rules

Water systems must provide a total level of filtration and disinfection treatment to remove/inactivate 99.9 percent (3-log) of *Giardia lamblia*, and to remove/inactivate 99.99 percent (4-log) of viruses. In addition, filtered water systems must physically remove 99 percent (2-log) of *Cryptosporidium.* Filtered water systems must meet specified performance standards for combined filter effluent turbidity levels, and water systems using conventional and direct filtration must also record individual filter effluent turbidity and take action if specified action levels are exceeded. When more than 1 filter exists, each filter’s effluent turbidity must be monitored continuously and recorded at least every 15 minutes. The combined flow from all filters must have a turbidity measurement at least every four hours by grab sampling or continuous monitoring. Compliance is based on the combined filter effluent and 100% of measurements must be less than or equal to 1 NTU and 95% of the readings taken in any month must be less than or equal to 0.3 NTU.

- Individual filter turbidity monitored continuously and recorded every 15 minutes or less
- Combined filter turbidity monitored continuously or grab sample taken at least every 4 hours
- Combined filter turbidity less than 1 NTU in 100% of measurements
- Combined filter turbidity less than or equal to 0.3 NTU in 95% of measurements in a month
- Specific follow-up actions if individual filter turbidity exceeds 1.0 NTU twice

All water systems must meet specified CxT [concentration x time] requirements for disinfection, and meet required removal/inactivation levels. In addition, a disinfectant residual must be maintained in the distribution system.

- Continuous recording of disinfectant residual at entry point to the distribution system. Small system may be allowed to substitute 1-4 daily grab samples.
- Daily calculation of CxT at highest flow (peak hourly flow)
- Provide adequate CxT to meet needed removal/inactivation levels
- Maintain a continuous minimum 0.2 mg/L disinfectant residual at entry point to the distribution system
- Maintain a minimum detectable disinfectant residual in 95% of the distribution system samples (collected at coliform bacteria monitoring points)
- Conduct disinfection profiling and benchmarking

Filtered water systems that recycle spent filter backwash water or other waste flows must return those flows through all treatment processes in the filtration plant. Systems wishing to recycle filter backwash water must provide notice to the State including a plant schematic showing the origin, conveyance, and return location of recycled flows. Design flows, observed flows, and typical recycle flows are also required along with a state-approved plant operating capacity.

Disinfectants and Disinfection Byproducts

Disinfection treatment chemicals used to kill microorganisms in drinking water can react with naturally occurring organic and inorganic matter in source water, called DBP precursors, to form disinfection byproducts (DBPs). Some disinfection byproducts have been shown to cause cancer and reproductive effects in lab animals and suggested bladder cancer and reproductive effects in humans. The challenge is to apply levels of disinfection treatment needed to kill disease-causing microorganisms while limiting the levels of disinfection byproducts produced. The primary disinfection byproducts of concern in Oregon are the trihalomethanes (TTHM) and the haloacetic acids (HAA5).

Disinfection byproducts must be monitored throughout the distribution system at frequencies daily, monthly, quarterly or annually, depending on the population served, type of water source, and the specific disinfectant applied, and in accordance with an approved monitoring plan. Disinfectant residuals must be monitored at the same locations and frequency as coliform bacteria.

Total organic carbon (TOC) is an indicator of the levels of DBP precursor compounds in the source water. Systems using surface water sources and conventional filtration treatment must monitor source water for TOC and alkalinity monthly and practice enhanced coagulation to remove TOC if it exceeds 2.0 mg/L as a running annual average.

Compliance is determined based on meeting maximum contaminant levels (MCLs) for disinfection byproducts and maximum levels for disinfectant residual (MRDLs) over a running annual average of the sample results, computed quarterly.

- TTHM/HAA5 monitoring required in distribution system. One sample per quarter for systems serving 500-9,999 persons. One sample per year in warmest month required for systems serving less than 500. MCL for TTHM is 0.080 mg/L. MCL for HAA5 is 0.060 mg/L.
- TOC and alkalinity monitoring in source water monthly. Enhanced coagulation if TOC greater than 2.0 mg/L
- Comply with MRDLs. Limit for chlorine (free Cl_2 residual) is 4.0 mg/L. Limit for chloramines is 4.0 mg/L (as total Cl_2 residual). Limit for chlorine dioxide is 0.8 mg/L (as ClO_2)
- Bromate MCL of 0.010 mg/L
- Chlorite MCL of 1.0 mg/L

Lead and Copper

Excessive levels of lead and copper are harmful and rules exist to limit exposure through drinking water. Lead and copper enter drinking water mainly from corrosion of plumbing materials containing lead and copper. Lead comes from solder and brass fixtures. Copper comes from copper tubing and brass fixtures. Protection is provided by limiting the corrosivity of water sent to the distribution system. Treatment alternatives include pH adjustment, alkalinity adjustment, or both, or adding passivating agents such as orthophosphates.

Samples from community systems are collected from homes built prior to the 1985 prohibition of lead solder in Oregon. One-liter samples of standing water (first draw after 6 hours of non-use) are collected at homes identified in the water system sampling plan. Two rounds of initial sampling are required, collected at 6-month intervals. Subsequent annual sampling from a reduced number of sites is required after demonstration that lead and copper action levels are met. After three rounds of annual sampling, samples are required every 3 years. The number of initial and reduced samples required is dependant on the population served by the water system.

In each sampling round, 90% of samples from homes must have lead levels less than or equal to the Action Level of 0.015 mg/L and copper levels less than or equal to 1.3 mg/L. Water systems with lead above the Action Level must conduct periodic public education, and either install corrosion control treatment, change water sources, or replace plumbing.

- Have Sampling Plan for applicable homes
- Collect required samples
- Meet Action Levels for Lead and Copper (0.015 mg/L for Lead and 1.3 mg/L for Copper)
- If Action Levels not met, provide corrosion control treatment and other steps

Inorganic Contaminants

The level of many inorganic contaminants is regulated for public health protection. These contaminants are both naturally occurring and can result from agriculture or industrial operations. Inorganic contaminants most often come from the source of water supply, but can also enter water from contact with materials used for pipes and storage tanks. Regulated inorganic contaminants include arsenic, asbestos, fluoride, mercury, nitrate, nitrite, and others. Compliance is achieved by meeting the established MCLs for each contaminant. Systems that cannot meet one or more MCL must either install treatment systems (such as ion exchange or reverse osmosis) or develop alternate sources of water.

- Sample quarterly for Nitrate (reduction to annual may be available)
- Communities with Asbestos Cement (AC) pipe must sample every 9 years for Asbestos
- Sample annually for Arsenic. New MCL of 0.010 mg/L effective January 2006
- Sample annually for all other inorganics. Waivers are available based on monitoring records showing three samples below MCLs. MCLs vary based on contaminant

Organic Chemicals

Organic contaminants are regulated to reduce exposure to harmful chemicals through drinking water. Examples include acrylamide, benzene, 2,4-D, styrene, toluene, and vinyl chloride. Major types of organic contaminants are Volatile Organic Chemicals (VOCs) and Synthetic Organic Chemicals (SOCs). Organic contaminants are usually associated with industrial or agricultural activities that affect sources of drinking water supply, including industrial and commercial solvents and chemicals, and pesticides. These contaminants can also enter from materials in contact with the water such as pipes, valves and paints and coatings used inside water storage tanks.

At least one test for each contaminant from each water source is required during every 3-year compliance period. Public water systems serving more than 3,300 people must test twice during each 3-year compliance period for SOCs. Public water systems using surface water sources must test for VOCs annually. Compliance is achieved by meeting the established MCL for each contaminant. Quarterly follow up testing is required for any contaminants that are detected above the specified MCL. Only those systems determined by the State to be at risk must monitor for dioxin. Water systems using polymers containing acrylamide or epichlorohydrin in their water treatment process must keep their dosages below specified levels. Systems that cannot meet one or more MCL must either install or modify water treatment systems (such as activated carbon and aeration) or develop alternate sources of water.

- At least one test for each contaminant (for each water source) every 3-year compliance period
- Sample twice each compliance period for each SOCs when system over 3,300 people
- Test VOCs annually
- Quarterly follow up testing required for any detects above MCL
- Maintain polymer dosages in treatment process below specified levels
- MCLs vary based on contaminant

Radiologic Contaminants

Radioactive contaminants, both natural and man-made, can result in an increased risk of cancer from long-term exposure and are regulated to reduce exposure through drinking water. Rules were recently revised to include a new MCL for uranium, and to clarify and modify monitoring requirements. Initial monitoring tests, quarterly for one year at the entry point from each source, must be completed by December 31, 2007 for gross alpha, radium-226, radium-228 and uranium. A single analysis for all four contaminants collected between June 2000 and December 2003 will substitute for the four initial samples. Gross alpha may substitute for radium-226 if the gross alpha result does not exceed 5 pCi/L and may substitute for uranium monitoring if the gross alpha result does not exceed 15 pCi/L. Subsequent monitoring is required every three, six, or nine years depending on the initial results, with a return to quarterly monitoring if the MCL is exceeded. Compliance with MCLs is based on the average of the four initial test results, or subsequent quarterly tests. Community water systems than cannot meet MCLs must install treatment (such as ion exchange or reverse osmosis) or develop alternate water sources.

- Conduct initial quarterly tests for one year by 12-31-2007 (prior tests may be accepted)
- Subsequent monitoring every 3, 6, or 9 years depending on initial results
- Comply with MCLs based on average of tests
- New MCL of 30 µg/L for Uranium. Other MCLs vary based on contaminant

3.3 Future Water System Regulations

The 1996 Safe Drinking Water Act (SDWA) requires EPA to review and revise as appropriate each current standard at least every six years. Data continues to be collected on contaminants currently unregulated in order to support development of future drinking water standards. Drinking water contaminant candidate lists (DWCCL) are prepared and revised every five years. The first DWCCL was published on March 2, 1998 which included 51 chemicals and 9 microbials. In 2003, EPA decided not to regulate any of the 9 microbials from the initial list. On April 2, 2004 EPA published a draft second DWCCL consisting of the remaining 51 contaminants from the first list. The EPA must publish a decision on whether to regulate at least five contaminants from the DWCCL every 5 years. As a result, additional contaminants can become regulated in the future.

In addition, rule revisions and new rules will occur to further address health risks from disinfection byproducts and pathogenic organisms. Rules such as the Long-Term Stage 1 Enhanced Surface Water Treatment Rule (LT1ESWTR) and the Stage 1 Disinfectants/Disinfection Byproducts (State 1 D/DBP) Rule have recently gone into effect. These rules added *Cryptosporidium* as a pathogen of concern, decreased the acceptable turbidity levels, addressed disinfectants and disinfection byproducts, and lowered MCLs for certain contaminants. New and revised drinking water quality standards are mandated under the 1996 federal SDWA. Known future standards (and their likely EPA promulgation date) include:

- Groundwater Rule (2005)
- Enhanced Surface Water Treatment Rule, Stage 2 (2005) – <u>EPA published Jan. 5, 2006</u>
- Disinfectants and Disinfection Byproducts Rule, Stage 2 (2005) – <u>EPA published Jan. 4, 2006</u>
- Radon Rule (2005-06)
- Distribution Rule, including revised coliform bacteria requirements (2008)

Water suppliers should be aware of and familiar with these mandates and deadlines, and plan strategically to meet them. DHS, under the Primacy Agreement with the EPA, has up to two years to adopt each federal rule after it is finalized. Water suppliers generally have at least three years to comply with each federal rule after it is finalized; however, some of these rules will likely establish a significant number of compliance dates for water suppliers that will occur prior to state adoption of the rules. These “early implementation” dates will likely have to be implemented in Oregon directly by the EPA, because the state program will not yet have the rules in place or the resources to carry them out.

These anticipated rules are described generally below. Additional details will be found in the final EPA rules once they are promulgated.

Groundwater Rule

Monitoring will be required for specific pathogenic organisms and/or indicator organisms such as enteric viruses, or surrogate organisms. In addition, all public water supply wells must be evaluated for their hydrogeologic sensitivity to viruses, including well construction, site geology and source water area. Compliance will be achieved by demonstrating a low hydrogeologic sensitivity to viruses, modifying well construction if needed, or by installing disinfection treatment to inactivate viruses.

Long-Term Stage 2 Enhanced Surface Water Treatment Rule (LT2ESWTR)

The rule will apply to all public water systems using surface water sources of supply. The rule will identify those surface water supplies that are at high risk of *Cryptosporidium*, and prescribe additional levels of treatment selected from a matrix of options. Future standards are likely to require water systems with high levels of pathogen in the source water to add treatment beyond standard filtration and disinfection. Monitoring of source water will be required for specific pathogenic organisms including *Cryptosporidium, E. coli*, and turbidity. Compliance will be demonstrated by meeting a maximum running annual average in source water for pathogens, or by meeting additional treatment technique requirements associated with the levels of pathogens found if those levels are above the maximum.

- Filtered water systems will be classified in one of four treatment categories (bins) based on their monitoring results. Most systems are expected to be classified in the lowest bin and will face no additional requirements. Systems classified in higher bins must provide additional water treatment to further reduce *Cryptosporidium* levels by 90 to 99.7 percent (1.0 to 2.5-log), depending on the bin. Systems will select from different treatment and management options in a "microbial toolbox" to meet their additional treatment requirements.

- All unfiltered water systems must provide at least 99 or 99.9 percent (2 or 3-log) inactivation of *Cryptosporidium*, depending on the results of their monitoring.

- Systems that store treated water in open reservoirs must either cover the reservoir or treat the reservoir discharge to inactivate 4-log virus, 3-log *Giardia lamblia*, and 2-log *Cryptosporidium*. These requirements are necessary to protect against the contamination of water that occurs in open reservoirs.

- Disinfection Benchmarking: Systems must review their current level of microbial treatment before making a significant change in their disinfection practice. This review will assist systems in maintaining protection against microbial pathogens as they take steps to reduce the formation of disinfection byproducts under the Stage 2 Disinfection Byproducts Rule, which EPA is finalizing along with the LT2ESWTR.

Disinfectants and Disinfection Byproducts Rule, State 2 (Stage 2 D/DBP)

The rule will apply to all water systems that apply disinfectants or distribute water that has been disinfected. The main goal of the Stage 2 rule is to control peak DBP levels within the water distribution system. Systems will monitor for DBP at sample locations where peak levels are expected, as identified in an Initial Distribution System Evaluation (IDSE). Large systems must complete the IDSE within two years of the final rule date and small systems within 4 years of the final rule date. Compliance is based on meeting the Locational Running Annual Average (LRAA) for DBPs at each sampling location in the distribution system in two phases. Phase 1: meet LRAA at each stage 1 sampling point for TTHM (120 µg/L) and HAA5 (100 µg/L). Phase 2: meet LRAA at each IDSE-identified sampling point for TTHM (80 µg/L) and HAA5 (60 µg/L) within 6-8½ years of the final rule, depending on system size.

- Systems will conduct an evaluation of their distribution systems, known as an Initial Distribution System Evaluation (IDSE), to identify the locations with high disinfection byproduct concentrations. These locations will then be used by the systems as the sampling sites for Stage 2 DBP rule compliance monitoring.

- Compliance with the MCLs for two groups of disinfection byproducts (TTHM and HAA5) will be calculated for each monitoring location in the distribution system. This approach, referred to as the locational running annual average (LRAA), differs from current requirements, which determine compliance by calculating the running annual average of samples from all monitoring locations across the system.

- The Stage 2 DBP rule also requires each system to determine if they have exceeded an operational evaluation level, which is identified using their compliance monitoring results. The operational evaluation level provides an early warning of possible future MCL violations, which allows the system to take proactive steps to remain in compliance. A system that exceeds an operational evaluation level is required to review their operational practices and submit a report to their state that identifies actions that may be taken to mitigate future high DBP levels, particularly those that may jeopardize their compliance with the DBP MCLs.

Radon Rule

All community water systems using groundwater sources will conduct quarterly initial sampling at distribution system entry points for one year. Subsequent sampling will occur once every 3 years. The Radon MCL is expected to be 300 pCi/L. An alternative MCL (AMCL) of 4,000 pCi/L is proposed if the State develops and adopts an EPA-approved statewide Multi-Media Mitigation (MMM) program. Local communities may have the option of developing an EPA-approved local MMM program in the absence of a statewide MMM program, and meeting the AMCL.

Distribution Rule

Under this rule, current requirements for coliform bacteria will be revised, emphasizing fecal coliforms and *E. coli*, and focusing on protection of water within the distribution system. The rule will apply to all public water systems and will involve identifying and correcting sanitary defects and hazards in water systems and using best management practices for disinfection to control coliform bacteria in the system.

3.4 Water Management and Conservation Plans

The Municipal Water Management and Conservation Planning (WMCP) program provides a process for municipal water suppliers to develop plans to meet future water needs. Many municipal water suppliers are required to prepare plans under water right permit conditions. In addition, with the revision of the permit extension rules in fall 2002, communities seeking long-term permit extensions will be required to prepare plans. These plans will be used to demonstrate the communities' needs for increased diversions of water under the permits as their demands grow. Rules for WMCPs are detailed in OAR 690, Division 86.

A WMCP provides a description of the water system, identifies the sources of water used by the community, and explains how the water supplier will manage and conserve supplies to meet future needs. Preparation of a plan is intended to represent a pro-active evaluation of the management and conservation measures that suppliers can undertake. The planning program requires municipal water suppliers to consider water that can be saved through conservation practices as a source of supply to meet growing demands if the saved water is less expensive that developing new supplies. As such, a plan represents an integrated resource management approach to securing a community's long-term water supply.

Many of the elements required in a plan are also required under similar plans by the Drinking Water Section of the state Department of Human Services (water system master plans) and Department of Land Conservation and Development (public facilities plans). Water providers can consolidate overlapping plan elements and create a single master plan that meets the requirements of all three programs.

Every municipal water supplier required to submit a WMCP shall exercise diligence in implementing the approved plan and shall update and resubmit a plan consistent with the requirements of the rules as prescribed during plan approval. Progress reports are required showing 5-year benchmarks, water use details, and a description of the progress made in implementing the associated conservation or other measures.

The WMCP shall include the following elements:

1) Water System Description including infrastructure details, supply sources, service area and population, details of water use permits and certificates, water use details, customer details, system schematic, and leakage information.

2) Water Conservation Element including description of conservation measures implemented and planned, water use and reporting program details, progress on conservation measures, and conservation benchmarks.

3) Water Curtailment Element including current capacity limitations and supply deficiencies, three or more stages of alert for potential water shortages or service difficulties, levels of water shortage severity and curtailment action triggers, and specific curtailment actions to be taken for each stage of alert.

4) Water Supply Element detailing current and future service areas, estimates of when water rights and permits will be fully exercised, demand projections for 10 and 20 years, evaluation of supply versus demand, and additional details should an expansion of water rights be anticipated.

Failure to comply with rules for WMCPs can result in enforcement actions by the Water Resources Department Director. Enforcement actions can include requirements for additional information and planning, water use regulation, cancellation of water use permits, or civil penalties under OAR 690-260-0005 to 690-260-0110.

Water Demand

Section 4

4.1 Existing Water Use

4.1.1 Definitions

Water demand is the quantity of water delivered to the system over a period of time to meet the needs of consumers and to supply the needs of fire fighting and system flushing. Additionally, virtually all systems have a small amount of leakage that cannot be economically removed and total demand usually includes some leakage. The difference between the amount of water sold and the amount delivered to the system is attributed to flushing, leakage, fire fighting and other non-metered usage. Demand varies seasonally with the lowest usage in winter months and the highest usage during summer months. Variations in demand also occur with respect to time of day (diurnal) with higher usage occurring during the morning breakfast and early evening periods and lowest usage during nighttime hours.

The objective of this section is to determine the current water demand characteristics and to project future demand requirements that will establish system component adequacy and sizing needs. Water demand is described in the following terms:

Average Annual Demand (AAD) - The total volume of water delivered to the system in a full year expressed in gallons. When demand fluctuates up and down over several years, an average is used.

Average Daily Demand (ADD) - The total volume of water delivered to the system over a year divided by 365 days. The average use in a single day expressed in gallons per day.

Maximum Monthly Demand (MMD) - The gallons per day average during the month with the highest water demand. The highest monthly usage typically occurs during a summer month.

Peak Weekly Demand (PWD) - The greatest 7-day average demand that occurs in a year. Expressed in gallons per day.

Maximum Day Demand (MDD) - The largest volume of water delivered to the system in a single day expressed in gallons per day. The water supply, treatment plant and transmission lines should be designed to handle the maximum day demand.

Peak Hourly Demand (PHD) - The maximum volume of water delivered to the system in a single hour expressed in gallons per day. Distribution systems should be designed to adequately handle the peak hourly demand. During this peak usage, storage reservoirs supply the demand in excess of the maximum day demand.

Demands described above, expressed in gallons per day (gpd), can be divided by the population or EDUs served to come up with a demand per person or a per capita demand which is expressed in gallons per capita per day (gpcd), or demand per EDU (gpd/EDU). These unit demands can be multiplied by future population or EDU projections to determine future water demands.

4.1.2 Existing Water Demand

The volume of water pumped from the source (South Umpqua River) and pumped into the WTP and system indicates the system demands. System leaks, distribution flushing, meter inaccuracies, backwash water, and other unmetered water uses cause production quantities to be higher than consumption quantities, however, portions of these unmetered flows that cannot be eliminated must still be provided for.

Plant production records were obtained for January 2001 through September 2005. Plant output to the system is approximately 1,000 gallons per minute. For this period the AAD is 195,403,075 gallons per year. Based on these records, an average of 16,165,820 gallons per month is treated at the WTP. Lowest monthly production of 10,253,700 gallons occurred in February 2002 and highest production of 26,891,300 gallons occurred in July 2003. On average, the plant must operate at least 9 hours per day to meet demand in the system. During peak summer months when more than 25,000,000 gallons of water is needed each month, the plant must operate an average of 14 hours per day.

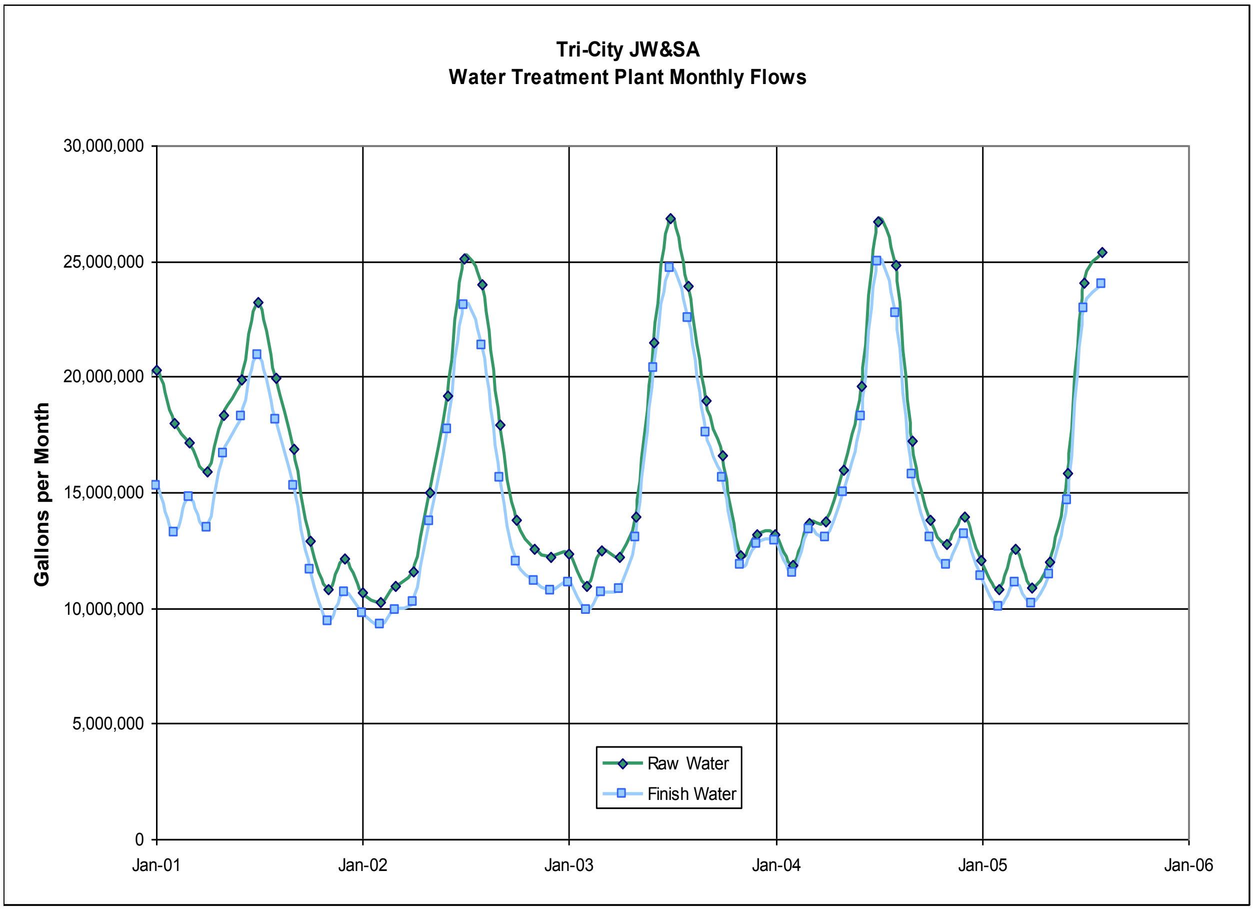


Figure 4.1.2-1 – Monthly WTP Production

An average of 1,435,320 gallons per month is used at the WTP for plant operations such as filter backwash, desludging of the sedimentation basins, turbidity monitoring, etc. This amount of water equals 8.9% of the raw water treated at the plant and is a fairly typical amount of water used to operate this size of a conventional filtration plant.

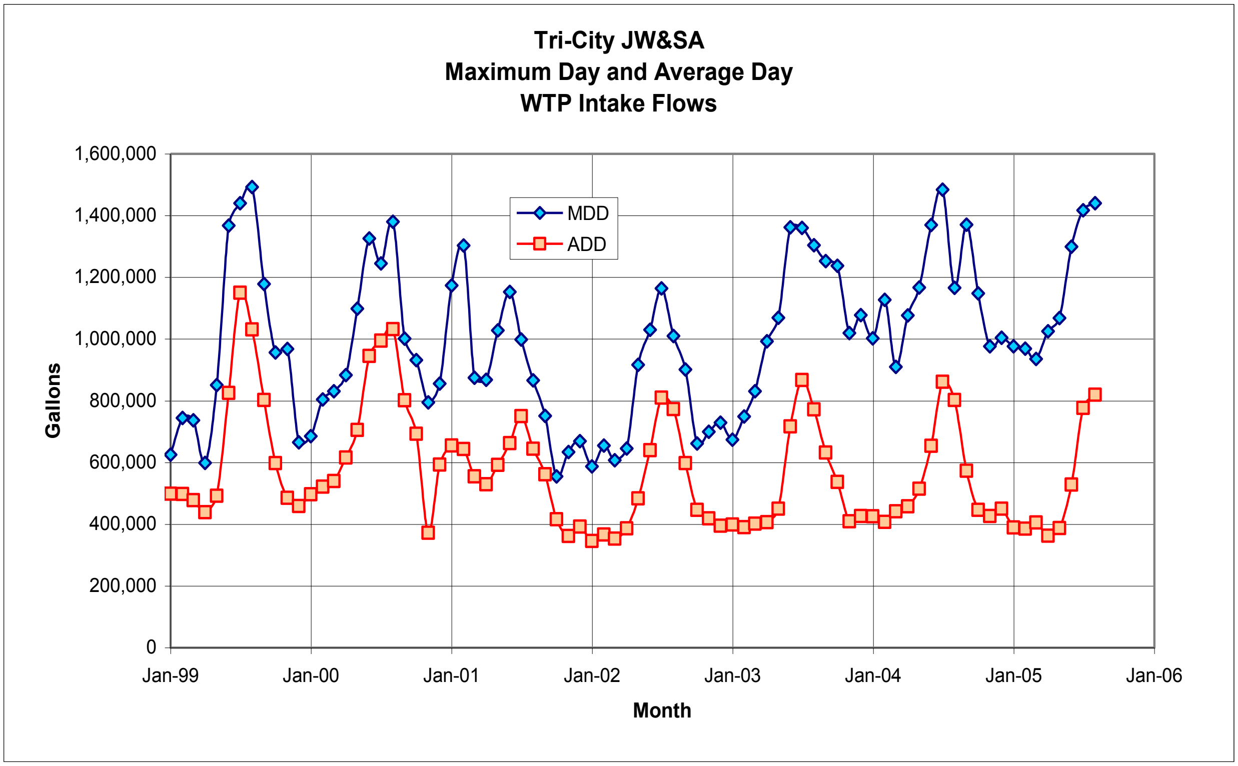


Figure 4.1.2-2 – Maximum Day and Average Day WTP Production

Table 4.1.2-1 – Water Production Summary

Year	AAD (gallons)	Max Month (gal.)	MMD (gpd)	ADD (gpd)	MDD (gpd)	P.F. MDD/ADD	P.F. MMD/ADD	Days Over 1 MGD
2001	205,583,800	23,238,200	749,619	563,243	1,302,900	2.31	1.33	1
2002	183,322,900	25,123,300	810,429	502,255	1,164,100	2.32	1.61	3
2003	195,308,600	26,891,300	867,461	535,092	1,361,800	2.54	1.62	55
2004	197,397,000	26,702,100	861,358	539,336	1,484,100	2.75	1.60	39
2005*		25,406,300	819,558	508,945	1,458,720	2.87	1.61	22
Average	*195,403,075*	*25,472,240*	*821,685*	*534,981*	*1,354,324*	*2.56*	*1.55*	

* January through August

Current Design Values, End of Year 2005					
AAD	197,100,000	gal. per year			
MMD	26,815,000	gal. per month			
MMD	865,000	gpd	601 gpm	213 gpcd	500 gpd/EDU
ADD	540,000	gpd	375 gpm	133 gpcd	312 gpd/EDU
MDD	1,405,000	gpd	976 gpm	346 gpcd	812 gpd/EDU
PHD	2,700,000	gpd	1,875 gpm	665 gpcd	1,560 gpd/EDU
MDD P.F.	2.60				
MMD P.F.	1.60				
PHD P.F.	5.00				

4.1.3 Water Consumption (Sales)

Water sold in Tri-City for the 12-month period from September 2004 through August 2005 totaled 146,965,273 gallons. For this period, the average monthly sales total is 12,247,106 gallons per month or 402,645 gpd. Of this total, 90% was residential use. The largest use sector is single-family dwellings (113.6 mg), followed by mobile home parks (16.5 mg), schools (3.7 mg), and small business commercial (3.4 mg). With a population of 4,059 water sales per capita is 99 gallons per person per day (gpcd). The average use per residential account is 7,075 gallons per month (7,075 gallons per month per EDU).

Table 4.1.3-1 – Water Sales

Type	EDU	Sales (gal.)	% Sales
Residential	1,561	132,532,312	90.2%
Restaurants	29	2,425,100	1.7%
Commercial	73	6,188,883	4.2%
Public	6	493,270	0.3%
Industrial	19	1,613,600	1.1%
Schools	44	3,712,108	2.5%
	1,731	146,965,273	100.0%

The water account in Tri-City with the largest use is the Tri-City Mobile Estates mobile home park with 80 units using 6.35 million gallons of water for the 12-month period from 9/1/2004 to 8/31/2005. Second and third largest are the Wooten Manor Mobile Park (4.61 mg) and the Horizon Mobile Home Park (2.62 mg). Of the non-residential users, the largest use is at Ireland Inc. with 2.53 million gallons of water for the one year period.

Table 4.1.3-2 – Top 15 Users

Rank	User Account Name	gallons/year
1	Tri-City Mobile Estates	6,353,850
2	Wooten Manor Mobile Park	4,612,100
3	Horizon Mobile Home Park	2,617,300
4	Ireland, Inc.	2,531,000
5	Cameron Mobile Park	2,300,100
6	South Umpqua Ballfield	2,023,100
7	Winco	1,613,600
8	South Umpqua High School	1,404,908
9	Tri-City Elementary	1,145,500
10	McDonalds	1,058,700
11	Riverside Trailer Park	645,100
12	Tri View Apartments	516,629
13	Chevron/A&W	372,690
14	Gary Fredericks Apartments	370,773
15	Country Club Tavern	341,450

Table 4.1.3-3 – Top 15 Non-Residential Users

Rank	User Account Name	gallons/year
1	Ireland, Inc.	2,531,000
2	South Umpqua Ballfield	2,023,100
3	Winco	1,613,600
4	South Umpqua High School	1,404,908
5	Tri-City Elementary	1,145,500
6	McDonalds	1,058,700
7	Chevron/A&W	372,690
8	Country Club Tavern	341,450
9	Douglas Co. Public Works	313,600
10	Shop Smart (Price Less)	310,540
11	Ireland	296,490
12	Pizza Palace	289,280
13	Broaster House	275,830
14	M.C. Body Shop	238,550
15	Suzie's	212,950

4.1.4 Unaccounted Water

The difference between water produced at the treatment plant and pumped into the system and the amount of water metered through water meters (water sold) is unaccounted water. The difference can be attributed to leakage in the distribution system, inaccuracies in water meters, water used for system flushing and fire fighting, and other public non-metered use.

System flushing through fire hydrants and blow-offs is conducted periodically but water quantities used for this are not known. It is also possible that many of the old water meters are reading inaccurately. Old meters tend to read lower quantities than are actually used causing apparent water loss to increase. It is also possible that the distribution system has many small leaks that are not readily visible.

Table 4.1.4-1 – Monthly Unaccounted Water Percentage

Month	Raw Water (gallons)	Plant Use (gallons)	Finish Flow (gallons)	Water Sales (gallons)	Water Loss (gallons)	% Loss
Dec-04	13,951,800	788,400	13,163,400	9,635,468	3,527,932	26.8%
Jan-05	12,059,000	659,000	11,400,000	10,398,222	1,001,778	8.8%
Feb-05	10,806,200	783,900	10,022,300	7,463,104	2,559,196	25.5%
Mar-05	12,564,800	1,493,500	11,071,300	8,483,966	2,587,334	23.4%
Apr-05	10,890,700	700,900	10,189,800	9,516,446	673,354	6.6%
May-05	12,010,600	545,700	11,464,900	10,140,028	1,324,872	11.6%
Jun-05	15,859,700	1,239,000	14,620,700	11,151,543	3,469,157	23.7%
Jul-05	24,076,300	1,096,200	22,980,100	16,216,350	6,763,750	29.4%
Aug-05	25,406,300	1,436,900	23,969,400	23,787,080	182,320	0.8%
Sep-05	18,123,000	1,442,100	16,680,900	15,598,680	1,082,220	6.5%
Average	15,574,840	1,018,560	14,556,280	12,239,089	2,317,191	16.3%

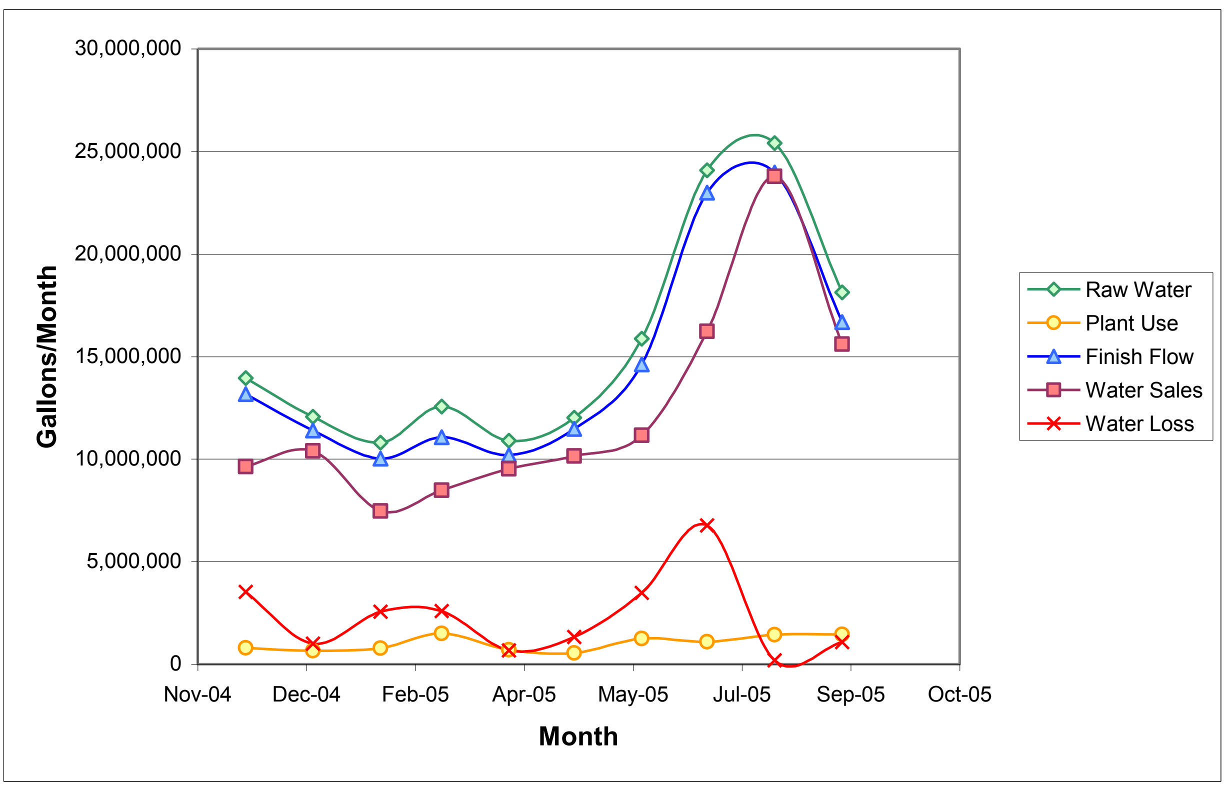


Figure 4.1.4-1 – Monthly Water Production, Sales, and Unaccounted Water

Unaccounted water over the 10-month period from December 2004 to September 2005 ranged from nearly zero to almost 30%. The average unaccounted water for this period totals 16.3% of the water pumped from the treatment plant. For the period from 9/1/2004 to 8/31/2005, a total of 169,561,200 gallons of water was pumped into the distribution system, a total of 146,965,273 gallons was sold, and 22,595,927 gallons was unaccounted for. For this one-year period, unaccounted water totaled 13.3% of total water pumped from the plant.

These figures indicated that the Tri-City water distribution system has a relatively low amount of leakage. Systems often have average unaccounted water exceeding 20%. Generally, systems with excessive water loss should strive to reduce unaccounted water to less than 15%. For systems with less than 15% loss, or for which achieving 15% loss is relatively easy, efforts to reduce unaccounted water to 10% or less should be made. The one-year period analyzed shows that unaccounted water in Tri-City is 13.3%. In the 1994 Water Master Plan, unaccounted water was estimated at 20%. This figure included water that was used at the plant. If the same 8.9% plant use that occurs today is applied, unaccounted water in 1994 was around 11%. The average amount of unaccounted water in Tri-City is considered acceptable.

Even though average values for unaccounted water appear acceptable, the monthly value varies widely and sometimes approaches 30%. It is suspected that this is due, in part, to variations in the day meter readings occur. For the most accurate accounting of water, all water meters should be read as quickly as possible at the end of the month to collect a true 1 month quantity. Each month, meter readings should occur at the same time. When meter readings are spread out over several days, or do not coincide with the month to month records at the plant, it becomes difficult to compare metered values with plant output values. Plant production is recorded for each month from the first day to the last day of the month. Effort should be made to record individual water meter totals for this same time period.

4.2 <u>Projected Water Demand</u>

4.2.1 Normal Water Demand In Oregon

Typical sales quantities guidelines are found in the “Guidelines for the Preparation of Planning Documents for Developing Community Water System Projects”, prepared by various State of Oregon, Federal and non-profit organizations. This guide states that normal water sales should be based on 100 gpcd or 250 gpd/EDU, or 7500 gallons per month per EDU. These numbers are for water sales rather than actual demand and water demand is always higher than actual water sales quantities. As discussed previously water sales in Tri-City for the one-year period from 8/2004 to 9/2005 was 99 gpcd and each residential account used an average of 7,075 gallons per month.

Per capita water use for Oregon is documented by the U.S. Department of the Interior in the 1995 U.S. Geological Survey - Circular 1200. According to the study, the average per capita water demand for Oregon is 235 gallons per capita day (gpcd) including domestic, commercial, industrial, and public use and loss. Of the total 235 gpcd, 53% is domestic use (124.6 gpcd), 14% is commercial (32.9 gpcd), 17% is industrial (40.0 gpcd), and 16% is public use and loss (37.6 gpcd). Typical values are as follows: ADD of 235 gpcd, MMD of 366 gpcd, a PWD of 520 gpcd, a MDD of 588 gpcd (2.5 times the ADD), and a PHD of 1,175 gpcd (5 times the ADD). These values, however, are state averages and values in a specific community can vary considerably.

4.2.2 Tri-City Current Water Demand Values

In Tri-City, average per capita water demand is 133 gpcd (see Table 4.1.2-1) based on a population of 4,059 persons and an average daily system demand of 540,000 gallons. At this time with 90% of water sales being to residential customers, other uses such as commercial and industrial have little impact to the total system gpcd numbers.

Table 4.2.2-1 – Current Water Demand Values

Demand	Gallons per Day	gpd/EDU	gpcd
Average Day, ADD	540,000	312	133
Max. Month, MMD	865,000	500	213
Max. Day, MDD	1,405,000	812	346
Peak Hour, PHD	2,700,000	1,560	665

4.2.3 Tri-City Projected Water Demand Values

Water demands are projected into the future using the current unit demand values (gpcd and gpd/EDU) along with projected population and EDU estimates. The goal of projecting future water demand is not to build larger facilities to accommodate excessive water consumption, but rather to evaluate the capability of existing improvements and to size new facilities for reasonable demand rates. Current unit water consumption in Tri-City is reasonable and these values will be used for future users as well.

The current population of 4,059 is projected to increase to 6,651 over the next 20 years. This is equivalent to a 2.5% average annual growth rate. If the mixture of commercial, industrial, public and residential use remains fairly close to what exists today, the number of EDUs will increase at the same rate as population. It is possible that EDUs could grow faster than population if significant industrial users begin to occupy available industrial land. It is also possible that population growth may not maintain an average 2.5% growth for the entire 20-year period. For these reasons, the number of EDUs in the system instead of population at any time is a better indicator of water needs at that time.

Table 4.2.3-1 – Projected Water Demand Values

Year	Population	EDU	ADD (gpd)	MMD (gpd)	MDD (gpd)	PHD (gpd)
2005	4,059	1,731	540,000	865,000	1,405,000	2,700,000
2006	4,160	1,774	553,500	886,625	1,440,125	2,767,500
2007	4,264	1,819	567,338	908,791	1,476,128	2,836,688
2008	4,371	1,864	581,521	931,510	1,513,031	2,907,605
2009	4,480	1,911	596,059	954,798	1,550,857	2,980,295
2010	4,592	1,958	610,960	978,668	1,589,629	3,054,802
2011	4,707	2,007	626,234	1,003,135	1,629,369	3,131,172
2012	4,825	2,058	641,890	1,028,213	1,670,103	3,209,452
2013	4,945	2,109	657,938	1,053,919	1,711,856	3,289,688
2014	5,069	2,162	674,386	1,080,266	1,754,652	3,371,930
2015	5,196	2,216	691,246	1,107,273	1,798,519	3,456,228
2016	5,326	2,271	708,527	1,134,955	1,843,482	3,542,634
2017	5,459	2,328	726,240	1,163,329	1,889,569	3,631,200
2018	5,595	2,386	744,396	1,192,412	1,936,808	3,721,980
2019	5,735	2,446	763,006	1,222,222	1,985,228	3,815,029
2020	5,879	2,507	782,081	1,252,778	2,034,859	3,910,405
2021	6,026	2,570	801,633	1,284,097	2,085,730	4,008,165
2022	6,176	2,634	821,674	1,316,200	2,137,874	4,108,369
2023	6,331	2,700	842,216	1,349,105	2,191,320	4,211,079
2024	6,489	2,767	863,271	1,382,832	2,246,104	4,316,356
2025	**6,651**	**2,836**	**884,853**	**1,417,403**	**2,302,256**	**4,424,264**

The current and projected MDD expressed in gallons per minute is 976 gpm and 1599 gpm respectively.

Design Criteria & Service Goals

Section 5

5.1 Design Life of Improvements

The design life of a water system component is sometimes referred to as its useful life or service life. The selection of a design life is a matter of judgment based on such factors as the type and intensity of use, type and quality of materials used in construction, and the quality of workmanship during installation. The estimated and actual design life for any particular component may vary depending on the above factors. The establishment of a design life provides a realistic projection of service upon which to base an economic analysis of new capital improvements.

As discussed in Section 1, the planning period for this Water System Master Plan is 20 years ending in the year 2025. The planning period is the time frame during which the recommended water system is expected to provide sufficient capacity to meet the needs of all anticipated users. The required system capacity is based on population, EDUs, water demand projections, and land use considerations.

The planning period for a water system and the design life for its components may not be identical. For example, a properly maintained steel storage tank may have a design life of 60 years, but the projected fire flow and consumptive water demand for a planning period of 20 years determine its size. At the end of the initial 20-year planning period, water demand may be such that an additional storage tank is required; however, the existing tank with a design life of 60 years would still be useful and remain in service for another 40 years. The typical design life for system components are discussed below.

5.1.1 Pumping Equipment and Structures

Major structures and buildings should have a design life of approximately 50 years. Pumps and equipment usually have a useful life of about 15 to 20 years. The useful life of some equipment can be extended, when properly maintained, if additional capacity is not required. Flowmeters typically have a design life of 10 to 15 years. Valves usually need to be replaced after 15 to 20 years of use.

5.1.2 Treated Water Transmission and Distribution Piping

Water transmission and distribution piping should easily have a useful life of 40 to 60 years if quality materials and workmanship are incorporated into the construction and the pipes are adequately sized. Steel piping used in the 1950’s and 60’s that has been buried, commonly exhibits significant corrosion and leakage within 30 years. Cement mortar lined ductile iron piping can last up to 100 years when properly designed and installed. PVC pipe manufacturer’s claim a 100-year service life for pipe as well.

5.1.3 Treated Water Storage

Distribution storage tanks should have a design life of 60 years (painted steel construction) to 80 years (concrete construction). Steel tanks with a glass-fused coating can have a design life similar to concrete construction. Actual design life will depend on the quality of materials, the workmanship during installation, and the timely administration of maintenance activities. Several practices, such as the use of cathodic protection, regular cleaning and frequent painting can extend or assure the service life of steel reservoirs. The life of steel tanks is greatly reduced if not repainted periodically as needed.

5.2 Sizing and Capacity Criteria and Goals

The 20-year projected water demands are used to size improvements. Various components of the system demand are used for sizing different improvements. Methods and demands used are discussed below.

5.2.1 Water Supply

Water supply should at minimum be sufficient to meet the projected maximum daily demand (MDD). If possible, raw water availability should meet the ultimate build out needs in a small community, especially when surface water rights are the only option. Currently the MDD is 1.4 mgd or 2.17 cfs. At the end of the planning period, the projected MDD is 2.3 mgd or 3.56 cfs. All of the water available to Tri-City above the current water rights for the South Umpqua River is held in the Galesville Storage Reservoir upstream. The Oregon Water Resource Department indicates that the vast majority of municipal water in Galesville is still available for future use. For this reason, it is not prudent to attempt to purchase and secure long-range water rights for ultimate build-out of the community. Tri-City has water permits/rights from the South Umpqua River totaling 4.87 cfs. Some of these rights are still in the permit stage and have not been certificated. The goal of the Authority for this planning period is to finalize all permits such that the full 4.87 cfs is certificated and converted to formal water rights. The water availability in Galesville should be monitored over time and additional storage rights purchased if availability becomes low or starts to deplete quickly.

Supply Goal – Minimum of 4.87 cubic feet per second (cfs)

5.2.2 Water Treatment

Water treatment plant equipment and components such as intake pumps, discharge pumps, and clearwells are typically sized to provide for the 20-year MDD. The actual plant capacity should be increased slightly to allow for the maximum daily demand to be met without requiring the plant to run 24 hours per day. This is required since the plant cannot typically run 24 hours per day since filter backwashing and other down-times are needed to produce safe drinking water. The goal is to produce the projected MDD with a maximum of 22 hours per day run-time. The projected design MDD is 2.3 mgd.

Treatment Capacity Goal – 20-year MDD in 22 hours plant run time, 2.5 mgd

5.2.3 Treated Water Storage

Total storage capacity must include reserve storage for fire suppression, equalization storage, and emergency storage. In larger communities it is common to provide storage capacity equal to the sum of equalization storage plus the larger of fire storage or emergency storage. In small communities it is recommended that total storage be the sum of fire plus equalization plus emergency storage. This is considered prudent since it is possible for fire danger to increase during water emergencies, such as power failures when alternatives sources of heating and cooking might be used.

Equalization storage is typically set at 20-25% of the MDD to balance out the difference between peak demand and supply capacity. When peak hour flows are known, equalization storage is the difference between the MDD and PHD for a duration of 8 hours [PHD-MDD x 8 hrs.].

Emergency storage is required to protect against a total loss of water supply such as would occur with a broken transmission line, an electrical outage, equipment breakdown, or source contamination. Emergency storage should be an adequate volume to supply the system's average daily demand for the duration of a possible emergency. For small systems, emergency storage should be equal to one maximum day of demand or 2.5 to 3 times the average day demand.

Fire reserve storage is needed to supply fire flow throughout the water system to fight a major fire. The fire reserve storage is based on the maximum flow and duration of flow required to confine a major fire. The guidelines published in "Fire Suppression Rating Schedule" by the Insurance Services Office (ISO) are typically used to determine the required fire flow and fire reserve storage. Generally, fire flows of 1,000 to 1,500 gpm are sufficient for one or two family dwellings not exceeding two stories in height. Commercial, industrial and institutional buildings require higher flows. Determination of these flows is unique to each building under consideration and involves detailed surveys of construction (type and area), occupancy (combustibility), exposure (construction type, distance, length/height of wall) and communications (openings).

The ISO also classifies fire protection capabilities on a numerical basis, called the Public Protection Classification (PPC) with Class 1 representing exemplary protection and Class 10 indicating less than minimum protection. This classification is used within the insurance industry for various purposes. The Public Protection Classification is determined from a complex analysis of the City's capabilities to receive and handle fire alarms, of the strength of the fire department, and of the adequacy of the water supply system. Analysis of the water supply system is further divided into equal parts of: 1) supply capabilities, 2) hydrant size, type, and installation, and 3) inspection and condition of hydrants. For a PPC Class 8 rating or better, fire storage should be adequate to support needed fire flows as follows: 2 hours when less than 3,000 gpm is needed, 3 hours when flows of 3,000 to 3,500 gpm are provided, or for 4 hours when flows greater than 3,500 gpm are needed.

For typical residential areas, the minimum recommended fire storage is 180,000 gallons to provide a flow up to 1,500 gpm for 2 hours. When significant non-residential structures exist with fire fighting requirements greater than typical residential requirements, additional fire protection storage can be justified.

In Tri-City there are several significant structures (i.e. South Umpqua High School, Tri-City Elementary Schools, Winco, etc.) which could justify the need for additional fire storage beyond the minimums recommended for residential areas. A fire flow of 3,000 gpm for 3 hours will consume a volume of 540,000 gallons.

Another important design parameter for reservoirs is elevation. Efforts should be made to locate all reservoirs at the same elevation when possible within a pressure zone. As a consistent water surface is maintained in all reservoirs, the need for altitude valves, PRV's, booster pumps, and other control devices may be eliminated. Distribution reservoirs should also be located at an elevation that maintains adequate water pressure throughout the system; sufficient water pressures at high elevations and reasonable pressures at lower elevations. The ideal pressure range in the system should be within the range of 40 to 80 psi.

For subdivisions at higher elevations than allowed within the main pressure zone, storage tanks should be required when possible rather than hydropneumatic tank booster pump stations. Tank size needs to be determined on a case-by-case basis as part of the design review. Fire pumps with a capacity of at least 1,000 gpm should be provided when a storage tank is not possible. Minimum tank size should be 120,000 gallons fire storage (1,000 gpm for 2 hours) plus 1 times the MDD per EDU.

In Tri-City, the MDD (see Section 4) is equal to 2.6 times the ADD. Therefore setting emergency storage to one maximum day demand is equivalent to 2.6 average days of storage. Equalization storage is set to 25% of the MDD since the PHD is not measured. Fire Storage should be 3,000 gpm for 3 hours.

Storage Capacity Goal – 1.25 x $MDD_{20\text{-}year}$ + 540,000 fire storage

5.2.4 Distribution System

Distribution mains are typically sized for fire flow and 20-year population demand, or fire flow and saturation development demand. The mains should be at least six inches in diameter to provide minimum fire flow capacity. All pipelines should be large enough to sustain a minimum line pressure of 25 psi during peak flow periods. The State of Oregon requires a water distribution system be designed and installed to maintain a pressure of at least 20 psi at all service connections (at the property line) at all times. The size and layout of pipelines must be adequate to handle peak hourly flows, and to provide fire flows during periods of peak demand while maintaining minimum system pressures.

In addition to the above design criteria, the following guidelines are recommended for the design of water distribution systems. In all cases a hydraulic analysis using peak domestic plus fire flows may result in pipes larger than the minimums stated below:

- Six-inch (6") diameter lines - minimum sized lateral water main for gridiron (looped) system and short dead-end mains never to be extended.
- Eight-inch (8") diameter lines - minimum size for permanently dead-ended mains supplying fire hydrants and for minor trunk mains.
- Ten-inch diameter (10”) and larger - as required for trunk (feeder) mains based on hydraulic analysis.

The distribution system lateral mains should be looped whenever possible. A lateral main is defined as a main not exceeding eight-inches in diameter, which is installed to provide water service and fire protection for a local area including the immediately adjacent property. The normal size of lateral mains for single-family residential areas is six-inches in diameter. However, eight-inch or larger lateral mains may be required to meet both the domestic and fire protection needs of an area.

The installation of permanent dead-end mains and dependence of relatively large areas on a single main should be avoided. For the placement of a fire hydrant on a permanently dead-ended main, the minimum size of such laterals should be eight inches in diameter. Six-inch diameter mains may be used for a stub-out not exceeding 500 feet in length supplying a single fire hydrant not on a public street and for internal fire protection. On new construction, the minimum size lateral main for supplying fire hydrants within public ways should be six-inches provided six-inch mains are looped.

A computer model of the distribution system will be made. The model will use actual pipe sizes and materials as well as system pipe junction elevations and storage tank elevations. The system will be checked for ability to provide fire flows during times when the system demand is at the current PHD to determine existing deficiencies. The projected PHD + fire flows will be used to size any needed improvements. System pressure must remain above 20 psi at all conditions.

Distribution Capacity Goal – Projected PHD + fire flow with at least 20 psi residual pressure

5.2.5 Fire Flows

The requirements for fire fighting at any point can vary between 500 gpm to 12,000 gpm for a single fire. Multiple fires will place a greater demand on the distribution system. A municipality must continue to serve its domestic, commercial, and industrial customers during a fire. The Insurance Services Office (ISO) recommends that the fire system be able to operate with the remainder of the potable water system operating at the MDD. For one- and two-family dwellings not exceeding two stories in height, ISO uses the following needed fire flows:

Distance Between Buildings	Needed Fire Flow
More than 100’	500 gpm
31 to 100’	750 gpm
11 to 30’	1000 gpm
10’ or less	1500 gpm

- When building has a wood-shingle roof covering that ISO determines to contribute to spreading fires, 500 gpm is added to the needed fire flow

- For other types of habitational buildings, the maximum fire flow is 3,500 gpm

For other types of structures, ISO has a formula to determine the needed fire flow which can result in higher flows needed compared to residential structures. Most insurance requirements will be met if the flow rate can be maintained for T hours, where T is the flow rate in 1000's of gpm, with a maximum of 10 hours.

The Tri-City Joint Water & Sanitary Authority’s goal is to provide at least 1,000 gpm to each fire hydrant in the system with a flow of at least 3,000 gpm available at the large public and commercial buildings.

Fire hydrants should be spaced at a maximum distance of 500 feet. They are ordinarily located at street corners where use from four directions is possible.

Fire Flow Capacity Goals – 1000 gpm minimum with 3000 gpm per hydrant at large structures

5.3 Basis for Cost Estimates

The cost estimates presented in this Plan will typically include four components: construction cost, engineering cost, contingency, and legal and administrative costs. Each of the cost components is discussed in this section. The estimates presented herein are preliminary and are based on the level and detail of planning presented in this Study. Construction costs are based on competitive bidding as public works projects. As projects proceed and as site-specific information becomes available, the estimates may require updating.

5.3.1 Construction Costs

The estimated construction costs in this Plan are based on actual construction bidding results from similar work, published cost guides, and other construction cost experience. Reference was made to the as-built drawings, and system maps of the existing facilities to determine construction quantities, elevations of the reservoirs and major components, and locations of distribution lines. Where required, estimates will be based on preliminary layouts of the proposed improvements.

Future changes in the cost of labor, equipment, and materials may justify comparable changes in the cost estimates presented herein. For this reason, common engineering practices usually tie the cost estimates to a particular index that varies in proportion to long-term changes in the national economy. The Engineering News Record (ENR) construction cost index (CCI) is most commonly used. This index is based on the value of 100 for the year 1913. Average yearly values for the past 15 years are summarized in Table 5.3.1-1.

Table 5.3.1-1 – ENR Index 1990 to 2005

YEAR	INDEX	% CHANGE/YR
1990	4732	2.54
1991	4835	2.18
1992	4985	3.10
1993	5210	4.51
1994	5408	3.80
1995	5471	1.16
1996	5620	2.72
1997	5826	3.67
1998	5920	1.61
1999	6059	2.35
2000	6221	2.67
2001	6343	1.96
2002	6538	3.07
2003	6694	2.39
2004	7115	6.29
2005	7444	3.84
	Average Annual Change =	3.04%
	Average since 1980	3.58%
	Average since 2000	3.50%

Cost estimates presented in this Plan are based on 2005 dollars with an ENR CCI of 7444. For construction performed in later years, costs should be projected based on the then current year ENR Index using the following method:

Updated Cost = Plan Cost Estimate x (current ENR CCI / 7444)

5.3.2 Contingencies

A contingency factor equal to approximately fifteen percent (15%) of the estimated construction cost has been added to the planning estimated in this Plan. In recognition that the cost estimates presented are based on conceptual planning, allowances must be made for variations in final quantities, bidding market conditions, adverse construction conditions, unanticipated specialized investigation and studies, and other difficulties which cannot be foreseen at this time but may tend to increase final costs. Upon final design completion of any project, the contingency can be reduced to 10%. A contingency of at least 10% should always be maintained going into a construction project to allow for variances in quantities of materials and unforeseen conditions.

5.3.3 Engineering

The cost of engineering services for major projects typically include special investigations, predesign reports, surveying, foundation exploration, preparation of contract drawings and specifications, bidding services, construction management, inspection, construction staking, start-up services, and the preparation of operation and maintenance manuals. Depending on the size and type of project, engineering costs may range from 18 to 25% of the contract cost when all of the above services are provided. The lower percentage applies to large projects without complicated mechanical systems. The higher percentage applies to small or complicated projects. Engineering costs for design and construction services presented in this Plan are based on 20% of the estimated construction cost.

5.3.4 Legal and Administrative

An allowance of four percent (4%) of construction cost has been added for legal and administrative services. This allowance is intended to include internal project planning and budgeting, grant administration, liaison, interest on interim loan financing, legal services, review fees, legal advertising, and other related expenses associated with the project that could be incurred.

5.3.5 Land Acquisition

Some projects may require the acquisition of additional right-of-way or property for construction of a specific improvement. The need and cost for such expenditures is difficult to predict and must be reviewed as a project is developed. Effort was made to include costs for land acquisition, where expected, within the cost estimates included in this Plan.

Existing Water System

6.1 Raw Water Supply

The source of water supply for Tri-City is the South Umpqua River. The South Umpqua River is a tributary of the Umpqua River. The South Umpqua is approximately 95 miles long and drains a portion of the Cascade Range through remote canyons in the upper reaches, ending in the South Umpqua Valley near Roseburg. It rises in the high Cascades north of Fish Mountain, formed by the confluence of two short forks in eastern Douglas County approximately 20 miles northwest of Crater Lake. It flows generally southwest through a remote canyon in the Umpqua National Forest to Tiller, then west past Milo. It emerges into the South Umpqua Valley at Canyonville, passing under Interstate 5 and flowing north along the highway past Tri-City, Myrtle Creek, and Roseburg. It joins the North Umpqua from the south to form the Umpqua approximately 6 miles northwest of Roseburg. It receives flows from Cow Creek from the south approximately 5 miles southwest of Tri-City. Flows in Cow Creek are regulated and storage is provided by the Galesville Storage Reservoir near Azalea. The 167 foot high dam stores 42,225 acre-feet of water, of which 4,450 acre-feet is designated for municipal purposes.

A concrete intake structure is located in the river channel with gravity flow into a buried concrete wetwell at the bank. Water is pumped from the wetwell to the nearby treatment plant. The intake structure is located approximately at river mile 153.75. The oldest water use permits list the point of diversion (POD) as 2030 feet south and 500 feet east of the northwest corner of Section 5 in Township 30 S, Range 5 West. The newer permits list the POD at 2400’ S and 550’ E of the NW corner of Section 5. Based on the Assessors Maps the POD scales to 1930’ S and 470’ E of the NW corner of Section 5. It is not know which distances are most accurate.

6.1.1 Water Quantity

No stream gauging station is located near Tri-City that has data available to represent stream flows accurately at this location. According to the DEQ Source Water Assessment for Tri-City, the watershed area is approximately 1,271 square miles. An estimation of stream flows may be obtained by using data from the gauging station on the S. Umpqua River near Days Creek combined with data obtained from the gauging station on Cow Creek near Riddle. Cow Creek joins the S. Umpqua a few miles upstream from Tri-City and no other major streams join before Tri-City. For these two stations, simultaneous data is available for the period from 3/1/1975 to 10/31/1992. Based on this summation of the two gauging stations, the average flow in the South Umpqua at Tri-City is 1,753 cfs. The minimum and maximum recorded streamflows for the period are 42 cfs and 52,900 cfs respectively. Over this 17 year period, flows dropped below 60 cfs a total of 41 days. Of these 41 days, 30 occurred in August and September of 1977, and 11 occurred in September of 1981.

Longer term data is available for the gauging station on the South Umpqua River at Tiller, a little farther upstream of the Days Creek station. When these records are combined with the data for the station on Cow Creek near Riddle, a period of record from 1/1/1970 to 9/30/2004 can be used. Data for the period from 10/1/1993 to 9/30/1994 is missing for Cow Creek. For this data the average streamflow is 1,844 cfs. The minimum and maximum flows are 44 cfs and 56,700 cfs respectively. For this 33 year data period, flows dropped below 60 cfs a total of 97 days. Days less than 60 cfs include 25 days in 1973, 27 days in 1977, 10 days in 1981, 23 days in 2001, and 12 days in 2002. All of these low flow periods were in August and September.

When flows drop below the 1958 instream water right of 60 cfs, rights with a junior priority date are required to be off as directed by the water master. The 60 cfs instream water right is as measured at the mouth of the S. Umpqua and only includes natural stream flow (does not include excess water that may be released from Galesville Storage Reservoir). The water use restriction occurs fairly regularly and sometimes as much as every other year. At these times, Tri-City must use water from the Galesville Storage Reservoir to supplement the pre-1958 water rights.

6.1.2 Water Quality

Raw source water quality parameters measured between 1/1/1999 and 6/30/2004 at the water treatment plant were collected and entered into a spreadsheet to determine the average, maximum, and minimum values for temperature, pH, and turbidity. The data indicates that the South Umpqua River at the intake location is of average water quality with occasional high temperatures, high pH, and high turbidity. Raw water total alkalinity is generally between 40 and 60 mg/L. Raw water total organic carbon (TOC) as measured in 21 samples taken from 1/9/04 to 10/10/05 ranges from 2.82 to 1.22 mg/L with an average of 1.89 mg/L.

Table 6.1.2-1 – S. Umpqua Water Quality Parameters

	Average	Maximum	Minimum
Temperature, °C (°F)	12.4 (54.3)	27.9 (82.2)	4.3 (39.7)
pH	7.7	9.25	6.8
Turbidity (NTU)	12.9	295	0.07

Detailed documentation of water quality concerns in the South Umpqua River basin is available in the US Geological Survey (USGS) Water-Resources Investigations Report 96-4082. Temperature and pH limit water quality in the South Umpqua River near Tri-City in summer months with pH exceeding 8.5 and temperatures exceeding 75°F. Portions of the South Umpqua River are listed by DEQ in the 303(d) list as quality limited for pH, temperature, dissolved oxygen, bacteria, algae, and sedimentation. Excessive growth of periphytic algae in the South Umpqua River becomes a problem each summer during low-flow periods. This growth results from excessive inputs of nutrients, primarily from point sources such as wastewater treatment facilities, pump station overflows, storm water outfalls, cropland runoff, etc. Eutrophication is the process of enrichment of water with nutrients, mainly nitrogen and phosphorous compounds, which results in excessive growth of algae and nuisance aquatic plants. It increases the amount of organic matter in the water and also increases pollution as this matter grows and then decays. Employing the process of photosynthesis for growth, algae and aquatic plants consume carbon dioxide (thus raising pH) and produce an overabundance of oxygen. At night the algae and plants respire, depleting available dissolved oxygen.

6.1.3 Water Rights

Tri-City has municipal water use permits allowing for the withdrawal of surface water from the South Umpqua River. The oldest permit was established in 1952 and the newest permit was established in 1979. For all permits, the point of diversion (POD) lies within Township 30 S, Range 5 W. Only two of the permits have been certificated at this point (S21179 and S24446). The remainder are still in the permit stage and eventually require perfection (Claim of Beneficial Use submitted to State) for conversion to certificated water rights.

Permit S24600 was originally for irrigation. Tri-City applied and received approval to change the use to municipal along with changes in the POD and place of use. The original certificates were cancelled and new permits issued to Tri-City. It is important to note that the application approval to change the place of use, POD, and character of use (irrigation to municipal) requires Tri-City to install and maintain a fish screen or fish bypass device with plans approved by ODFW. The Claim of Beneficial Use for these permits (S24600) was submitted as required on 9/26/2001 but no new certificates have yet been issued.

A Claim of Beneficial Use for permit S44336 was submitted on 4/12/2004 however no final certificate has yet been issued.

Table 6.1.3-1 – S. Umpqua Municipal Water Use Permits for Tri-City

Permit	Certificate	Priority Date	Rate (cfs)	POD	Comments
S21179	52975	3/13/1952	0.125	2030' S, 500' E of NW corner of Section 5	25.0 acre-feet maximum during irrigation season
S24446	30263	8/13/1956	1.00	SW¼, NW¼ Sec. 5 proj. from DLC 39	
S24600		9/19/1956	0.25	2030' S, 500' E of NW corner of Section 5	50 acre-feet maximum during irrigation season
S24600		9/19/1956	0.07	2030' S, 500' E of NW corner of Section 5	12.5 acre-feet maximum during irrigation season
S40699		10/24/1973	3.0	2400' S, 550' E of NW corner of Section 5	Extension granted to perfect
S44336		4/19/1979	0.425	2400' S, 550' E of NW corner of Section 5	
U&I 94-2	Galesville Storage Use Contract, 95 acre-feet (30,956,010 gallons)				

- Pre-1958 Water Rights = 1.445 cfs (648.6 gpm; 933,928 gpd)
- Post-1958 Water Rights = 3.425 cfs (1537.2 gpm; 2,213,635 gpd)
- Total Tri-City Water Rights = 4.87 cfs (2185.8 gpm; 3,147,563 gpd)

The acre-feet limits shown for permits S21179 and S24600 (both) should never be an issue since the acre-ft limit indicated is greater than the withdrawal rate allowed over a 24 hour period assuming a 90-day irrigation season. The limits could come into effect if an irrigation season greater than 90 days occurs.

Instream water rights of 60 cfs were established by the State on the South Umpqua River in 1958 to protect fish and wildlife. During low-flow periods, the Water Resources Department can restrict withdrawals from the river to maintain the instream water right. When this occurs, all use of water permitted after 1958 ceases and Tri-City must only use their pre-1958 water rights and supplement flow with water from the Galesville Reservoir. As discussed in Section 6.1.1, flows in the river have dropped below the 60 cfs water right quantity several times in the past and will continue to do so. The most recent restriction was from 8/16/2004 to 9/12/2004.

Currently, the plant withdrawal from the river is approximately 1050 gpm. During water restriction periods, 649 gpm of this is supplied with pre-1958 surface water permits and 400 gpm is supplemental flow from Galesville Reservoir. By using the 95 acre-feet of Galesville water at a rate of 400 gpm for 24 hours per day, this use could continue for 53 days before the 95 acre-feet is depleted. With the plant operating for 16 hours per day, the use could continue for 80 days.

Without instream water rights restrictions, the water use permits held by Tri-City are adequate for the 20-year planning period and beyond. The 20-year projected plant capacity needed is 2.5 mgd or an instantaneous flow of 1,740 gpm. During water restriction periods, 1,091 gpm would be needed from the Galesville Reservoir. Operating at 8 hours per day, the 95 acre-feet of Galesville water would last 59 days. At the end of the planning period, approximately 14 hours of plant run time per day is anticipated to meet maximum monthly demands (MMD) as would occur in summer months. Operating at 14 hours per day, the 95 acre-feet of Galesville water would last 33 days. It is possible for the restriction period to last longer than 33 days. As a result, Tri-City will most likely need additional water from the Galesville Storage Reservoir above the 95 acre-ft currently contracted.

6.1.4 River Intake Structure

The existing water intake is a perforated circular concrete caisson structure constructed directly on the river bottom approximately 15 feet from the bank. A concrete base for the circular section was poured on the river bottom (assumed to be placed on rock), and a concrete cap 8 feet in diameter covers the structure. The intake is shown as a pre-existing structure in the 1978 plans for the original package plant installation. Holes in the structure collect river water which then flows by gravity to the pump station wetwell through approximately 50 feet of 15-inch concrete pipe. Details for the structure discussed in this plan are based on "record drawings" produced in 2001 for the water treatment plant upgrade. The concrete structure has an inside diameter of 6'-8" and is perforated with 48, 2-inch diameter holes. The elevation at the top of the cap is 588.69 feet, and the elevation at the bottom invert is 582.12 feet giving a height of 6.57 feet. Six rows of 2-inch holes (with 8 holes each row) are located on 8-inch centers vertically. The river bottom in this location is approximately 583 feet. Problems with the existing intake include, sand accumulation, lack of fish screening, and capacity deficiencies during low water conditions.

With 2-inch orifice opening near the river bottom, sand and rocks enter the intake structure and accumulate over time. The intake must be cleaned manually with buckets and shovels when the water is low. When water levels are high, the structure cannot be accessed for cleaning. Sediment build-up in the intake structure eventually causes build-up in the outlet piping to the wetwell, decreasing the capacity of the pipe. Additionally, this causes rocks and sand to enter the pump station wetwell.

Historically, Tri-City staff has needed to move rocks in the river in attempt to maintain adequate water depth at the intake during low flow periods. Equipment (dozer) was used to create a berm of rock downstream of the intake for a partial width of the river. State regulatory agencies have since prohibited Tri-City from this practice. Water level in the river has not been low enough in recent years and current staff has not had to make efforts to increase local water depth. Summer water depths are reported to be as low as 2 feet at the intake location. The previous master plan stated that the low water depth was 2.25 feet. At this low water depth, only three of the six rows of 2-inch orifice holes would be submerged and the top of the three rows would barely be submerged. Assuming the top row of the three submerged rows is 1-inch below the water surface, the capacity of the intake is approximately 1200 gpm. If the water level was to drop a few inches farther such that only 2 rows were submerged, the intake capacity would be reduced to 900 gpm with 6-inches of water over the 2nd row.

If the water depth is 2 feet, approximately 1.63 feet of head will exist over the 15-inch intake pipe to drive water to the pump station. At this condition, considering pipe friction losses and entrance and exit head losses, the capacity of the 15-inch intake pipe is 3,730 gpm.

Currently, screening is provided only be the 2-inch diameter orifices. The 2-inch openings do not meet fish screening regulations. Fish passage requirements in Oregon are set forth in Oregon Administrative Rules (OAR) 635-412. The rules require screening of intakes to protect fish. Pump intake screen requirements are set by the National Marine Fisheries Service. Generally, screen openings must be small enough and approach velocities slow enough to prevent juvenile fish from becoming entrapped. For perforated plate material and mesh or woven wire screens, the openings shall no be larger than 3/32-inch (2.38 mm). For bar/wedge screens, the openings shall not exceed 1.75 mm. Screen open area must be at least 27% of the total wetted screen area. Screen approach velocities shall not exceed 0.4 fps for active pump screens (self cleaning), and shall not exceed 0.2 fps for passive screens (manual cleaning).

The existing intake was constructed prior to fish screening regulations but would now be classified as a passive screen. The current regulations require Tri-City to modify the intake to comply with fish screening criteria. Plans for modifying the intake to add screens were prepared in 1999 along with the plant upgrade; however funding was not available to construct the intake modifications. The then planned improvements would have satisfied the fish screening regulations but may have been problematic with frequent plugging of the screens and no provisions to clean the screens.

6.1.5 Raw Water Pump Station and Transmission Piping

The raw water pump station is located approximately 50 feet from the intake structure on the river bank. The pump station consists of a buried concrete wetwell (7 foot inside diameter) and two submersible pumps. As with the intake, the pump station is shown as a pre-existing structure in the 1978 plans for the original package plant installation. Improvements to the pump station in 2000 included adding a concrete cap with access door, installing a 16-inch butterfly valve on the 15-inch inlet pipe, and installing two new pumps with guiderails for pump removal. A level transducer was also installed for pump control based on water level. The wetwell invert elevation is 577 feet. The invert of the 15-inch pipe coming from the intake is 582 feet. The top of the concrete wetwell lid is at elevation 597.50 feet. The water surface at the treatment plant which the pump station pumps to is 633.50. Worst case static lift required by the pumps is estimated to be 48.5 feet (river elevation at 585 feet, 2 foot depth).

Approximately 735 feet of 16-inch PVC pipe, and 20 feet of 12-inch pipe (at chemical injection vault) was installed in 2000 between the pump station and the treatment plant flocculation/sedimentation basin. At the sedimentation basin, the flow splits into two 12-inch pipes. Limiting velocity to 5 fps, the capacity of the 16-inch transmission pipe is 3,130 gpm.

The two pumps installed in the year 2000 were Fairbanks Morse, non-clog submersibles, 4-inch model D5433MV, 9.875-inch impellers, with 30 Hp inverter duty motors operating at 460 volts, 3 phase, 60 Hz, 1770 rpm. Each pump is rated for 950 gpm at 68 feet total dynamic head (TDH). Motors are operated with ABB variable frequency drives. The pump discharge elbows installed were 4x4-inch, followed by a 4x8 flange reducer and an 8-inch diameter riser pipe. The original check valves installed were 8-inch Flomatic ball check valves. Problems with debris sticking in the check valves occurred frequently and the check valves were replaced with 8-inch Flomatic rubber flap check valves.

During the first 4 years of operation of the pumps, frequent problems with seal failures occurred and the pumps were rebuilt on a regular basis. Additional problems occurred with water getting into the motor. Some of the problems appeared to be caused from improper rebuilds by the service company. In 2004, one of the pumps was replaced with a 4-inch Flygt model NP3171.180, type HT with a 4x8 discharge elbow, and a 25 Hp inverter duty motor operating at 460 volts, 3 phase, 1755 rpm. The pump is rated for 1000 gpm at 55 feet TDH.

Normally, the pumps alternate duty cycles and the VFDs are adjusted so that each pump supplies approximately 1000 gpm to the plant. With both pumps running at full speed, approximately 2000 gpm can be supplied to the plant.

An improvement need at the raw water pump station not related to capacity building is safer access to the wetwell. The wetwell has manhole type steps cast in the wall however the access door is centered over the wetwell making it difficult to reach the first step. This situation creates a significant falling hazard and improvements should be made to increase safety.

6.2 Water Treatment Facilities

Tri-City's initial plant was constructed in the 1950's; however none of these existing facilities remain today. Much of the existing plant was constructed in 1979. Improvements at that time included the main building (119' x 54') with filter equipment room, pump room, and chlorine storage and chlorine equipment rooms. An underground concrete clearwell was constructed below the pump room. A Keystone package plant was installed inside the building which included hydraulic flocculation, tube settler sedimentation, and 4 dual-media filters. The 1979 plant had some components designed for a 2.6 mgd capacity and others for a 1.3 mgd capacity. The intent was to allow for a future upgrade to 2.6 mgd without the need for new major equipment. In the year 2000, plant upgrades were constructed in attempt to bring the plant capacity up to a full 2.6 mgd (1805 gpm). Improvements in 2000 included a second clearwell, a new exterior flocculation/sedimentation basin, new chemical feed and storage equipment and space, new pumps, and updated control system.

The 2000 plant improvements were not successful at increasing the plant capacity to 2.6 mgd as planned. Attempting to filter the design flow of 1805 gpm results in hydraulic overloading in the piping downstream of the filters causing filter overflows. Additionally, excessive flocc carryover into the filters and air problems during filter backwashing is noted by operations staff.

6.2.1 Raw Water Chemical Addition

Chemicals used at the Tri-City water treatment plant are shown below in Table 6.2.1-1. The primary coagulant and coagulant aid are injected into the raw water stream ahead of the flocculation basins. The 16-inch raw water transmission pipe reduces to 12-inch through the chemical addition vault just upstream from the flocculation basins. The concrete vault houses the chemical injection spool followed by a 12-inch diameter static mixer. Since the static mixer was designed for 1805 gpm, it is likely not providing optimum mixing at the current flow of 1000 gpm. Filter aid polymer is added just upstream from the filters. Soda ash is fed after the clearwell as needed to raise pH for corrosion control in the distribution system. Much of the time, no pH increase is required since raw water pH tends to be rather high.

Table 6.2.1-1 – Water Treatment Chemicals, TCWTP

Use	Type	Trade Name	Typical Plant Dose	NSF/ANSI Standard 60 Maximum Dose
Coagulant	Aluminum Chlorohydrate	PAX-XL19	6-10 mg/L	250 mg/L
Coagulant Aid	polyamine polymer, cationic, solution, low MW	Superfloc C573	0.25-1.0 mg/L	20 mg/L
Filter Aid	polyacrylamide polymer, nonionic, dry, high MW	Superfloc N-300	0.005-0.01 mg/L	1 mg/L
pH Adjustment	Sodium Carbonate	Soda Ash	As needed	NA

Coagulant storage is provided in two 3000 gallon polyethylene tanks inside the chemical storage/feed room. Also included are 500 gallon tanks for coagulant aid and soda ash. Filter aid polymer is mixed in a 155 gallon polyethylene tank. All chemical feed pumps are US Filter/Wallace & Tiernan Encore 700 diaphragm metering pumps. A stream of finished water (after clearwell) is fed back to the chemical feed room to function as "carrier" water for the chemical solutions. The chemical pump discharge pipes are joined to this carrier water piping to aid in rapid transport of each chemical to the injection location and to help prevent chemical deposition in the piping. A Chemtrac streaming current monitor aids the operator in selecting the proper coagulant dosages but at this time is not used for automatic chemical feed control.

Other potential chemical additions include caustic soda as an alternative to soda ash, and sulfuric acid to lower pH as needed. Testing has been conducted at the plant using sulfuric acid to lower the pH during summer periods when high pH interferes with coagulation. The acid is commonly used for this purpose however it is dangerous to store and handle and can easily be overdosed. Pilot tests have also been conducted using a carbon dioxide feed system. This equipment generates carbonic acid which lowers the pH in a safe and controllable manner. With summer pH often exceeding 9.0, pH control is considered an essential part of treatment at the Tri-City plant. The Authority is currently planning for the installation of a permanent carbon dioxide feed system to lower the pH to between 6.0 and 7.0. The lower pH will increase disinfection effectiveness and improve turbidity removal. Once this system is in place, pH increase after the clearwell may be required on a regular basis to raise the pH back up to approximately 7.2 to 7.4. At that time, a switch to caustic soda may be more economical than soda ash.

6.2.2 Flocculation

Flocculation occurs in a separate baffled area at the front end of the exterior sedimentation basin. A divider wall running lengthwise separates the area into two flocculation trains, each with two-stage flocculation. Incoming chemically treated raw water splits in two and half of the flow enters each flocculation train. 4 vertical-paddle, motor-driven flocculators provide the two-stage flocculation. The flocculators are manufactured by Eimco and have Baldor motors (1725 rpm, 0.75 Hp, TENV motors, model VWDM3542). Motors are connected to a variable speed traction device and then to a gear reduction box to allow for speed adjustment and slow mixing. The diameter of the flocculator paddles is approximately 114 inches.

Each of the 4 flocculation areas is 12'-4" wide. The first stage is 11'-5" long and the second stage is 11'-4" long. Depth varies due to a tapered grout layer designed to help clean the bottom of the basins. Average water depth in the first stage is approximately 11'-10.5". Average water depth is the second stage is 12'-1.5". Volume of each first stage is 1,672 ft^3 or 12,508 gallons. Volume of each second stage is 1,695 ft3 or 12,678 gallons. Total flocculation volume is therefore 50,372 gallons. At a flowrate of 1000 gpm, the theoretical detention time in the flocculator is 50 minutes. At the plant design capacity of 1805 gpm, the flocculation detention time is 28 minutes. Flocculators are typically designed to provide detention times of 15 to 45 minutes. Excessive flocculation time can break up fragile flocs, creating a need for excessive amounts of coagulant. A large amount of settled floc in the flocculation basin may indicate excessive detention time. At the current flowrate of 1000 gpm, Tri-City may see improved performance by turning off the second stage flocculators during warm water temperatures.

A concrete divider wall separates the first stage of flocculation from the second stage. The top of this divider wall is 2 feet below the water surface and three 6-inch square openings are located at the bottom of the wall. At a flowrate of 1000 gpm, the velocity over the baffle wall between the first and second stages of flocculation is 0.045 fps. At the plant stated design capacity of 1805 gpm, the velocity at this point is 0.08 fps.

A wooden baffle wall with 40 holes (8 in^2 each) separates the second stage flocculation area from the sedimentation area (80 opening total). At a flow of 1000 gpm, the velocity in the wooden baffle wall openings is 0.5 fps. At the plant design flow of 1805 gpm, the velocity in the openings is 0.9 fps.

In dual stage flocculators, the speed of the paddles in the first stage is typically higher than in the second stage to provide a tapering down of the velocity gradient (mixing energy) through the stages. The plant operator can control the speed of the flocculators to provide optimum gentle mixing. A rule of thumb is to provide a tip speed of about 1 fps in the final flocculator stage with a normal range of 0.9 to 1.3 fps. The first stage tip speed would be slightly higher. In colder water or high turbidity periods, a faster tip speed is usually needed.

Table 6.2.2-1 – Flocculator Basin Data

	Q = 1000 gpm	Q = 1740 gpm	Q = 1805 gpm	Recommended
Theoretical Detention Time (min)	50	29	28	30-45
Number of Stages	2	2	2	2-3
Inlet Velocity (fps)	1.42	2.47	2.56	1.5-3.0
Exit Velocity (fps)	0.50	0.87	0.90	0.7-1.0

Generally, the flocculators are suitable for the current flow rates required as well as the flows projected for the planning period. At the projected 20-year MDD rate of 1740 gpm, detention time in cold weather (29 minutes) will be marginal but within the range of accepted values. The EPA suggests that 30 minutes of detention time be provided when water temperatures drop below 5°C. The often cited “10-State Recommended Standards for Waterworks” also requires at least 30 minutes for flocculation. The steel paddles have needed repainting in the past and are again rusting and in need of painting.

6.2.3 Sedimentation

The concrete sedimentation basin is 24 feet wide by 48.42 feet long with a surface area of 1162 ft^2 and a length to width ratio of 2:1. The bottom elevation is 621.25 feet and the water surface elevation is 633.50 feet providing a water depth 12.25 feet. The volume of the basin is 14,235 ft^3 or 106,489 gallons. With the sludge collection equipment installed, the remaining volume is estimated to be 105,000 gallons. Sludge collection and removal is accomplished with an Eimco Trac-Vac. Effluent from the sedimentation basin is drawn from the surface with 4 finger weirs 24 feet in length. Total weir length is 192 feet. Tube settlers are installed under the finger weirs. The water surface area over the tube settlers is 576 ft^2 and the tube settlers are 4 feet deep with 60° plates.

Table 6.2.3-1 – Sedimentation Basin Data

	Q = 1000 gpm	Q = 1150 gpm	Q = 1740 gpm	Q = 1805 gpm	Recommended
Theoretical Detention Time (min)	105	91	60.3	58.2	90-180
Theoretical Detention Time (hrs)	1.75	1.52	1.00	0.97	1.5-4.0
Overall SOR (gpm/ft^2)	0.86	0.99	1.50	1.55	0.5-0.6
Tube Settler SOR (gpm/ft^2)	1.74	2.00	3.02	3.13	2.0
Weir Loading Rate (gpd/ft)	7,500	8,625	13,050	13,538	15,000 or less
Horizontal Through Basin Velocity (ft/min)	0.45	0.52	0.79	0.82	0.5 or less
Water Depth	12.25	12.25	12.25	12.25	10-14

SOR = surface overflow rate

Sedimentation basin design criteria according to EPA (Optimizing Water Treatment Plant Performance Using the Composite Correction Program, 1998, EPA/625/6-91/027) suggests a surface overflow rate (SOR) of 0.6 gpm/ft^2 for turbidity removal and 0.4 gpm/ft^2 for color removal for conventional rectangular basins with depth between 12 and 14 feet. With vertical tube settlers (>45°), the SOR can be increased to 2.0 gpm/ft^2 for turbidity removal and 0.75 gpm/ft^2 for color removal (based on area over tubes only). AWWA/ASCE recommends (Water Treatment Plant Design, Third Edition) a SOR of 0.55 to 0.83 for turbidity removal with reduction to 0.35 to 0.55 gpm/ft^2 for water with high algae content. The AWWA/ASCE text also recommends SOR of 1.0 to 3.0 gpm/ft^2 over tube settlers with the normal design based on 2.0 gpm/ft^2. The 10-State Recommended Standards for Waterworks, requires 4 hours of detention time as well as a maximum horizontal through velocity of 0.5 fpm. Detention time may be reduced when the SOR is less than 0.5 gpm/ft^2.

The AWWA/ASCE text and most other references recommend weir loading rates of 20,000 gpd/ft or less. When turbidity can exceed 50 NTU, rates of 15,000 gpd/ft are commonly used. Typically the sedimentation basin has a length to width ratio of 3:1 to 5:1 and the weirs extend into the basin 1/3 of the length or less. The existing sedimentation basin has a length to width ratio of 2:1 with weirs extending 1/2 of the length. Length to width ratios of less than 3:1 can result in excessive short circuiting. Another problem that can occur due to short basins with excessive horizontal through velocities (should be 0.5 ft/min or less in short basins) is an excessive rise rate at the end of the basin causing carryover of floc into the far end of the finger weirs.

At the outlet end of the sedimentation basin, the finger weirs terminate in a collection trough that is 34-inches wide and extends to full width of the basin. A single 18-inch pipe drops vertically from the center of the trough and travels into the building were is splits into two 12-inch pipes and connects to the old original package plant flocculator inlet. Velocity in the 18-inch pipe is 1.26 fps at 1000 gpm, 2.19 fps at 1740 gpm, and 2.28 fps at 1805 gpm. Velocity in the 12-inch pipes ranges from 1.42 to 2.56 fps over the flow rates mentioned above.

The original flocculation/sedimentation basins from the 1979 plant were left in place and effluent from the new exterior basin enters these 2 aluminum package units. Each of the two original sedimentations basins has a surface are of 403 ft^2 and contains tube settlers. The SOR in these basins is 1.24 gpm/ft^2 at the current flow of 1000 gpm and would be 2.16 gpm/ft^2 at 1740 gpm. Since coagulation and flocculation has already occurred in the exterior basin, and any floc carryover from the exterior basin is sheared by local high velocities in the finger weirs and outlet piping, the interior basins do little to help treatment. Some additional settling does occur in the interior basins, especially with the small size of the exterior basin, but plant operations staff has found that flow rates much higher than the current 1000 gpm causes this light flocc to quickly carry over the to filters. The interior sedimentation basins should not be relied upon as part of the treatment process and are not included as part of the plant flocculation and sedimentation process.

The Tri-City sedimentation basin is sized properly for the current plant flow of 1000 gpm, and meets most standard guidelines for sedimentation basins for flows up to 1150 gpm. At flows above this, the horizontal velocity becomes greater than desired, the detention time becomes smaller than desired, and the surface overflow rate (SOR) becomes greater than desired. At the projected flow of 1740 gpm, problems will most likely occur with substantial floc carryover. Even at current flows, the relatively short length of the basin combined with a horizontal velocity near the upper recommended value of 0.5 fpm should cause some sludge upset with potential floc carryover at the effluent end of the finger weirs. Staff in fact reports that this does occur periodically at the current flows.

6.2.4 Filtration

The Tri-City water treatment plant contains four filters that were originally installed as part of the 1979 plant. The filters are aluminum tanks manufactured by Keystone. Each filter has dimensions of 13 feet by 9.17 feet providing a surface area of 119 ft^2 each or a total filter area of 477 ft^2. As part of the 2000 plant upgrade, the filter media was replaced with 16-inches of anthracite (0.9-1.0 mm), 9-inches of silica sand (0.45-0.55 mm), and 3-inches of garnet sand (0.15-0.25 mm) underlain by 19-inches of support gravel of various sizes. According to AWWA/ASCE "Water Treatment Plant Design, Third Edition", a proper mixed-media filter should have 3 to 4 inches of garnet, 6 to 9 inches of sand, and 18 to 24 inches of anthracite. The filters are operated at a steady rate with a 6-inch Cla-Val level control valve at each filtrate pipe modulating to maintain a constant water surface in the filter bays.

At the current flow of 1000 gpm, the filter loading rate is 2.1 gpm/ft^2. At a flowrate of 1740 gpm, the filter loading rate would be 3.7 gpm/ft^2. At the stated design capacity of the plant of 1805 gpm, the filter loading rate would be 3.8 gpm/ft2. A maximum filter loading rate of 4.0 gpm/ft2 is recommended by EPA and AWWA for mixed media filters in good condition and no signs of air binding. In practice, acceptable water quality can usually only be achieved at a rate as high as 4.0 gpm/ft^2 when flocculation and sedimentation processes are optimal and chemical feed is controlled very closely. Since the sedimentation process in Tri-City is not ideal, and only 28-inches of filter media exists in the filters, it is not recommended that filter rates above 3.0 gpm/ft^2 (1,430 gpm) be used in Tri-City.

Rotary surface washers are installed in each filter to agitate the surface of the media. The washers should be approximately 2-inches above the surface of the anthracite and become submerged in the media during backwashing. A flowrate of at least 60 gpm (0.5 gpm/ft^2) to each filter at a minimum pressure of 50 psi is required for proper surface wash function. A flow of 120 gpm at 75 psi is recommended. 2-inch piping runs from a tap on the 12-inch plant discharge pipe (~100 psi at tap), through a Cla-Val rate-of-flow controller, and then to each filter. A solenoid-actuated diaphragm valve at each filter opens and closes automatically to start/stop the surface washers based on the PLC program sequence. No flowmeter is installed on the surface wash line so verification of flowrate is difficult. Since this flow is not metered, the quantity of water used for surface washing will show up as unaccounted water. It is recommended that a flowmeter be installed to verify flowrate and record daily volumes. A significant deficiency is the lack of a backflow prevention device on the surface wash line. The surface washers become submerged in unfiltered water during a backwash and the piping is tied directly to the finish water pipe leaving the plant. Current regulations require a backflow preventer in this installation and Tri-City should install one as soon as possible.

Filter backwash is accomplished with hydraulic upflow water and surface washers. No auxiliary air scour is provided. The original backwash pump from the 1979 plant remains in use today. The pump conveys treated water from the clearwell and forces the water upwards through the filter media to expand the bed and remove sediment. The backwash pump is a Byron Jackson single stage vertical turbine, model 12HQRH, with a 40 Hp 460V motor, rated for 2125 gpm at 48 feet TDH. Drawdown test show that the pump currently pumps out about 2430 gpm. Backwash rates required will vary between 17 and 23 gpm/ft^2 depending on the media configuration and the water temperature. The goal for proper hydraulic backwash is to achieve a 30-50% expansion of the media. For each 1°C increase in water temperature, an increase in the backwash rate of approximately 2% is required to prevent a reduction in bed expansion. For the filter area of 119 ft2, a backwash rate of 2740 gpm should be available. In cold water periods, a rate of 2050 gpm may suffice. During a backwash the media should expand at least 9-inches in the existing filters. The backwash pump output (20 gpm/ft^2 maximum) is slightly less than optimal and mudball formation is possible over time.

A recommended backwash procedure is as follows: 1) activate surface washer alone for 1 to 3 minutes. 2) Apply low rate backwash (~13 gpm/ft^2) and surface wash simultaneously for 5 to 10 minutes. 3) Terminate surface wash and apply full high rate backwash to expand media bed to 30-50%. Wash only as long as necessary, until wash water turbidity is 10 NTU, generally 1 to 5 minutes. 4) After backwashing, leave the filter off-line in a rest period. 5) Begin filter-to-waste and continue until turbidity drops to acceptable level. 6) Begin normal filtration. Some research shows that applying a subfluidization backwash (~10 gpm/ft^2) for a few minutes after the high rate backwash can reduce ripening time.

Problems with air in the backwash water reaching the filters is evident most of the time. The filter media build is not designed for air scour and this entering air has the potential to disrupt the bed and eventually destroy the filter. It is suspected that the air problem is due to a combination of an improper air release valve at the pump together with backwash control butterfly valve actuators which are undersized and open too rapidly. Improvements to the backwash piping should be made as soon as possible including replacement of the air valve, removal of the non-functioning Griswold flow restrictor, and replacement of the valve actuators. The manufacturer of the existing air/vacuum valve, Crispin, recommends that a 2-inch Crispin UL20/SC Universal valve be installed. The air valve should also be relocated to a point upstream from the pump check valve. During the installation of new piping and valve actuators, it would be prudent to replace the 25 year old butterfly valves. Installation of a flowmeter is also recommended to allow the operator to properly adjust flowrates and to record volumes of water used for backwashing.

As mentioned in the beginning of Section 6.2, the plant is unable to produce the design flowrate of 1805 gpm due to an apparent hydraulic overloading in the filtrate piping. The major restriction in the filtrate piping is the 6-inch Cla-Val filter level control valve. At the design flow of 1805 gpm, the headloss through the Cla-Val is approximately 2.25 feet. The total friction headloss between the filter outlet and the clearwell is estimated to be approximately 3.9 feet. When the filters are clean there is approximately 6 feet of head available at the filter outlet. As the filter media becomes dirty, the head available can drop to near 4 feet and the flow can no longer be conveyed to the clearwell. A simple solution is to replace the Cla-Val valves with automatically actuated butterfly valves. A portion of the interior piping can also be increased in size as required to reduce friction loss.

6.2.5 Disinfection

Gaseous chlorine is used to provide disinfection at the Tri-City water treatment plant. Separate feed lines allow pre-chlorination and final chlorination. The two original Wallace & Tiernan V-Notch vacuum feed chlorinators from 1979 is still in use today. The chlorine dosage equipment is housed in a chlorine room. Two ton containers of chlorine gas are housed in a separate storage room with access from an exterior door only. Approximately 2400 lbs of chlorine gas is stored on site. The chlorine facility lacks the forced exhaust and chlorine detector alarms required today. In addition, a chlorine scrubber may be required by current regulations. The chlorination facilities in the plant are a safety hazard for workers and significant improvements to the installation should be made or a switch to sodium hypochlorite should be made.

Finished water storage at the plant used for backwashing and to provide contact time for chlorine disinfection is provided in two clearwells. Filtered water flows into the baffled newer clearwell and is then piped to the original clearwell. The combined clearwells have a volume of approximately 1,317 gallons per inch depth.

The newer clearwell is a concrete basin constructed in 2000 under an office/storage building adjacent to the filtration building. The new clearwell has dimensions of 42.33 feet by 22.33 feet by 7.83 feet deep. The newer clearwell has significant serpentine baffling to provide greater chlorine contact time. Subtracting area for the baffle walls, the new clearwell has a surface area of 886 ft^2. Maximum water depth in the clearwell is approximately 7.5 feet (elev. 621.25') resulting in a maximum volume of 49,708 gallons. The PLC settings allow the clearwell water level to drop to 4 feet depth during plant operation.

The original clearwell, constructed in 1979, is a single concrete basin under the filtration building with dimensions of 48.33 feet by 25.66 feet by 8.33 to 9 feet deep. Subtracting area for support columns and pump baffle walls, the original clearwell has a surface area of 1,226 ft^2. Other than small baffles between the pump suction strainers, the clearwell has no baffling. Maximum water depth in the clearwell is approximately 7.5 feet (elev. 621.25') resulting in a maximum volume of 68,783 gallons. Actual operating water depth in the clearwell can vary between 7.5 to 4.0 feet based on the PLC settings. At 4 foot depth, the finished water pumps are forced off.

In 2003, a contact time tracer study was conducted to determine the contact time available in the clearwells. Water level was maintained at 4.5 feet to simulate the worst case normal conditions as required by EPA. A continuous flowrate of 1040 gpm was held during the test. A contact time of 34 minutes was measured under these conditions. An overall efficiency of 50% resulted for the water volume in the clearwells.

6.2.6 Finished Water Pumping

Three vertical turbine pumps located over the original clearwell function to pump treated water from the plant into the distribution system. Two 100 Hp pumps were installed in 2000. The third 75 Hp pump remains from the 1979 improvements and is only used as a backup. A single 100 Hp pump has a capacity of approximately 1000 to 1060 gpm depending on the water level in the distribution storage tanks. With both 100 Hp pumps operating, a flow of approximately 1800 gpm results. Only a single pump is operated at a time since at higher flowrates the plant hydraulically overloads as discussed previously. The Authority is in the process of installing variable frequency drives (VFDs) for the two 100 Hp motors to allow better control over the flowrate. Once these are installed, the maximum flow before hydraulic overload can be determined and the plant can be operated at this slightly higher flowrate to extend the useful life of the current facility.

6.2.7 Treatment Performance

At the current flowrate of 1000 gpm, the plant treatment performance is good. Raw water turbidities sometimes approach 300 NTUs while finished water turbidity averaged 0.047 NTU (measurements from 1/1999 to 6/2004). The plant is required to have a finished water turbidity of 0.3 NTU or less in at least 95% of the measurements taken each month and no samples with a turbidity greater than 1 NTU. No turbidity violations have occurred since the plant upgrade in 2000. The water pumped into the system is almost always less than 0.1 NTU. The occasional spikes recorded above 0.1 NTU are generally very short duration spikes occurring when the plant first starts up, or are values recorded when the plant is off.

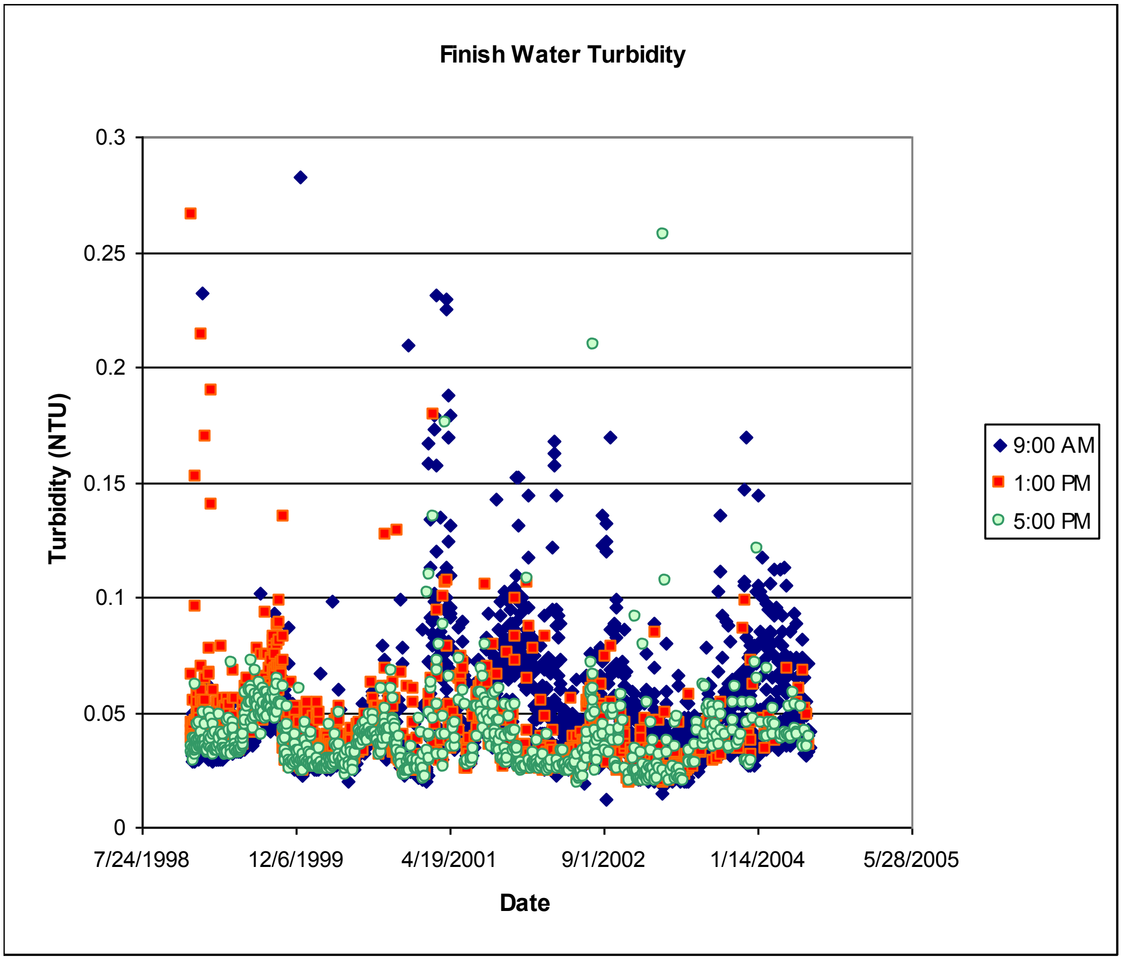


Figure 6.2.7-1 – Finished (Filtered) Water Turbidity

Based on 21 months of collected total organic carbon (TOC) tests, the source water TOC averages 1.92 mg/L and finished water TOC averages 1.11 mg/L. The quarterly average TOC exceeded 2.0 mg/L one time but the running annual average is less than 2.0 mg/L. The average TOC removal by the plant is 41.8% and all finish water TOC measurements were below 2.0 mg/L. Based on this data Tri-City is not required to practice enhanced coagulation. Currently the average TOC reduction being achieved is 0.83 mg/L.

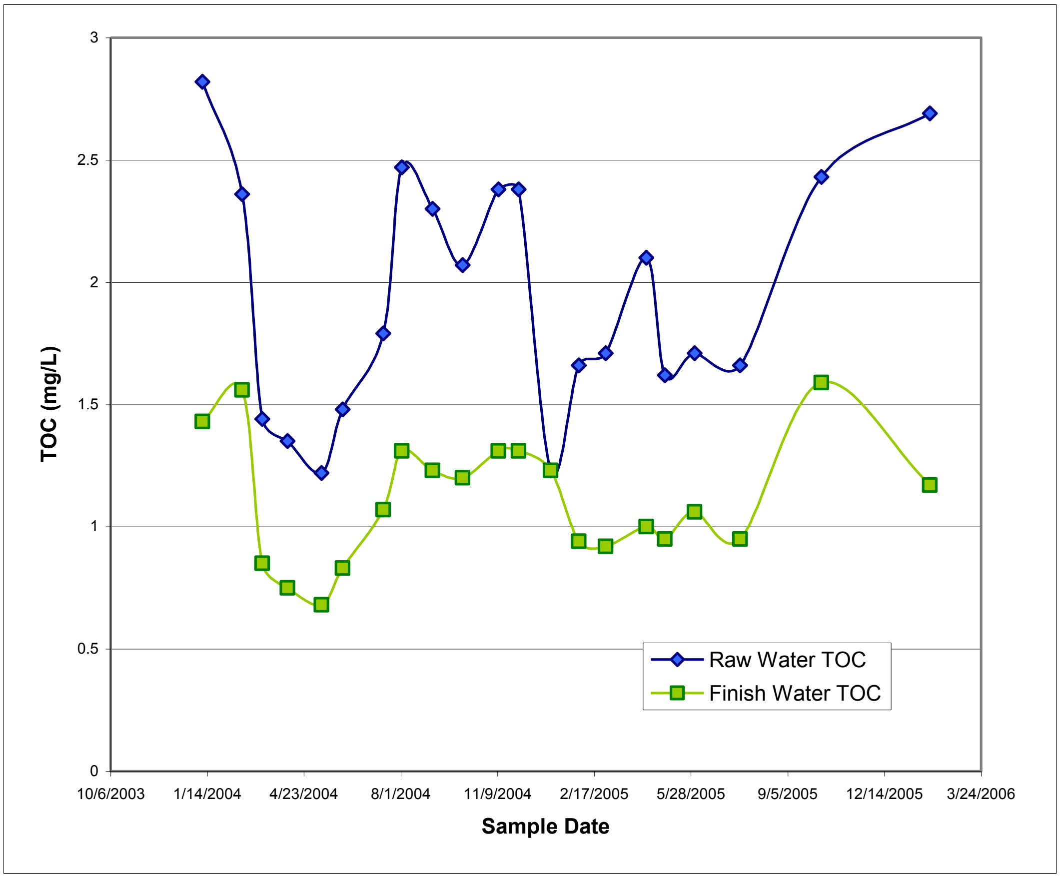


Figure 6.2.7-2 – Source Water and Finished Water TOC

In addition, no chlorine contact time violations have occurred although the required minimum CT values have been only marginally exceeded at times. In general the plant performs well at the current flowrate of 1000 gpm. The successful plant performance is due in large part to the qualified staff that operates the plant carefully. Chemical feed is monitored and adjusted regularly based on raw water quality, filter performance is monitored closely, backwashing and basin cleaning is done regularly, plant operation is ceased during major high turbidity events when full time attendance is not possible, and the plant is monitored remotely from the operator's home. In addition, the plant is clean and well maintained. As increased flows are demanded by the community, plant operation will become more difficult. The limiting plant process is the sedimentation basin. To meet design guidelines for sedimentation processes, the sedimentation basin is limited to 1150 gpm. It may be possible to push plant flows up to around 1400 gpm with improvements to clearwell baffling to increase contact time, and correction of hydraulic limitations in the filtrate piping, however it is difficult to predict how plant performance will be affected since the sedimentation process will be overloaded and filtration rates will approach the maximum recommended with non-ideal sedimentation.

6.3 Treated Water Storage Facilities

Tri-City has four treated water storage tanks in the distribution system. Three of the tanks (October, Walnut, and Aker Tanks) are filled by the treatment plant discharge pumps pressurizing the system. The fourth tank (Back Acres Tank) occurs at a higher elevation and is filled with a booster pump station. All tanks have level transmitters with water depth recorded at the treatment plant. The water level output from any of the three lower tanks can be used for plant on/off control. Water level output from the Back Acres tank is used to control the Valley Drive Booster Pumps Station.

In normal operation, as influenced by local area water demands and distribution piping hydraulics, the Aker Tank typically fills the quickest. When the Aker Tank is full, its altitude valve shuts diverting all water to the other tanks. Second, the Walnut tank reaches full depth and its altitude valve shuts. Last, the October tank reaches full depth and the plant discharge pumps are automatically shut off.

The tank interiors are inspected and cleaned periodically by hired tank divers trained in tank cleaning. Little sediment build-up has been reported and no interior repairs have been required nor are indicated as being needed.

6.3.1 October Drive Storage Tank

The October Drive storage tank is a pre-stressed concrete tank constructed in 2001.

October Drive Storage Tank

The tank is 66 feet in diameter and 40 feet tall. Floor elevation of the tank is 816.0 feet, overflow elevation is 855.0 feet (39 feet water depth).

The current high water alarm setting is at a depth of 38 feet (elev. 854.0). Current maximum water depth when full is 37 feet (elev. 853.0) providing a storage volume of 947,000 gallons.

Separate inlet and outlet pipes exist with the inlet pipe extending vertically at the tank center to elevation 853.0. The outlet pipe discharges from near the tank bottom. Check valves located in an exterior vault force the water in one way and out the other for improved turnover in the tank.

The tank is in good condition and should not need attention during the planning period. Unauthorized access is deterred through adequate fencing and a locking ladder access cage. A hatch alarm should be considered to notify the Authority in case of unauthorized opening of the top access hatch.

6.3.2 Walnut Street Storage Tank

The Walnut Street storage tank is a welded steel tank constructed in 1979.

Walnut Street Storage Tank

The tank is 50 feet in diameter by 36 feet tall. The tank has a floor elevation of 821.0 feet, an overflow elevation of 856.5 feet (35.5 feet water depth).

The current high water alarm setting is at a depth of 35 feet (elev. 856.0). Current maximum water depth when full is 34.5 feet (elev. 855.5) providing a storage volume 506,700 gallons.

The tank has a single 12-inch inlet/outlet pipe with an altitude valve which functions to close when the tank is full preventing an overflow.

Vandalism has been a problem at the Walnut Street tank. A bullet hole in the tank has been patched. Evidence of recent vandalism includes damage to fencing and the roof vent structure.

The Walnut tank has cathodic protection since power is available at the site. The interior is unpainted but the cathodic equipment appears to have prevented interior rusting. The cathodic anodes and rectifier have been replaced recently. The tank exterior is deteriorated and should be cleaned and painted during the planning period.

Improvements are needed to the ladder cage to hinder unauthorized access to the tank roof. In addition, a hatch alarm and detection cameras should be considered. Interior piping or baffling is needed to improve mixing and help maintain proper chlorine residuals.

6.3.3 Aker Drive Storage Tank

The Aker Drive storage tank is a welded steel tank constructed prior to 1979.

Aker Drive Storage Tank

The tank is 52 feet in diameter by 32 feet tall. The tank has a floor elevation of 823.50 feet. The overflow elevation is unknown.

Current high water alarm setting is at a depth of 31 feet (elev. 854.5). Current maximum water depth when full is 30 feet (elev. 853.50) providing a storage volume of 476,600 gallons.

The tank has a single 10-inch inlet/outlet pipe with an altitude valve. The altitude valve was replaced in 1979.

No electrical power is currently available at the Aker tank so no cathodic protection exists. The epoxy coated tank interior is in good condition however. The tank is in need of exterior cleaning and painting. The exterior refurbishment should be conducted as soon as possible to prevent damage to the steel substrate.

Improvements to the ladder cage should also be considered along with a hatch alarm. Interior piping or baffling is needed to improve mixing and help maintain proper chlorine residuals.

6.3.4 Back Acres Storage Tank

The Back Acres storage tank is a welded steel tank constructed in 1979. The tank is filled by the Valley Drive Pump Station.

Valley Drive Storage Tank

The tank diameter is 33.75 feet and the nominal depth is 16 feet. The tank has a floor elevation of 1015.5 feet, an overflow elevation of 1031.0 feet (15.5 feet water depth).

Current maximum water depth when full is 13 feet (elev. 1028.5) providing a storage volume of 87,000 gallons.

The tank has a single 8-inch inlet/outlet pipe and no interior baffling.

Electrical power exists at the site and the Valley Drive Tank includes cathodic protection. The interior is unpainted but is in good condition. The anodes and rectifier are scheduled for replacement in 2006 at a cost of approximately $6200. The tank exterior exhibits some deterioration of the coating and the tank exterior should be refurbished during the planning period. The tank also lack fencing. Per OAR 333-061-0050(a)(6)(P), a fence or other method of vandal deterrence shall be provided around distribution reservoirs. Interior piping or baffling is needed to improve mixing and help maintain proper chlorine residuals.

6.3.5 Storage Summary

Total system storage volume provided by the four tanks is 2.02 million gallons. Approximately 51,000 gallons of additional storage could be provided by raising the various high water alarms and full depth settings slightly closer to the overflow elevations.

Table 6.3.5-1 – Storage Tank Data

	Walnut Tank	Aker Tank	October Tank	Back Acres Tank
Overflow (Depth/Elev)	35.5 (856.5)	?	39.0 (855.0)	15.5 (1031.0)
Floor Elevation	821.0	823.5	816.0	1015.5
HW Alarm (ft depth)	35.0	31.0	38.0 (38.5)	14.0 (15.25)
Full, Pumps Off	34.5	30.0	37.0 (37.5)	13.0 (15.0)
Full W.S. Elevation	855.5	853.5	853.0	1028.5
Lead Pump On, Winter	20.0 (841.0)	15.0 (838.5)	27.0 (843.0)	9.0 (1024.5)
Lead Pump On, Summer	24.0	18.0	29.0	9.0
Lag Pump On	19.0	14.0	26.0	8.0 (8.5)
Low Water Alarm	18.0	13.0	25.0	8.0
Current Volume at "Full"	506,700 gal.	476,000 gal.	947,000 gal.	87,000 gal.
Maximum Volume*	506,700 gal.	?	985,000 gal.	100,300 gal.

* *Maximum Volume with "Full" setting 6-inches below overflow*
(recent changes)

The storage tank water levels are allowed to drop fairly low in efforts to cause more water “turn-over” in the tanks and promote higher chlorine residuals in the tanks. As the tank water levels drop to the “lead pump on” setting, total storage volume in the system drops by nearly 50% to 1.1 million gallons. In summer months when water demand in the system is greater and “turn-over” in the tanks increases, the plant operator changes the settings to minimize the storage loss. If the operator does not allow the tanks to drop in volume to the extents currently used, violations with chlorine residual occur.

Drinking water quality rules require that the operator maintain at least a detectable free chlorine residual at all times at all points in the system. Storage tanks are often the point in the system where it is difficult to maintain adequate chlorine residuals. In extreme cases, communities must install chlorine booster stations to add supplemental chlorine at storage tanks or other reaches of the distribution system.

In Tri-City, with three out of the four tanks having single inlet/outlet piping arrangements and no internal baffling to promote mixing and turn-over, chlorine residuals drop too low if the tanks are kept fairly full. According to OAR 333-061-0050(a)(6)(O), where a single inlet/outlet pipe is installed and the reservoir floats on the system, provisions shall be made to insure an adequate exchange of water and to prevent degradation of the water quality and to assure the disinfection levels are maintained at adequate levels. To comply with this rule, it is recommended that piping modifications be done at the tanks (except October Tank) to promote water turn-over.

6.4 Distribution Pumping Facilities

Two public pumping stations exist in the Tri-City water system. The Valley Drive Pump Station pumps water to the upper level Back Acres Storage Tank. The Woodcrest Booster Pump Station boosts pressure in a small local area around upper Woodcrest Drive.

6.4.1 Valley Drive Pump Station

The Valley Drive pump station consists of two multi-stage centrifugal pumps housed in a concrete block building. The building floor is at elevation 730 feet according to the original construction plans. The pump station was originally constructed in 1960 and was upgraded in the late 1970’s and again in 1980. After the 1980 upgrade, each pump had a capacity of 135 gpm. By 2004, the pump station was experiencing difficulty keeping up with water demand and the pump station equipment and electrical service was upgraded in late 2005.

Pump suction occurs from the distribution system and a low suction pressure cut-off switch turns the pumps off and sends an alarm signal to the water treatment plant dialer should pressure drop below 20 psi. Normal pump suction pressure ranges from 47 psi when the storage tanks are low to 54 psi when the tanks are full. The pumps convey water to the Back Acres high level storage tank. Flows through the pump station are displayed and totaled locally and at the SCADA computer at the water treatment plant. A water level transmitter at the Back Acres storage tank sends an analog signal corresponding to the tank depth back to the pump station to turn the pumps off when the tank is full and turn the pumps off at the adjustable low level setting. Normally, a single pump operates at a time with the lag pump starting only in extreme cases of continued tank water level falling such as may occur during a fire demand.

Each pump is a Grundfos model CR64-2 rated for 339 gpm at 205 feet total dynamic head (89 psi discharge pressure). Drivers are 25 Hp, 3500 rpm, 60 Hz, 460 Volt, 3 phase, premium efficiency motors manufactured by Grundfos. Motors are started using soft-start motor starters manufactured by Square D. The station flowmeter is a 4-inch Water Specialties Model VF-32-D, vertical downflow propeller meter installed in a tee.

The Valley Drive pump station pump capacity is sufficient for approximately 300 homes which is greater than the expected build-out of the local high level service area. The Valley Drive pump station should not need improvements during the planning period other than possible repainting of the building exterior and door as needed.

6.4.2 Woodcrest Booster Pump Station

The Woodcrest booster pump station is a small duplex pump station with a hydropneumatic tanks housed in a fiberglass tilt-up enclosure. The equipment package was designed and manufactured by Engineered Fluid Inc. (EFI) and installed in Tri-City in 1997 to serve approximately 30 potential homes located near the maximum service elevation of the main pressure zone. The station sits at an elevation of approximately 690 feet. Normal suction pressures range from 64 to 71 psi depending on the water level in the storage tanks. The pump station controls are set to start the pumps when the pressure in the hydropneumatic tank drops to 65 psi, and stop the pumps at 107 psi. Discharge piping from the pump station extends to an elevation of 754 feet. When the station discharge pressure is at 65 psi, the static pressure at the upper fire hydrant connected to the station discharge piping is 40 psi.

The station contains two Carver 3 Hp, 3500 rpm, single phase pumps and two 86 gallon Well-X-Trol model WX-302 hydropneumatic tanks. The tanks are precharged to 38 psi and have a total drawdown of 46 gallons (both tanks together) when the pressure settings are 40 psi low and 60 psi high. The pump station is rated for 55 gpm at 85 feet total dynamic head. A pump station dependent on a hydropneumatic tank should have the capacity to pump peak hourly demands as compared to peak daily demands when a storage tank is provided. In Tri-City, the peak hourly demand is approximately 1.1 gpm per household. With a capacity of 55 gpm, the Woodcrest BPS can serve up to 50 homes. The station appears to have sufficient capacity to handle all potential growth in its local service area and should not need capacity improvements during the planning period. The pressure tanks in the station however are relatively small resulting in short pump run times, frequent cycling, and reduced pump life, especially as the service area expands beyond 20 homes.

The pumping equipment will be over 25 years old at the end of the planning period and the need for replacement or repair during the planning period is likely. When replacement becomes necessary, at least one pump with a variable frequency drive should be installed along with a second constant speed pump. Alternatively, larger hydropneumatic tanks can be installed to provide at least a two minute pump run time and no more than 6 starts per hour.

6.5 Distribution Piping System

The Tri-City water distribution system contains 4 storage tanks, 2 pump stations, approximately 145 fire hydrants, and 29.8 miles of piping ranging from 1 to 16 inches in diameter. The system is separated into three pressure zones.

A map of the system showing the piping, WTP, pump stations and storage tanks is shown in Figure 6.5-1. The map also indicates the maximum service elevations.

6.5.1 History

According to past planning documents, the water system generally began in the early 1950s with significant expansion in the 1960s and 1970s to accommodate rapid growth in the area. By the early 1990s the system had expanded to include approximately 115,000 feet of water piping, 4 storage tanks, and 2 pressure zones. The 1994 Water Master Plan indicated that at that time, over 20% of the piping was 2-inches and smaller, 92% of the piping was 8-inches or smaller, and only 3% was 12-inches or larger. Over the last decade significant piping improvements have been constructed eliminating most of the 2-inch and smaller pipe, adding over 16,000 feet of 8-inch pipe, almost 40,000 feet of 12-inch pipe, and approximately 2,400 feet of 16-inch pipe. The majority of the piping is PVC in good condition however some of the AC piping from the 1970s remains in service.

6.5.2 Pressure Zones

The Tri-City water system contains 3 pressure zones, the Main Pressure Zone, the Back Acres Pressure Zone, and the Woodcrest Pressure Zone.

The majority of the system is in the main pressure zone served by the October, Walnut, and Aker Storage Tanks, all filled by the water treatment plant discharge pumps. The maximum service elevation (for 25 psi at connection) in the main zone is 780 feet. Homes above elevation 745 feet will have less than 40 psi and may required individual household booster pumps.

The Back Acres high level service zone is served by the Back Acres Storage Tank, filled by the Valley Drive pump station. The maximum service elevation (for 25 psi at connection) in the Back Acres pressure zone is 965 feet. Homes above elevation 930 feet will have less than 40 psi and may need individual booster pumps.

The Woodcrest pressure zone is a minor zone servicing a small area at the upper end of Woodcrest Drive. Even though the minimum pressure requirement of 20 psi at the property line would be provided without the pump station, homes in this area would have marginal pressures ranging from 25 to 45 psi without the booster pump station. The maximum service elevation (for 25 psi at connection) in the Woodcrest pressure zone is 782 feet. Homes above elevation 748 feet will have less than 40 psi and may need individual booster pumps.

Per OAR-333-061, water providers must ensure that at least 20 psi exists in the water distribution piping at all service connections (at the property line) at all times. With adequately sized and looped distribution piping, a static pressure of 25 psi is usually enough to ensure that at least 20 psi remains during high flow events. A pressure of 35 to 40 psi is considered the minimum acceptable pressure at a household and pressures above 80 psi can damage some residential devices such as hot water heaters. Height of water in feet can be converted to psi by dividing the height by 2.308 (i.e. 10 feet of water equals 4.3 psi).

Public piping should never be constructed above the maximum service elevation in any zone without careful planning and the addition of pump stations. When piping is extended too high in elevation, pressure violations may occur. Often, points in the distribution system at the highest elevations, near the maximum service elevation, limit fire flows since the pressure tends to approach 20 psi quickly as large fire flows are pulled from lower areas.

Table 6.5.2-1 – Pressure Zones

Zone	HGL Creation	Max HGL (ft)	Max Service Elev. (25 psi)	Max Elev. w/o BPS (40 psi)	Min Service Elev. w/o PRV (80 psi)	Elev. (100 psi)
1 (Main)	Tank WS	856-839	780	745	670	625
2 (Back Acres)	Tank WS	1028.5-1024.5	965	930	840	800
3 (Woodcrest)	690' + 65-107 psi boost	840-937	782	748	752	706

6.5.3 Pipe Inventory

Table 6.5.3-1 – Pipe Inventory

	Length (feet)	% of Total Length
Less than 2"	620	0.4%
2"	9,875	6.3%
4"	7,095	4.5%
6"	53,500	34.0%
8"	37,500	23.8%
10"	6,030	3.8%
12"	40,380	25.7%
16"	2,410	1.5%
Total	**157,410 (29.8 miles)**	**100%**

6.5.4 Condition of the Distribution System

A majority of the piping is PVC installed in and after 1980. Almost 25% of the system piping has been constructed within the last 12 years. The PVC piping installed over the last 25 years is assumed to be in good condition with a service life extending beyond the planning period. Portions of old piping not replaced in the last 25 years include 6-inch and 8-inch AC (asbestos cement) scattered throughout the area and some sections of 10-inch AC pipe. The exact percentage and length of this older piping is unknown however virtually all 6-inch pipe (1/3 of the system) is AC installed over 30 years ago. These older sections of pipe may be the primary source of any leakage in the system and will become more problematic as time passes. Almost 7% of the system piping is 2-inches or smaller in diameter and long runs of this small piping should be replaced when feasible. Unaccounted water averages around 16% including any water used at fire hydrants, water used for the filter surface washers, and all other unmetered usage. Actual leakage is less than 15% on average and the piping system is generally in good condition.

Due to the nature of the terrain and platted lots, approximately 50 dead-end pipes exist in the system. Dead-end pipes can cause water quality problems if not purged with blow-off valves periodically. Some of the dead-end pipes cannot be looped to other pipes economically and periodic attention is required. Other dead-ends could be eliminated with relatively short sections of piping however terrain and easement acquisition may be a concern. According to OAR 333-061-0050(8)(h) "Wherever possible, dead ends shall be minimized by looping. Where dead-ends are installed, or low points exist, blow-offs of adequate size shall be provided for flushing;". The Authority now has a policy to require looped waterlines when feasible for all new development. Improvements to eliminate existing dead-ends where economically feasible will be considered in this Plan.

6.5.5 Disinfection By-Products in the Distribution System

Disinfectant Byproducts (DBPs) include Total Trihalomethanes (TTHM) and Haloacetic Acids (HAA5). These byproducts are formed when chemical disinfectants (chlorine, etc.) react with organic compounds (such as total organic carbon, TOC). DBPs are a health concern because they may cause cancer, as well as liver, kidney, and central nervous system problems. The USEPA and the State has set maximum contaminant levels (MCLs) for DBPs including a limit of 0.080 mg/L for TTHM and 0.060 mg/L for HAA5 computed as a running annual average.

Tri-City initially has to conduct sampling and testing for DBPs 4 times per year. A community qualifies for reduced monitoring (once per year) if TOC is less than 4 mg/L and the DBP results are less than half of the MCL. The 2004 running annual average results in Tri-City were 0.033 mg/L for TTHM and 0.021 mg/L for HAA5. The 2004 test results showed levels low enough to allow Tri-City to drop to annual testing. No MCL violations have occurred however the sample from 1/30/2006 showed the TTHM concentration was 81.75% of the MCL value. High TTHM can result from pre-chlorination, inadequate TOC removal, or high pH. If TTHM values begin to violate the MCL, Tri-City may need to lower the pH in the coagulation process on a regular basis and/or switch to an alternative oxidant prior to filtration.

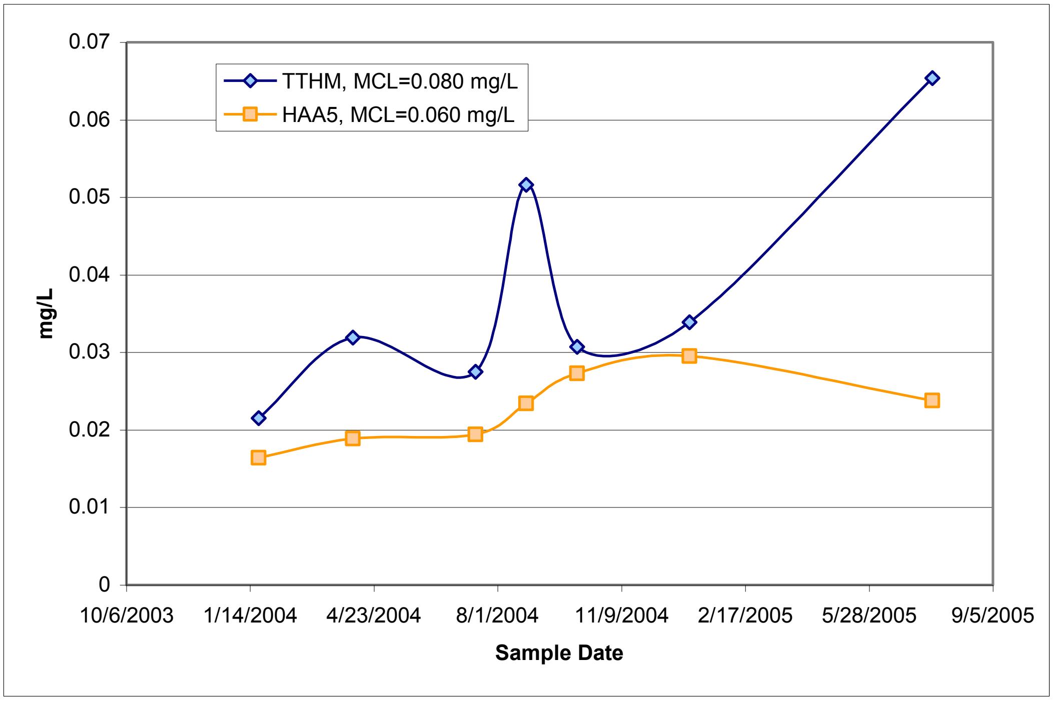


Figure 6.5.5-1 – Distribution System TTHM and HAA5 Concentrations

6.6 Water Charges and Financial Status

6.6.1 Water Use Rates

In the beginning of 2006, water rates were based on a minimum base fee of $30 per EDU plus a charge of $1.50 per 1000 gallons over 4000 gallons. In this structure, the first 4000 gallons of water used cost $0.0075/gallon and excess water cost $0.0015/gallon.

In Tri-City, the average residential water use per household is 7,075 gallons per month (see Section 2.3.3). One EDU is therefore equal to 7,075 gallons per month and a total of 1,731 EDUs exist in the system. Based on the current rate structure, the resulting average water bill is $34.61 per month for a typical single-family dwelling with an average cost per gallon of $0.0049/gallon. With an ideal rate structure, revenue generated by water sales would be $59,910 per month ($34.61 x 1731) or $719,000 per year. For fiscal year 2005 (6/30/04 to 6/30/05), water sales revenue totaled only $600,342. The actual average bill per EDU for 2005 was $28.90. The fact that the actual bill per EDU is less than the expected average bill per single-family dwelling indicates that large water users may be paying less for water than is fair and equitable. This could be due in part to the current rate structure which results in a lower charge per gallon the higher water consumption goes, the possibility that some large users are not being allocated the number of EDUs they actually are based on water use, and special water rates for certain large users.

In March of 2006, the Authority initiated a rate increase to bring the base rate from $30 to $35 per EDU. This rate increase is expected to go in effect around June of 2006. This will result in a bill of $39.61 for the average household use of 7,075 gallons per month with an average cost per gallon of $0.0056 or $5.60 per 1000 gallons. If a rate structure were used such that each user's bill accurately reflected the number of EDUs they were, annual revenue of $822,000 would be generated by water sales. Based on the 2000 Census data for the Tri-City CDP (census designated place) the median household income (MHI) is $33,306. The new average water bill is 1.43% of the local MHI.

It is recommended that a new rate structure be developed which encourages water conservation and does not require small consumers to subsidize large water users. To simplify recordkeeping and rate determination, a rate structure based only on meter size and actual consumption should be considered. Ideally, normal consumption charges would equal the actual cost to produce water. An increasing block structure, where the cost per gallon goes up when amounts over the normal consumption amount are used, will tend to encourage water conservation.

6.6.2 System Development Charges and Connection Fees

The existing system development charge is $1,625 per EDU. The SDC methodology was established in 2001 and charges were based on 1233 new EDUs over a 20-year period ending in 2021. Capital improvements planned for were detailed in the 1994 Water Master Plan and as updated in the 2001 methodology. In 2004, a Resolution was passed adding the cost of this Master Plan to the CIP which resulted in a potential SDC of $1,667 per EDU. The Tri-City Board elected to maintain the SDC at $1,625 rather than raise the charge at that time. Since that time, the $83,100 Valley Drive Pump Station project has been completed. Capacity of the pump station was increased from 135 gpm to 340 gpm with 40% of the capacity being to serve existing customers and 60% being excess capacity to serve growth. It is possible for a water SDC of $45 per EDU (based on 1106 new EDU over 20 years) to be charged now equating to 60% of the pump station upgrade cost. The current water system SDC, without improvements considered in this Master Plan, could be $1,712 based on the current SDC plus the Master Plan and the Valley Drive Pump Station.

SDC revenue collected for fiscal year 2005 was $50,375. For the first 8 months of FY 2006, SDC revenues of $57,105 have been collected.

6.6.3 Water Revenue and Operating Expenses

The most current information is for the 2005 fiscal year. Total revenue exceeded normal expenses by $59,299 for this period however no major improvement projects or maintenance/repair projects occurred. Actual water sales accounted for $600,342 of the operating revenue for an average of $50,028 per month. Approximately $260,650 of the total expense is for debt payment for past projects. The oldest loan is Bonded Dept (General Obligation Bond) serviced by GMAC. The 30-year loan began in 1980 and will paid off in 2010.

For the first 8 months of FY 2006, water sales have averaged $55,665 per month and slightly more revenue is expected as compared to 2005 due to an increased number of customers and changes in various fees. Expenditures will also be higher in 2006 with the purchase of pH control equipment for the water treatment plant, new pump drives, and a new loan payment for the recent Douglas County road widening project (estimated payment of $26,000 per year). Based on initial estimates for 2006, expenditures may exceed revenue slightly. The $5 per month rate increase may go into effect for the last couple of months of FY 2006 eliminating the possible shortfall.

Table 6.6.3-1 – Water System Revenue and Expense

	2006 Estimate	FY 2005
Operating Revenue	$741,871	$683,534
Tax Revenue	$114,888	$114,888
Other Revenue (interest, rent, sales)	$7,000	$6,982
Total Revenue (excluding SDCs)	**$863,759**	**$805,404**
General Operating Expense	$459,402	$463,590
Capital Expenditures (non-SDC eligible)	$133,655	$21,934
Bonded Debt (GMAC)	$89,168	$89,168
Phase I Loan Debt (RD)	$72,438	$72,438
Phase II & III Loan Debt (RD)	$98,705	$99,045
Phase 1 Road Project Debt (County)	$26,000	
Total Expense	**$879,368**	**$746,175**
Remaining	**($15,609)**	**$59,299**
Operating Ratio (Revenue/Expense)	**0.98**	**1.08**

In general, the water system budget has allowed for basic operation and maintenance of the system but has not been sufficient to provide for the accumulation of adequate reserve funds to conduct major system repairs or replacement. Ideally, revenue generated would be sufficient not only to cover normal operation and short-term maintenance of capital assets, but also sufficient to cover long-term capital improvements and component replacement. With adequate planning and resources, the system can develop sufficient reserve accounts for future needs rather than requiring loans which require interest payments as well as principal payments. A healthy water system usually has an operating ratio (revenue/expense) of 1.2 or higher although this will depend on needed capital improvements.

In FY 2005, 184 million gallons of water was produced at an expense of $746,175. The cost to produce this water was $0.0041/gallon or $4.10 per 1000 gallons.

Improvement Needs

Section 7

7.1 Background

This Section discusses water system components needing improvement and details options and recommendations for such improvements for a 20-year planning period. The 20-year projected water demands that the system must meet are discussed in Section 4. Section 5 discussed planning criteria and goals to which existing capabilities are compared in order to determine improvement needs. Section 6 details existing facilities and deficiencies.

7.2 Water Supply

Currently, the peak daily demand (MDD) is 1.4 mgd. At least 975 gpm must be supplied to the treatment plant for 24 consecutive hours to meet this demand. A supply of approximately 1,000 gpm is withdrawn from the river to meet demands and operate the plant properly allowing for some down-time during the day for filter backwashing and other plant maintenance. Peak daily demand in the community is projected to increase to 2.3 mgd over the next 20 years. To meet the projected MDD and allow for plant operation, a supply of at least 1,740 gpm (2.5 MGD) to the water treatment facility is needed.

7.2.1 Water Rights

Current water permits and certificates held by Tri-City allow for a withdrawal of 2,185.8 gpm (4.87 cfs) from the South Umpqua River. In addition, a storage volume of 95 acre-feet (30.956 million gallons) is permitted from the Galesville Storage Reservoir as needed. During low streamflow years, summer withdrawals from the river are limited to 648.6 gpm (pre-1958 permits) and the Galesville water is required.

In years when sufficient water is available in the South Umpqua River and the Water Master does not impose water restrictions required to enforce the instream water rights, the 4.87 cfs allowed is sufficient for the planning period projections. If the anticipated growth were to continue beyond the planning period, the 4.87 cfs would be exceeded in the year 2037.

Since water restriction periods have occurred on a regular basis and will continue to do so, Tri-City must plan for the situation when withdrawal of natural streamflow is limited to 1.445 cfs (pre-1958 permits) and Galesville storage water is used to supplement the flows. In summer months when the potential for water restrictions exists, the maximum daily demands (MDD) and maximum monthly demands (MMD) also occur. The water required from Galesville must not exceed the permitted use over the entire period of time the water restriction is in place.

Currently, the plant must run an average of 14.4 hours per day at 1000 gpm to meet the MMD of 865,000 gallons per day. When the water restriction is in place, 648.6 gpm out of the 1000 gpm is available from the pre-1958 water permits and 351.4 gpm is the supplemental flow from the Galesville storage. When using 351.4 gpm of the Galesville water for an average of 14.4 hours per day, total use from Galesville will be a little over 300,000 gallons per day. At this rate the 30.956 million gallons of water permitted from Galesville will last a total of 102 days. The current 95 acre-feet permitted from Galesville is adequate for existing conditions.

Over the 20-year planning period, the MMD is projected to increase to an average of 1,417,400 gallons per day. Plant capacity will need to increase to 1,740 gpm to meet peak summer day demands. At the 20-year projected demands, the upgraded plant will need to run 22 hours per day to meet MDDs and 13.6 hours per day to meet MMDs.

When the water restriction is in place, 648.6 gpm out of the 1740 gpm will be available from the pre-1958 water permits and 1091.4 gpm is the supplemental flow from the Galesville storage. When using 1091.4 gpm of the Galesville water for an average of 13.6 hours per day, total use from Galesville will be 890,582 gallons per day. At this rate the 30.956 million gallons of water permitted from Galesville will last a total of 34 days. Since it is possible for the water restriction period to extend longer than 34 days, the current 95 acre-feet permitted from Galesville is not adequate for the planning period projections. The most recent water restriction period was for 28 days in the summer of 2004.

Assuming the worst case (longest) water restriction period could be a total of 75 days from August 1 to October 15, a total of 67 million gallons (205 acre-feet) of water would be needed from the Galesville Storage Reservoir under the projected 20-year demands. If the water restriction period were to last only 60 days, 53 million gallons (164 acre-feet) of water would be needed from Galesville. If the water restriction period were to last only 45 days, 40 million gallons (123 acre-feet) of water would be needed from Galesville. Based on this analysis, additional permitted use from Galesville (above the 95 acre-feet currently permitted) will be needed within 20 years ranging from 28 to 110 acre-feet depending on the length of water restriction planned for.

Recommended Action for Water Rights

The Tri-City Joint Water & Sanitary Authority should continue to pursue the acquisition of all potential water rights that occur upstream of the intake. If the owner of such water right is willing to discuss transferring the right, discussions with the Water Master should begin immediately regarding the potential to change the point of diversion, the place of use, and the change in character of use to municipal. Acquisition of pre-1958 water rights would be especially valuable to the Authority for long-term water supply reliability for the community.

The likelihood for obtaining significant water rights from others is low and Tri-City must also plan for possible purchase of additional water from the Galesville Storage Reservoir. At this time, over 95% of the 4,450 acre-feet of Galesville storage allocated for municipal use remains available so there is little risk in waiting to obtain additional storage. It is recommended that the current permitted use of 95 acre-feet be renewed each period as required so that the priority date is maintained. This is important since it is possible for water shortages to occur in the Galesville Reservoir whereby seniority in permitted use becomes important.

As time progresses the allocation of Galesville water and the length of water restriction periods should be monitored. Should available municipal water in Galesville become low or begin to decrease rapidly from other municipalities purchasing permits, Tri-City should then initiate a new permit for the use of Galesville water. The records of the restriction period lengths can then be used to help determine how much additional storage should be purchased. Based on the projections in this Plan, it is likely that the existing 95 acre-feet permit plus at least an additional 30 acre-feet will be needed near the end of the planning period.

7.2.2 Intake and Pump Station

The water intake and pump station will need to have a capacity of at least 1,740 gpm for the planning period. The existing pump station is able to pump the required amount of water to the plant with both pumps operating at the same time but this will not provide the desired reliability and redundancy. Larger pumps will be needed at some point during the planning period so that the full flow requirement can be provided by a single pump as done now. The second pump will alternate duty cycle with the lead pump. Required plant flows can still be provided during times when one pump is out of service for repairs or maintenance. This is the normal design criteria for raw water pumping facilities.

The intake is deficient in fish screening criteria and low water conditions capacity. The intake also tends to fill with sand and cleaning must be done manually with shovels and buckets. Access to the intake for cleaning can only be accomplished during summer months when river levels are low. To comply with fish screening requirements and increase capacity modifications to the existing intake or a new intake will be required. If the intake is relocated away from its current location, a new raw water pump station may also be needed.

As discussed in Section 6.1.4, the intake structure is a round concrete caisson with 48 holes, with each hole (orifice) having a diameter of 2-inches. During low water level periods in the summer, a portion of the intake is exposed above the stream water level reducing the capacity of the intake. River water depth at the intake is reported to get as shallow as 2 to 2.25 feet during some summers. According to the past Water System Master Plan, the water depth at the intake was only 1 foot in July of 1972. The Galesville Dam was completed in 1986. The dam potentially alleviates such extreme shallow depths and no depth lower than 2 feet has been noted in the last 30 years according to past plans.

The existing capacity of the intake is limited to approximately 1200 gpm during low water conditions when only half (3 rows) of the intake orifices are submerged. When only two rows of orifices are submerged such as would occur when the river depth is only 2 feet, the capacity of the intake is estimated at 900 gpm. The 15-inch intake pipe to the pump station has an estimated capacity of 2,500 gpm at worst case conditions. At the current plant flow of 1000 gpm, no capacity problems have occurred during the past several years although records indicate that past staff had to move boulders and gravel in the river in order to raise water level slightly and maintain a 1000 gpm capacity. Based on estimates of the intake capacity and past observations, it is clear that the existing intake will not allow a 1,740 gpm withdrawal from the river during summer months. The intake is also essentially at or near maximum capacity now under current withdrawal flow rates of 1000 gpm.

Tri-City maintains a Department of State Lands (DSL) permit number RP-14252 which allows in-channel alterations to build up a dike structure to divert water towards the intake structure. The permit authorizes 10-15 yards of gravel to be moved in accordance with the drawings and details of the permit. The alteration is temporary and winter flows return the stream bed to its original condition. In-stream work is limited to the period of July 1 through August 31. The current renewal expires January 10, 2009.

Fish protection criteria at intakes requires fine screening to prevent fish fry from being pulled into the intake. The Oregon Water Resource Department (WRD) has attached a written requirement to install fish screening to Tri-City's water use permit S24600. In addition, fish passage provisions are required for all intakes according to OAR 635-412. The general requirements for an active pump intake screen where fry-sized salmonids are present include an approach velocity of 0.4 feet per second (fps) and screen openings of 1.75 mm with profile wire screens or 2.38 mm (3/32-inch) for woven wire or perforated plate screens. Such screens shall also have a minimum open area of 27%. If a state approved biologist determines that fry-sized salmonids are never present at the site, screen openings shall not exceed ¼-inch with a minimum open area of 40%. With 2-inch openings at the Tri-City intake structure, it is clear that fish screening requirements are not met.

The shallow summer depth of the river at the location of the current intake presents the greatest challenge in determining feasible options for corrections. Intake screens need adequate submergence to avoid the creation of vortices and problems with pulling air into the screens. Screens also need clearance off the bottom of the stream to reduce the amount of sediment pulled into the screen which can create plugging problems.

Options for correcting the intake deficiencies include modifying the existing structure, constructing a new surface water intake, or abandoning the intake and constructing wells near the site. Plans were prepared in the past that would have modified the existing intake to comply with screening requirements in 2000 but the improvements were cut from the overall project due to funding shortfalls. The planned improvements did not have provisions for cleaning the screens and plugging of the fine screens may have been extremely problematic.

Due to the difficult and expensive nature of instream work involved with stream intake repair or reconstruction, it is recommended that any improvements to the intake be designed at least for the full water right allowance of 2,186 gpm rather than the 20-year projected demand of 1,740 gpm. If new major structures or buried intake piping are required, they should be sized to handle 50-year flows of 3,700 gpm. Any new pumping equipment will generally last about 20 years and thus should only be sized for the planning period maximum day flow of 1,740 gpm. If possible, improvements to the existing pump station should include corrections in the safety hazards related to access.

7.2.2.1 Option A – Modify Existing Intake and Pump Station

This option involves modifying the existing concrete intake structure to provide fish screening and increased capacity, providing an air burst or other system to clean the screen, and replacing the intake pumps with higher capacity pumps. Alternatively, the existing pumps could be operated simultaneously to provide increased raw water feed to the plant with a third pump on hand and available for quick installation should one pump fail. If a pump should require replacement, it would then be replaced with a higher capacity pump.

As stated previously, the shallow summer water depths at the intake pose a challenge to having a properly functioning compliant intake screen. According to fish screen criteria developed by the Portland office of the National Marine Fisheries Service (NMFS), tee or drum screens shall be submerged at least one screen diameter below the water surface, and shall have at least one screen diameter clearance off the bottom or from other obstructions. With a minimum water depth of 24-inches, the resulting maximum screen diameter is 8-inches. It would be possible to provide two or more tee screens to obtain the needed capacity however the slot velocities will be high and plugging could be a problem. It also appears that there is insufficient room within the caisson interior to install tee screens along with the necessary fittings to obtain the clearance off the bottom and connect to the existing 15-inch concrete pipe.

A potential screen that will work is a mechanically cleaned cone shaped screen manufactured by ISI Intake Screens, Inc. The screen has approval from NMFS and is designed for shallow water and heavy debris loads. The Model ISI 30-66 cone screen has a diameter of 66-inches and a height of 25-inches. At a flow of 2,720 gpm the approach velocity is 0.3 fps so the capacity is more than adequate. The screen would fit inside the 80-inch diameter concrete caisson for protection. The screen assembly includes a submersible hydraulic motor mounted under the cone which operates exterior brushes that clean the screen. An electronic control panel is included that allows scheduling of automatic cleaning cycles, typically once per day. The hydraulic motor uses food grade oil. A 110-volt power supply is needed or battery packs and solar panels can be used. A disadvantage to this type of screen is that the hydraulic motor is not readily accessible in high water. Should the motor fail when access is not possible, water backwashing and/or air bursting would need to be considered if the screen plugs sufficiently to prevent adequate flow from entering the wet well. The second disadvantage is that the brushes which clean the screen will require periodic replacement, maybe as often as once per year. The third disadvantage is that the screen requires moving parts in the under water portion of the equipment and access is not possible during much of the year.

Intake Modifications, Mechanically Cleaned Cone Screen					
Item	**Description**	**Unit**	**Quantity**	**Unit Cost**	**Construction Cost**
1	Mobilization, Bonds (10%)	LS	All	$9,800.00	$9,800
2	Screen Equipment (screen, hydraulic pump, panel)	LS	1	$19,000.00	$19,000
3	Temporary Sandbagging, Water Diversion	LS	All	$12,000.00	$12,000
4	Equipment Installation	LS	All	$7,500.00	$7,500
5	Core Drilling of Existing Caisson	EA	6	$250.00	$1,500
6	Hydraulic Flush Piping	LF	180	$70.00	$12,600
7	Electrical Extension, 1Ø to Control Enclosure	LF	310	$15.00	$4,650
8	Hydraulic Hose (2)	LF	575	$35.00	$20,125
9	Remove/Replace Concrete Cap (4500 lb)	LS	1	$2,500.00	$2,500
10	Control Enclosure, Concrete Pad	LS	1	$5,000.00	$5,000
11	Clean-Up, O&M Manuals, Misc.	LS	1	$3,500	$3,500
		Construction Total			$98,175
		Contingency			$14,725
		Predesign and Permitting			$5,000
		Engineering			$19,635
		Administration			$3,927
		Project Total			**$141,462**

The other option for modification of the existing intake for fish screening compliance is the installation of stainless steel tee screens. Due to the potential low water depth, the maximum screen diameter must be less than 9-inches. At least 4 screens are required to obtain the needed flowrate. Tee screens will require a control building to house an air compressor and control panel, and 4 separate air lines will need to be installed so that each screen can be air burst cleaned individually. The greatest advantage the tee screen design concept has over the cone screen is that no moving parts are required in the river. Other advantages include no brushes to replace and no hydraulic system to maintain. A disadvantage is that a larger building is required for the air system (unless an air burst system is installed for the cone screen as well).

The tee screen option will require a small building to be constructed to house the air compressor. The air compressor should have an air receiver tank with a capacity of at least 80 gallons and a recharge rate of at least 5.1 acfm. It is expected that a 5 to 7.5 Hp motor will be required with 240V single phase power feed. Automatic valves will be required for the air lines connected to a timer control system to allow programmable intervals between air bursts. Air lines at least 1-inch in diameter will be required from the compressor to each screen. If individual air lines are not routed to each screen, problems will arise with cleaning since the air burst will always tend to go to the cleanest screen.

A concern with any screen system is the accumulation of sand around the screens. This concern is amplified at the existing Tri-City intake since the screen must be mounted close to the bottom due to the low summer water depths. For this reason it is recommended that piping also be installed to allow a hydraulic water burst of the intake to help flush sand away. A portion of the concrete caisson on the downstream side should be removed to provide an escape for flushed sand. In addition, some of the openings in the caisson need to be enlarged to provide increased capacity in low water conditions. Water flushing through the screens may work adequately however a direct spray jet inside the caisson and outside the screens, directed toward the downstream caisson opening, may be more effective in flushing sand away from the screens and outside the caisson. The water flush can be initiated by opening a manual butterfly or gate valve however the valve should be located high enough in elevation to allow access during high water conditions. Staff reports that water elevations are high enough every year to submerge the entire raw water pump station wetwell, and in some years submerge the entire concrete ramp leading down to the intake location. According to the recent (dated October 2001) water treatment plant project record drawings, the river bottom at the intake is at elevation 583 feet, the top of the wetwell is at elevation 597.5, and the concrete ramp extends to elevation 611 feet. This data suggests that the manual valve for water flushing should be located at least above elevation 611. According to the FEMA floodplain map, the 100-year flood elevation in this area is 624 feet. The compressor/control building slab should be above the 100-year floodplain.

Intake Modifications, Air Cleaned Tee Screens					
Item	**Description**	**Unit**	**Quantity**	**Unit Cost**	**Construction Cost**
1	Mobilization, Bonds (10%)	LS	All	$12,900.00	$12,900
2	Screen Equipment (4 tee screens)	LS	1	$10,000.00	$10,000
3	Air Burst Cleaning System	LS	1	$16,000.00	$16,000
4	Air Piping, Compressor to Screens (4 pipes)	LF	575	$25.00	$14,375
5	Equipment Installation	LS	All	$10,000.00	$10,000
6	Temporary Sandbagging, Water Diversion	LS	All	$12,000.00	$12,000
7	Core Drilling of Existing Caisson	EA	6	$250.00	$1,500
8	Hydraulic Flush Piping	LF	180	$70.00	$12,600
9	Electrical Extension, 1Ø to Compressor Bldg.	LF	310	$15.00	$4,650
10	Remove/Replace Concrete Cap (4500 lb)	LS	1	$2,500.00	$2,500
11	Control/Compressor Building	SF	200	$150.00	$30,000
12	Clean-Up, O&M Manuals, Misc.	LS	1	$3,500	$3,500
		Construction Total			$130,025
		Contingency			$19,504
		Predesign and Permitting			$5,000
		Engineering			$26,005
		Administration			$5,201
		Project Total			**$185,735**

Even though the cone-screen option has a lower cost, it is considered less desirable due to the periodic maintenance required and the potential for failure of moving parts under water when access is not possible. If the hydraulic brush cleaning system were to fail during winter months, the screen would quickly become clogged and the intake would cease to function. If an air burst system were to be installed with the cone-screen as a backup cleaning system, the cost would be approximately equal to the tee-screen option. For these reasons, the tee-screen option is the preferred option for adding regulatory compliant fish-screening to the existing intake.

It is important to remember that the potential for plugging of the intake will always be greater when "fish-friendly" fine screening is installed at this site as compared to the existing intake. These concerns stem from the extreme low summer water depths necessitating screen placement very near the streambed where sediment can be a problem. Tri-City may wish to install a second screen assembly higher above the streambed for use in high water conditions. As an alternative, it would be prudent to have a portable trailer mounted, diesel or gasoline motor driven pump that could be used in emergency situations. A suitable pump will have a capacity of 2000 gpm at up to 50 feet of dynamic head with a suction lift capability of at least 10 feet. The normal 100 to 120 gallon diesel tank on these pumps should provide at least 30 hours of continuous operation. A budget price for a trailer-mounted, diesel powered, emergency water pump is $35,000.

In addition to the intake modifications to add screening, larger capacity pumps will be required during the planning period. Initially, it will be possible to provide sufficient raw water flows to the plant using the existing intake pumps however both pumps will need to operate simultaneously. To provide the proper firm capacity, each raw water pump should convey the full plant flow required and the pumps operated in a lead-lag configuration. Initial sizing calculations based on Flygt pumps indicated the need for 60 Hp motors and model NP 3202 MT pumps. New variable frequency drives would be required for the larger pumps and larger conductors would need to be run to the pump station.

According to Flygt, two NP 3202 pumps will fit in the existing 7 foot diameter wetwell however the piping immediately out of the wetwell will have to be modified. The existing installation was not properly installed and the discharge pipes are not symmetrical nor centered at the wetwell. Additionally, the butterfly valve on the 15-inch gravity pipe from the intake caisson projects into the wetwell creating less available space. To gain the required distance between the pumps and allow larger pump installation while avoiding the butterfly valve, the entire pump installation needs to be rotated approximately 20 degrees. This will require all existing 8-inch pipe at the wetwell to be relocated and then reconnected to the 16-inch transmission pipe.

The larger pumps will also require a larger access hatch and new pump removal guide rails.

Intake Modifications, Pump Replacement					
Item	**Description**	**Unit**	**Quantity**	**Unit Cost**	**Construction Cost**
1	Mobilization, Bonds (10%)	LS	All	$9,500.00	$9,500
2	Removal of Existing Equipment	LS	1	$1,000.00	$1,000
3	New Raw Water Pump, 1800 gpm	EA	2	$18,000.00	$36,000
4	Piping Modifications	LS	All	$10,000.00	$10,000
5	Concrete Cutting and Patching	LS	All	$2,000.00	$2,000
6	Motor Starters, VFD Drives	EA	2	$8,000.00	$16,000
7	Electrical (upsize conduit and wire)	LF	835	$25.00	$20,875
8	Pump Installation	LS	All	$7,000.00	$7,000
9	New Lid and Access Hatch	LS	1	$5,000.00	$5,000
10	Clean-Up, O&M Manuals, Misc.	LS	1	$1,500	$1,500
		Construction Total			$108,875
		Contingency			$16,331
		Engineering			$21,775
		Administration			$4,355
		Project Total			**$151,336**

7.2.2.2 Option B – New Intake and Pump Station

This option involves constructing a new intake structure and raw water pump station at a location where adequate water depth occurs to allow installation of typical tee screens. The Authority owned property (tax lots 1500 and 1600) with a boundary along the river bank would allow the intake to be moved approximately 470 feet downstream while still allowing the pump station to lie on the property.

New Raw Water Intake and Pump Station					
Item	Description	Unit	Quantity	Unit Cost	Construction Cost
1	Mobilization, Bonds (10%)	LS	All	$50,000.00	$50,000
2	Submerged Tee Screen	LS	1	$8,000.00	$8,000
3	Temporary Water Diversion and Control	LS	All	$18,000.00	$18,000
4	Screen Support and Protection Piling	EA	9	$5,000.00	$45,000
5	Wetwell, 10 foot diameter, 40 feet deep	LS	1	$175,000.00	$175,000
6	Air Burst Cleaning System	LS	1	$16,000.00	$16,000
7	Air Piping, Compressor to Screen	LF	780	$10.00	$7,800
8	Duplex Pumping Equipment	LS	2	$18,000.00	$36,000
9	Motor Starters, VFD Drives	EA	2	$8,000.00	$16,000
10	Equipment Installation	LS	All	$10,000.00	$10,000
11	Electrical Extension, 1Ø to Compressor Bldg.	LF	310	$15.00	$4,650
12	Electrical Extension, 3Ø to Pumps	LF	930	$25.00	$23,250
13	Control/Compressor Building	SF	200	$150.00	$30,000
14	Piping, Screen to Wetwell	LF	110	$350.00	$38,500
15	Piping, Wetwell to Existing Raw Water Piping	LF	360	$80.00	$28,800
16	Gravel Surfacing, Access Road	LS	1	$15,000	$15,000
		Construction Total			$522,000
		Contingency			$78,300
		Predesign and Permitting			$7,500
		Engineering			$104,400
		Administration			$20,880
		Project Total			**$733,080**

7.2.2.3 Option C – Raw Water Supply Wells

This option looks at the potential for drilling multiple drilled vertical wells near the plant. If 4 successful wells were developed, each with a yield of 500 gpm, the entire community need could be met for the planning period. Wells would offer the benefit of reduced raw water turbidity and stable pH but may have higher levels of iron and manganese.

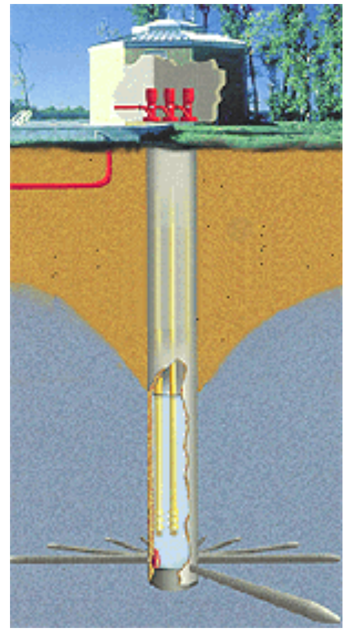
The cost of each well with pumping equipment would be approximately $50,000. The recommended casing diameter for a 500 gpm well is 12-inches. If an 1800 gpm well is possible, the casing should be 20 to 24-inch diameter. Cost to run 3 phase power to each well would vary depending on where the wells were located. Assuming 4 wells, the total project budget with contingency, administrative and engineering fees would be approximately $325,000.

A query of the Water Resource Department (WRD) database shows a total of 90 well logs in a large area surrounding the plant and along the river (T30S R5W Sections 5, 6, 7, 8, T30S R6W Sections 1, 12 and T29S R5W Sections 31, 32). Of these 90 wells, 34 were abandoned and 56 remain. Of the 56 remaining wells, 16 are monitoring wells rather than water wells. Of the 40 water wells, 12 wells (30%) produced no water, the average maximum yield is 6 gpm and the highest yield was 40 gpm. Average depth is 123 feet. The best wells seem to hit a water producing gravel layer at about 20 feet deep. Most well logs show hard sandstone or sandrock below clay or sand/gravel. In some cases a suitable domestic well was developed very close to a well which produced no water.

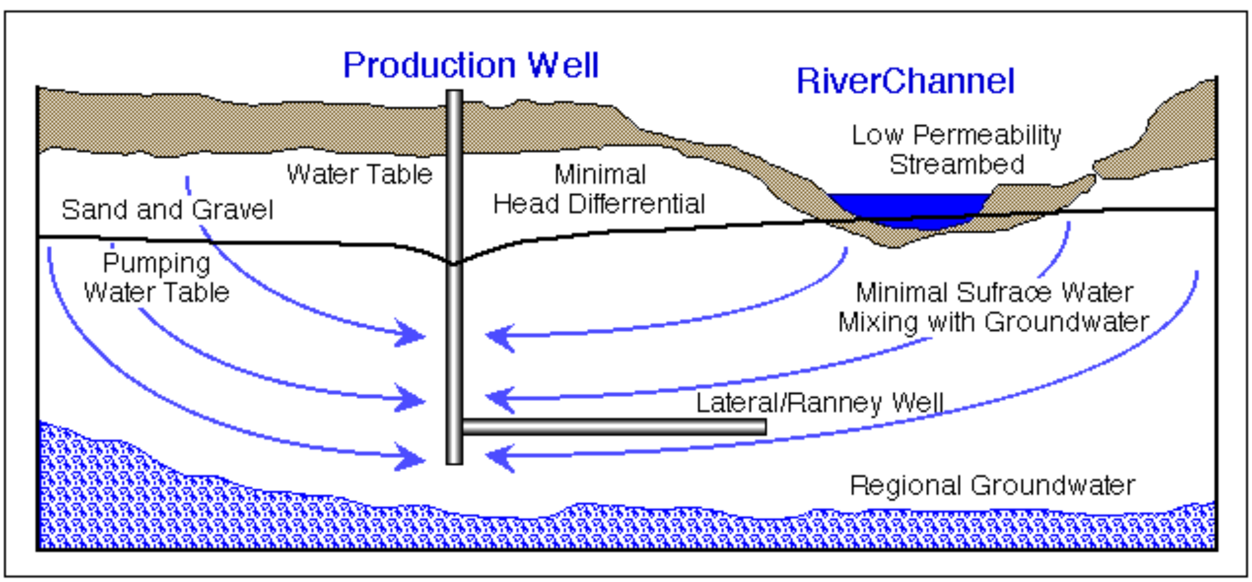
Based on the WRD database well logs, it does not appear that vertical wells near the plant would be successful in providing 1,740 gpm. The Myrtle Creek Comprehensive Plan (Chapter 3) also states that groundwater resources in the area are often insufficient for even domestic use. Additionally, according to the U.S. Geological Survey (Water-Resources Investigations Report 96-4082) groundwater use in the area is not prevalent since most of the basin is underlain by relatively impermeable aquifer units.

7.2.2.4 Option D – Raw Water Horizontal (Ranney) Collection Wells

A Ranney type collection well has the ability to provide greater quantities of water than a typical vertical well. The Ranney well consists of a center concrete caisson with horizontal perforated pipes extending radially into the ground from the bottom of the caisson. Caisson diameters are often 3 to 10 feet in diameter for smaller wells. The horizontal collector pipes are pushed or directionally drilled into the surrounding material. Similar to vertical wells, a fairly porous water-bearing ground material is needed. As with wells and infiltration galleries, the possibility for plugging of the perforated screens or the surround granular material is a possibility over time. As a rule of thumb, about 1 ft^2 of perforated pipe surface area is needed for each gpm of withdrawal.



Budget construction cost of a typical Ranney well with a 10 foot diameter caisson with sufficient horizontal collectors to provide at least 2000 gpm is expected to be approximately 20% higher than the cost for a new water intake and pump station.



As with vertical water wells, the initial information indicates that a successful horizontal collector well is not likely. The potential for a successful Ranney well is however greater than that of a typical vertical drilled well due to the greater length of screen possible in any water bearing strata found. Virtually all available well logs show clay material within the upper 20 feet, followed by sandstone. A few well logs show a gravel layer a few feet thick at depths around 20 to 25 feet. In general, if the a gravel layer is found the well produces small amounts of water; if the gravel layer is not found the well produces no water. There is a potential that a shallow Ranney-type well located near the river at the plant site could produce water if the permeable gravels are found. One or more test wells would be required to determine the feasibility of such a project. It is estimated that a budget of around $20,000 per test well would be needed.

7.2.3 Intake and Pump Station Recommended Alternative

The lowest cost feasible option for raw water supply improvements is to modify the existing intake to provide tee screens with an air burst cleaning system and a supplemental water flush system to help reduce sand accumulation in the caisson. Estimated project cost for the intake modification is $185,735. Regardless of the raw water intake option, a portable trailer-mounted emergency pump capable of full plant flow is recommended. A suitable trailer-mounted pump can be configured and purchased for approximately $35,000.

To complete the needed raw water intake and pump station improvements for the 20-year planning period the raw water pumps and drives will also need to be replaced. The estimated cost for the pump and drive replacement project is $151,336. This project will also correct the safety hazards involved with access into the existing wetwell.

The total recommended project cost is $372,000. This cost is approximately half the cost of constructing a new intake and pump station. Schematics of the potential intake improvements are shown in Figures 7.2-1 and 7.2-2.

7.3 Water Treatment Facility

Details on each process component in the plant along with capacity limitations are discussed in Section 6.2. For the planning period, the Water Treatment Plant needs the capacity to treat a flow of at least 1,740 gpm (2.5 mgd). By the end of the planning period, the plant will run a total of 22 hours per day to meet summer demands based on the Plan population projections. Currently, plant capacity is limited to 1000 gpm due to limitations on pumping control. Summer plant run times of 18 hours per day indicate that current plant capacity is almost reached now.

When flows are increased, the filter bays begin to overflow due to hydraulic piping restrictions in the filtrate piping. The exact flow at which overflows begin is not known but is suspected to be around 1,300 to 1,500 gpm. When a second plant discharge pump is turned on, flows out of the plant approach 1,800 gpm and they begin to cycle frequently due to the difference between the incoming flow and the outgoing flow. Attempts at throttling the discharge pumps to match the incoming plant flow are unsuccessful due to extreme cavitation at the throttling valve. For these reasons, only a single discharge pump can be operated at a time providing a flow of 1000 gpm and the raw water pumps are adjusted to match the flow.

Variable frequency drives are being added to the plant discharge pumps to allow plant output to be maximized. Once this is completed, it should be possible to run the plant at a higher flowrate until the point at which filter overflows begin. This should allow the plant to meet the communities demand for a few more years. At this point, the other plant process deficiencies will begin to limit treatment capacity.

7.3.1 Flocculation / Sedimentation

In general, the sedimentation process is the greatest limitation in the existing plant treatment capacity. Flows above 1,150 gpm will result in surface overflow rates (SOR) and horizontal through basin velocities exceeding recommended limits. In addition, overall detention time will be less than recommended in the sedimentation basin. Flocculation time will be slightly less than recommended at the projected flow. For the planning period, a second sedimentation basin will be required unless alternative filtration technology is used.

New Flocculation/Sedimentation Basin					
Item	**Description**	**Unit**	**Quantity**	**Unit Cost**	**Construction Cost**
1	Mobilization, Bonds (10%)	LS	All	$66,000.00	$66,000
2	Concrete	CY	350	$800.00	$280,000
3	Vertical Flocculators	LS	4	$18,000.00	$72,000
4	Baffle Wall	LS	1	$7,000.00	$7,000
5	Tube Settlers and Finger Weirs	LS	All	$55,000.00	$55,000
6	Sludge Collection Equipment	LS	1	$70,000.00	$70,000
7	Site Piping	LS	All	$55,000.00	$55,000
8	Electrical	LS	All	$35,000.00	$35,000
9	Handrailing	LS	1	$25,000.00	$25,000
		Construction Total			$665,000
		Contingency			$99,750
		Engineering			$133,000
		Administration			$26,600
		Project Total			**$924,350**

For the purposes of this Study, a second basin similar to the existing basin is assumed.

7.3.2 Filtration

After sedimentation, the filtration area becomes the limiting process. The current filter loading rate is 2.1 gpm/ft^2 with a flow of 1000 gpm split equally into the four filters. By limiting the normal filter loading rate to 3.0 gpm/ft^2 (4.0 gpm/ft^2 during backwash of one filter) the maximum capacity of the filters is 1,430 gpm. At the planning period design flow of 1,740 gpm the normal filter loading rate will be 3.7 gpm/ft^2 if no additional filter area is provided. During a filter backwash, the remaining filters would experience a loading rate of 4.9 gpm/ft^2 if all three remaining filters continue to produce water. EPA and AWWA guidelines indicate that loading rates up to 4.0 gpm/ft2 are acceptable for filters in good condition. To limit the filter loading rate to EPA and AWWA guidelines, a flow of 1,430 gpm should not be exceeded.

Even though is it theoretically possible to increase flows through the current filters it is expected that water quality will decrease as flows increase, especially since the current filter media depths are at the lower end of acceptable depths. At current conditions the filters produce water with an average turbidity of 0.05 NTU with spikes after backwashing of around 0.25 to 0.28 NTU. Current filtration goals set forth by EPA include filtrate turbidity always less than 0.1 NTU with spikes after backwash less than 0.3 NTU returning to below 0.1 NTU within 15 minutes. Without additional filter area, the filter loading rate will almost double for the planning period flow. It is expected that meeting the treatment standards will become difficult. In addition, future standards will include Stage 2 of the Long Term Enhanced Surface Water Treatment Rule (LT2) which will require additional treatment performance to remove *Cryptosporidium* based on results of source water monitoring. To properly treat the flow needed for the planning period, additional filtration area or advanced filtration technology is recommended.

The existing filters were installed in 1979 and the media was replaced in the year 2000. Typically, filter media has a design life of 12 to 15 years. It should be expected that all of the existing filters will require media replacement around 2012 if still in use. At current prices, the filter rebuild cost would be approximately $100,000.

7.3.2.1 Option A – Conventional Granular Media Filtration

This option consists of adding several new mixed-media filters similar to the existing filters and modifying the mechanical piping to accommodate the new filters and eliminate hydraulic restrictions in the current piping. The 27 year old backwash pump would also be replaced along with the backwash piping. Improved surface wash piping would also be required with a required backflow preventer and extensions to the new filters. The existing interior flocc/sedimentation equipment would need to be removed. Since the interior basin functions as the flow conduit and splitter box to the filters, new filter influent piping and splitting provisions would be required.

With improvements to the sedimentation process provided by a second exterior flocculation / sedimentation basin, a maximum filter loading rate of 3 gpm/ft^2 is used with the goal of not reducing water quality below that currently being achieved. This will require at least two new filters similar in size to the existing filters.

The layout of the existing building and filter equipment is not conducive to expansion. Additional filters and elimination of the existing hydraulic restrictions will require extensive modifications to virtually all of the interior mechanical piping.

Item	Description	Unit	Quantity	Unit Cost	Construction Cost
Additional Conventional Filtration, 2.5 MGD					
1	Mobilization, Bonds (10%)	LS	All	$53,000.00	$53,000
2	New Filter Tanks and Media	EA	2	$85,000.00	$170,000
3	Influent Piping Modifications and Splitter Box	LS	All	$60,000.00	$60,000
4	New Backwash and Effluent Piping	LS	All	$35,000.00	$35,000
5	Hydraulic Restriction Corrections	LS	All	$45,000.00	$45,000
6	PLC Programming Changes	LS	1	$8,000.00	$8,000
7	Backwash Pump, Drive, and Piping	LS	All	$80,000.00	$80,000
8	Demolition and Removal, Floc/Sed	LS	All	$45,000.00	$45,000
9	Controls, Instrumentation, Electrical	LS	All	$35,000.00	$35,000
10	Walkways, Handrailing	LS	1	$25,000.00	$25,000
		Construction Total			$556,000
		Contingency			$83,400
		Engineering			$111,200
		Administration			$22,240
		Project Total			**$772,840**

The conventional filtration option will require the construction of the second exterior flocculation/ sedimentation basin as discussed in Section 7.3.1 in addition to the additional filters. Improvements to the backwash pond are also recommended to allow better settling of the increased backwash water and to facilitate cleaning. An approximate budget of $65,000 should be included to separate the backwash pond into two cells and relocate the outlet location to provide the greatest detention time feasible. Combining these three project needs results in an estimated budget cost for Option A of $1.76 million.

7.3.2.3 Option B – Package Membrane Microfiltration

This option involves removing all interior tanks (flocculation/sedimentation and 4 filters) and installing a skid-mounted microfiltration membrane package. The existing filtrate piping and control valves would also be removed as would the backwash pump and piping. The membrane equipment would include a blower to assist with backwashing, filtrate suction pumps (also used for backwash), a chemical clean-in-place system, and all necessary valves, instrumentation and control.

Membrane filtration is considered the best available technology for surface water treatment and will produce higher quality filtrate with less operator attention and skill as compared to conventional processes. In general, less chemical addition will be required and backwash water quantities will be lower than would be required for the current conventional processes expanded to higher production.

Due to high winter turbidities, chemical coagulation and sedimentation is still recommended through the exterior basin(s). However, it is anticipated that turbidity in the raw water is low enough for over half the year such that chemical coagulation may not be required. Coagulant chemical costs could be approximately $6000 per year lower than would be experienced with conventional filtration.

The proposed system consists of two USFilter Memcor CMF-S units, each with 128 membrane modules. Expansion to 144 modules each is easily accomplished. The system is designed to provide 2.5 MGD when the water temperature is 10°C (6.78 gpm per module). During summer months when turbidity is low and water temperature is higher, the system should be able to provide 3.3 MGD. By merely adding additional membrane modules capacity could be increased to 2.8 MGD winter and possibly as high as 3.7 MGD summer.

New Skid-Mount Membrane Equipment, 2.5 MGD					
Item	Description	Unit	Quantity	Unit Cost	Construction Cost
1	Mobilization, Bonds (10%)	LS	All	$170,000.00	$170,000
2	Membrane Filtration System	LS	1	$1,250,000.00	$1,250,000
3	Mechanical Piping Modifications	LS	All	$50,000.00	$50,000
4	PLC Programming Changes	LS	1	$5,000.00	$5,000
5	Demolition and Removal, Floc/Sed and Filters	LS	All	$55,000.00	$55,000
6	Electrical, Filtrate/Backwash Pump VFDs	LS	All	$80,000.00	$80,000
7	Equipment Installation Labor	LS	1	$100,000	$100,000
		Construction Total			$1,710,000
		Contingency			$256,500
		Engineering			$342,000
		Administration			$68,400
		Project Total			**$2,376,900**

Based on preliminary planning and discussions with membrane filtration equipment manufacturers it is believed that the second sedimentation basin will not be required if membrane filtration is provided. Even though increasing flows to 1740 gpm through the existing sedimentation basin will result in unacceptable performance using conventional filtration, the performance should be acceptable using membrane filtration. It is expected that filter feed water consistently less than 20 NTU would be possible through the existing basin which is acceptable to the membrane equipment providers. It is also expected that water quality would be higher than the plant is able to produce today.

7.3.2.3 Option C – Immersed Membrane Ultrafiltration

This option involves installing immersed membranes such as manufactured by Zenon into the existing building. The existing tank depth is insufficient to accommodate the membranes so new membrane tanks will be required. All existing equipment in the filter room would be removed. As with Option B, the second exterior flocculation/sedimentation basin is not required.

New Immersed Membrane Equipment, 2.6 MGD					
Item	Description	Unit	Quantity	Unit Cost	Construction Cost
1	Mobilization, Bonds (10%)	LS	All	$270,000.00	$270,000
2	Membrane Filtration System	LS	1	$2,200,000.00	$2,200,000
3	Mechanical Piping Modifications	LS	All	$50,000.00	$50,000
4	PLC Programming Changes	LS	1	$5,000.00	$5,000
5	Demolition and Removal, Floc/Sed and Filters	LS	All	$55,000.00	$55,000
6	Electrical, Filtrate/Backwash Pump VFDs	LS	All	$50,000.00	$50,000
7	Equipment Installation Labor	LS	1	$100,000	$100,000
		Construction Total			$2,730,000
		Contingency			$409,500
		Engineering			$546,000
		Administration			$109,200
		Project Total			**$3,794,700**

The equipment recommended by Zenon consists of a 2-train Z-Box-L240 package with a design capacity of 2.6 MGD. During summer months when turbidity is low and water temperature is higher, the system should be able to provide 3.3 MGD.

7.3.3 Disinfection

Another limiting process in the plant is the contact time available for disinfection. At current flows and conditions, approximately 34 minutes of contact time is provided in the clearwells. Even if pump control and plant operation is changed to keep the clearwells at least 7 feet deep at all times (drops to 4 feet now), contact time will be approximately 2 minutes less than currently provided if flows are increased to 1,740 gpm. At worst case conditions with a water temperature of 5°C, a free chlorine residual of 1 mg/L in the clearwells, and pH > 7.5, a contact time of at least 30 minutes is required. If clearwell levels drop, chlorine residual goes down, or pH goes up, a violation in contact time will occur. It is recommended that additional contact time be provided by adding baffling to the original clearwell. Baffling should increase contact time by 15 to 25 minutes depending on the depth in the clearwell and the extent of baffling provided.

An additional improvement recommended is a switch from gaseous chlorine to liquid sodium hypochlorite. The existing gas system is deficient in several areas including ventilation and safety alarms. When plant flows are increased to meet demand, chlorine consumption will increase and it may be necessary to store more than 2500 pounds on site. This will bring another level of regulation including hazard planning and possibly a gas scrubber. Due to the age of the existing equipment and the safety hazards surround the storage and use of chlorine gas, liquid sodium hypochlorite should be considered. To avoid degradation of bulk hypochlorite over time, the disinfectant can be generated on-site using salt and electricity.

Disinfection System Improvements					
Item	**Description**	**Unit**	**Quantity**	**Unit Cost**	**Construction Cost**
1	Mobilization, Bonds (10%)	LS	All	$19,000.00	$19,000
2	Clearwell Baffling	LS	1	$25,000.00	$25,000
3	On-Site Hypochlorite Generation Equipment	LS	All	$75,000.00	$75,000
4	PLC Programming Changes	LS	1	$3,500.00	$3,500
5	Demolition and Removal of Gas Equipment	LS	All	$10,000.00	$10,000
6	Electrical, Hypo Feed Pumps	LS	All	$35,000.00	$35,000
7	Equipment Installation Labor	LS	1	$20,000	$20,000
		Construction Total			$187,500
		Contingency			$28,125
		Engineering			$37,500
		Administration			$7,500
		Project Total			**$260,625**

7.3.4 Filtration Recommended Alternative

The lowest cost option for improvements necessary to treat 1740 gpm is Option A consisting of a second exterior sedimentation basin, 2 new filters to supplement the 4 existing filters, and improvements to the backwash pond. The estimated project cost for Option A is $1.76 million.

The second lowest cost option is Option B consisting of submerged microfiltration membranes. Improvements to the backwash pond should also be considered with membrane filtration even though less backwash waste will be generated. The estimated project cost for Option B is $2.43 million including backwash pond improvements.

The highest cost is Option C using ultrafiltration immersed membranes. The estimated project cost for Option C is $3.86 million including backwash pond improvements.

With Option A conventional rapid sand filtration, non-personnel operation costs will increase by approximately 70% over current costs due to increased pumping and chemical costs. To replace the filter media every 15 years at current prices, a media replacement budget of $10,400 per year is needed. With membrane filtration, additional pumping from the membranes to the clearwell is required however this operational cost is largely offset by the elimination of the large backwash pump required for media filters. The cost for periodic (monthly) membrane chemical cleaning is offset by the reduction in chemical coagulant costs. To replace membrane modules every 10 years at current prices, a membrane replacement budget of $16,600 per year is needed. The largest difference in operation cost between media filters and membrane filters will be staff time required for proper plant operation.

With conventional coagulation/sedimentation/media filtration, successful treatment is extremely dependant on precise chemical dosages and close supervision. Significant time and expertise is required to continuously monitor raw water quality and adjust chemical dosages. With increased flows, operator time of 8 hours per day should be planned. If improper chemical dosage occurs, filtrate turbidity will rise causing an alarm condition and a cease in production. For this reason, advanced operator skills are required.

With membrane filtration, improper coagulation will merely result in shorter filter run times between backwashes and no degradation in water quality results. The membranes provide a physical barrier to all particles larger than 0.1 micron regardless of chemical dosage. It will still be important during high turbidity periods to provide chemical coagulation to increase filter runs but this will only be required to minimize electrical costs, not to ensure against turbidity violations. If high TOC periods occur, coagulation may be required to minimize disinfection by-product formation. Typically, operators of intermediate skill can produce safe drinking water with membrane filtration. For the membrane filtration options, approximately 4 hours per day of operator time should be planned for. At an hourly rate of $35 including benefits, membrane filtration has the potential to save $51,000 per year in staff time.

Option C is considered cost prohibitive at this time. Option B uses advanced technology and has the highest potential for meeting any future treatment standards imposed but has an estimated capital cost $676,710 higher than Option A. Option B has an estimated annual O&M cost approximately $45,000 lower than Option A when considering staff time. This will allow staff to conduct other tasks in the water system which currently cannot be accomplished without additional personnel. The present worth value of $45,000 per year is $669,500 assuming 3% inflation over 20 years. When long-term O&M costs are considered, Option B using membrane microfiltration has approximately the same present worth cost as Option A.

Since membrane filtration is the preferred option in terms of water quality, treatment compliance, expandability for future capacity increases, and has the same present worth cost as Option A; Option B is the recommended treatment option.

Schematics of potential plant improvements are shown in Figures 7.3-1 and 7.3-2.

7.4 Treated Water Storage

Section 5.2.3 discusses storage capacity requirements including storage needed for equalization, emergency, and fire reserve. In general, the storage goal is to provide 1.25 times the maximum daily water demand (MDD) plus 540,000 gallons of reserve storage for fire protection equal to that needed to supply 3,000 gpm for 3 hours. Section 6.3 details existing storage facilities.

7.4.1 Existing Storage Vs. Storage Need

A total storage of 2.02 million gallons exists in the system when the tanks are full based on the current water level control settings. By raising the maximum water level settings closer to the overflow point (6-inches below overflow), an additional 51,000 gallons of total storage could be realized. Even if water depths in the tanks are maximized, the system is still deficient by almost 230,000 gallons today. As water demands increase, the system will be deficient in storage capacity by 1.4 million gallons at the end of the planning period.

Table 7.4.1-1 – Water Storage Improvement Needs

Year	Existing Storage (gallons)	Required Storage (gallons)	Additional Storage Need
2005	2,017,300	2,296,250	278,950
2010	2,017,300	2,527,035	509,735
2015	2,017,300	2,788,150	770,850
2020	2,017,300	3,083,575	1,066,275
2025	2,017,300	3,417,820	1,400,520

It is important to note that the lack of separate inlet/outlet pipes and/or baffling in the tanks (all except October Dr. Tank) results in poor mixing and low chlorine residuals. Currently, the Authority must allow the tanks to drop to about half full before filling again in order to maintain required detectable chlorine residuals. During these times when the tanks are half full, the system is extremely vulnerable with a storage deficiency of over 1 million gallons.

The Back Acres high level service area must be considered separately from the lower level main service area. Currently, the high level service area has 87,000 gallons of storage in the Back Acres Tank. Being entirely a residential service area, a reduced fire reserve storage of 180,000 gallons should be provided (1,500 gpm for 2 hours). In addition, equalization and emergency storage should be 1.25 x 812 gpd/EDU. Assuming approximately 150 homes now, total storage should be 330,000 gallons. By the end of the planning period, storage should be 430,000 gallons assuming 250 homes. To provide adequate storage for the planning period, the high level service area needs an additional 340,000 gallons of storage.

To correct the storage deficiencies in Tri-City, two steps need to be taken. First, improvements need to be made at the Walnut Street, Aker Drive, and Back Acres Storage Tanks to improve mixing. Second, additional storage capacity of at least 1.4 million gallons needs to be added to the system. At least two storage tanks are needed. One tank with a capacity of around 340,000 gallons should be constructed in the high level service area (430,000 gallons if existing tank is removed). A second tank with a capacity of 1.1 million gallons should be constructed in the main service area.

7.4.2 Improvements to Existing Tanks

Both the Aker Dr. Tank and the Walnut St. Tank need exterior painting and improvements to the tank piping to promote mixing and prevent low chlorine residuals. The Back Acres Tank is in better condition but will still likely require repainting within the next 20 years. The Back Acres Tank also needs piping improvements to promote mixing and requires fencing around the site.

Walnut St. Tank Exterior Repainting					
Item	**Description**	**Unit**	**Quantity**	**Unit Cost**	**Construction Cost**
1	Construction Facilities and Temporary Controls	LS	All	$3,000	$3,000
2	Surface Preparation	SF	8000	$2.30	$18,400
3	Coating Material and Application	SF	8000	$1.20	$9,600
4	Disinfection	LS	All	$1,200	$1,200
		Construction Total			$32,200
		Contingency			$4,830
		Engineering			$6,440
		Administration			$1,288
		Project Total			**$44,758**

Aker Dr. Tank Exterior Repainting					
Item	**Description**	**Unit**	**Quantity**	**Unit Cost**	**Construction Cost**
1	Construction Facilities and Temporary Controls	LS	All	$3,000	$3,000
2	Surface Preparation	SF	7725	$2.30	$17,768
3	Coating Material and Application	SF	7725	$1.20	$9,270
4	Disinfection	LS	All	$1,200	$1,200
		Construction Total			$31,238
		Contingency			$4,686
		Engineering			$6,248
		Administration			$1,250
		Project Total			**$43,420**

Back Acres Tank Exterior Repainting and Fencing					
Item	**Description**	**Unit**	**Quantity**	**Unit Cost**	**Construction Cost**
1	Construction Facilities and Temporary Controls	LS	All	$3,000	$3,000
2	Surface Preparation	SF	2720	$2.30	$6,256
3	Coating Material and Application	SF	2720	$1.20	$3,264
4	Fencing	LF	300	$25.00	$7,500
5	Disinfection	LS	All	$1,200	$1,200
		Construction Total			$21,220
		Contingency			$3,183
		Engineering			$4,244
		Administration			$849
		Project Total			**$29,496**

Tank mixing can be improved by adding piping and check valves to the tank inlets to separate the inlet and outlet piping locations at the tank. The least costly method will be to install the check valves and piping inside the tank.

Tank Mixing Improvements, Typical Tank					
Item	**Description**	**Unit**	**Quantity**	**Unit Cost**	**Construction Cost**
1	Construction Facilities and Temporary Controls	LS	All	$1,000	$1,000
2	Check Valves	EA	2	$1,000.00	$2,000
3	Piping, Fittings	LF	20	$75.00	$1,500
4	Painting (Touch-Up), Welding	LS	All	$1,000.00	$1,000
5	Disinfection, Cleaning	LS	All	$3,000	$3,000
		Construction Total			$8,500
		Contingency			$1,275
		Engineering			$1,700
		Administration			$340
		Project Total			**$11,815**

7.4.3 Additional Storage Alternatives

As discussed above, a 340,000 gallon tank is needed in the Back Acres high level service area and a 1.0 million gallon tank is needed in the lower level main pressure zone.

New 0.34 MG Storage Tank, Bolted Glass Fused Steel, New Location					
Item	**Description**	**Unit**	**Quantity**	**Unit Cost**	**Construction Cost**
1	Mobilization, Bonds (10%)	LS	All	$53,000.00	$53,000
2	Tank, Slab, Ladders, Aluminum Dome Roof	LS	1	$265,000.00	$265,000
3	Valves and Valve Vault	LS	All	$15,000.00	$15,000
4	Level Monitoring Equipment, Telemetry	LS	1	$10,000.00	$10,000
5	Site Work, Grading, Excavation, Fill, Gabion Wall	LS	All	$85,000.00	$85,000
6	Electrical to Site	LF	275	$20.00	$5,500
7	Fencing	LF	300	$25.00	$7,500
8	Piping to Site, 12-Inch	LF	670	$75.00	$50,250
9	Gravel Surfacing, Access Road	CY	145	$25.00	$3,625
10	Misc, Disinfection, Testing	LS	All	$10,000	$10,000
		Construction Total			$504,875
		Contingency			$75,731
		Engineering			$100,975
		Administration			$20,195
		Land and Easements			$50,000
		Project Total			**$751,776**

Few options exist for placing a new tank in the Back Acres area at the proper elevation to match the existing tank. One potential site for a new tank is at the southeast corner inside the UGB as shown in Figure 7.5-2, approximately 700 feet to the south of the existing tank. This site appears to have less significant sloping than other nearby areas but is a privately owned parcel. Another option is to build the tank on the site currently owned by the Authority where the existing Back Acres Tank is located. Due to the size and slope of the lot the existing tank would likely need to be removed. In the event the existing tank is removed, a larger 430,000 gallon tank will be required. Even considering the demolition cost and larger tank, the existing tank site is the lower cost option unless land was donated for another site.

New 0.43 MG Storage Tank, Bolted Glass Fused Steel, Exist. Tank Location					
Item	**Description**	**Unit**	**Quantity**	**Unit Cost**	**Construction Cost**
1	Mobilization, Bonds (10%)	LS	All	$53,000.00	$53,000
2	Tank, Slab, Ladders, Aluminum Dome Roof	LS	1	$295,000.00	$295,000
3	Valves and Valve Vault, Piping	LS	All	$25,000.00	$25,000
4	Level Monitoring Equipment, Telemetry	LS	1	$5,000.00	$5,000
5	Temporary Tank and Controls	LS	1	$15,000.00	$15,000
6	Site Work, Grading, Excavation, Fill, Gabion Wall	LS	All	$50,000.00	$50,000
7	Demolition and Removal of Exist. 0.1 MG Tank	LS	All	$40,000.00	$40,000
8	Fencing	LF	300	$25.00	$7,500
9	Gravel Surfacing	CY	50	$25.00	$1,250
10	Misc, Disinfection, Testing	LS	All	$10,000	$10,000
		Construction Total			$501,750
		Contingency			$75,263
		Engineering			$100,350
		Administration			$20,070
		Land and Easements			$0
		Project Total			**$697,433**

Many alternatives exist for locating the 1.1 million gallon tank needed in the main pressure zone. Essentially, any available and suitable site between the October Dr. Tank at the south to above Norton Ln. at the north end will work. Due to the fact that several tanks already exist in fairly disperse locations, the location of the new tank is not important hydraulically. Finding an available site at the proper elevation becomes the deciding factor in locating the tank. Seven potential sites are shown in Figure 7.5-2. The average distance to connect these sites to the system is 890 feet so this average is used in estimating the cost. Using a site adjacent to the existing October Dr. Tank or Walnut St. Tank will save approximately $75,000 in piping.

New 1.1 MG Storage Tank, Bolted Glass Fused Steel, New Location					
Item	**Description**	**Unit**	**Quantity**	**Unit Cost**	**Construction Cost**
1	Mobilization, Bonds (10%)	LS	All	$78,000.00	$78,000
2	Tank, Slab, Ladders, Aluminum Dome Roof	LS	1	$500,000.00	$500,000
3	Valves and Valve Vault	LS	All	$15,000.00	$15,000
4	Level Monitoring Equipment, Telemetry	LS	1	$10,000.00	$10,000
5	Site Work, Grading, Excavation, Fill, Gabion Wall	LS	All	$85,000.00	$85,000
6	Electrical to Site	LF	890	$20.00	$17,800
7	Fencing	LF	450	$22.00	$9,900
8	Piping to Site, 12-Inch	LF	890	$75.00	$66,750
9	Gravel Surfacing, Access Road	CY	200	$25.00	$5,000
10	Misc, Disinfection, Testing	LS	All	$10,000	$10,000
		Construction Total			$797,450
		Contingency			$119,618
		Engineering			$159,490
		Administration			$31,898
		Land and Easements			$70,000
		Project Total			**$1,178,456**

7.5 Water Distribution Piping System Deficiencies

The piping network was analyzed using Haestad Methods WaterCAD. Node elevations were obtained from aerial photogrammetry mapping with 2 foot contours. The system was analyzed for ability to provide fire flows during a period of maximum daily demand. Desired fire flows are 3,000 gpm at large commercial buildings and 1,000 gpm for residential areas. No area in the public system can fall below 20 psi. The minimum pressure must be maintained at all times, even during periods of fire flow demand.

Tank levels were varied during the analysis between full depth and current minimum operating depth (50% full). Areas are considered deficient if the minimum required fire flows cannot be obtained when the tanks are at minimum operating depth.

7.5.1 Existing Pressure Deficiencies

Per OAR 333-061-061-0050(8)(e) and 333-061-0025(7) the District must ensure that no less than 20 psi exists in the public water system piping at all service connections (at property line) at all times.

There are no areas in the public system where less than 20 psi occurs under normal conditions. The lowest system pressure occurs at the upper end of Luke Court where pressures approach 21 psi when the storage tanks are at lowest levels. Homes served off this piping must have individual household booster pumps. If modifications are done to the existing storage tanks to provide improved mixing, the storage tanks water low level set-points can be raised providing a slight increase in the worst case pressure conditions. The second lowest pressure occurs at Rollin Court. The piping at the end of the cul-de-sac on Rollin Court extends to the maximum service elevation resulting in approximately 25 psi during low storage tank levels. Preventing pipe extension above the maximum service elevation will prevent pressure problems from occurring in the future. Development located above the maximum service elevation will require booster pump stations and possibly high level storage tanks.

7.5.2 Existing Fire Flow Deficiencies

Three areas in the system have fire flow deficiencies due to undersized piping and/or lack of proper looping. The first area is at the south end of the service boundary around Arnold Lane, Esther Ct., Briggs Dr., Celestial Dr, and Matthews. This area primarily consists of older 6-inch AC pipe with most streets terminating in dead-ends. Most of this area is deficient in fire flow availability even when the tanks are at full depth; however fire flows on Esther Ct. are suitable when the tanks are 80% full or greater.

The second problem area is around Arburnia St., Cook St., Allan St., Luke Ct., Rollin Ct., and Margie Ct. This area is isolated from the rest of the system by 6-inch piping. When the tanks are full, the problem areas are limited to Arburnia St. and Luke Ct. When the tanks are half full, the fire flow deficiencies spread to Cook St., Margie Ct., and Rollin Ct.

The third area is on Ridgewood Place. The existing 4-inch pipe cannot provide adequate fire flows and no hydrant exists.

Several other areas in the system lack fire protection only due to lack of fire hydrants. The current fire protection situation is shown in Figure 7.5-1.

7.6 Water Distribution System Alternatives

7.6.1 South Area Improvement Alternatives

Piping improvements are required to create looping and provide the minimum required fire flows to the south area (Arnold Ln., Celestial, Briggs Dr., Matthews Ln., etc). Currently, some of the hydrants in this area can only provide 550 gpm of fire flow. Priority 1 Improvements correct the immediate fire flow deficiency.

7.6.1.1 Priority 1 Option A – Replace Old Hwy. 99 Piping

One option is to replace the existing 6-inch along Old Hwy. 99 from Gael Ln. to Celestial Ln. with 10-inch pipe, replace the existing 6-inch on Celestial Ln. with 8-inch pipe, and replace the existing 4-inch pipe at the end of Matthews Ln. with 8-inch pipe. This option provides at least 1000 gpm to each hydrant in the area but does not eliminate any of the 7 dead-end pipes in the area. An advantage to this option is that approximately 2,280 feet of old 6-inch AC piping is replaced.

Piping Improvements - South End Option A					
Item	**Description**	**Unit**	**Quantity**	**Unit Cost**	**Construction Cost**
1	Mobilization, Bonds (10%)	LS	All	$26,000.00	$26,000
2	New 10-Inch along old Hwy. 99	LF	1100	$75.00	$82,500
3	New 8-Inch along Celestial	LF	1180	$60.00	$70,800
4	New 8-Inch along Matthews	LF	690	$60.00	$41,400
5	Service Reconnects	EA	22	$800.00	$17,600
6	Hydrant Reconnects	EA	7	$1,500.00	$10,500
7	Traffic Control	LS	All	$10,000.00	$10,000
8	Testing, Disinfection, Misc.	LS	All	$5,000.00	$5,000
		Construction Total			$263,800
		Contingency			$39,570
		Engineering			$52,760
		Administration			$10,552
		Project Total			**$366,682**

7.6.1.2 Priority 1 Option B – Loop East Side

The other option is to install 8-inch piping along easements from Mar-Wan Dr. to the end of Celestial Ln. connecting to Arnold Ln. and Esther Ct. This will correct all fire flow deficiencies on the east side of Highway 99 and eliminate 4 dead-end pipes. To correct fire flow deficiencies at the west end of Matthews Ln., the existing 4-inch pipe will also need to be replaced with 8-inch. Option B is the least costly option however easements will be required. Option B is also the preferred route since significant dead-ends will be eliminated and a loop will be created which would allow continued service to residents should a line break occur along the highway piping. Option A will need to be constructed if easements cannot be obtained.

Piping Improvements - South End Option B					
Item	Description	Unit	Quantity	Unit Cost	Construction Cost
1	Mobilization, Bonds (10%)	LS	All	$14,000.00	$14,000
2	New 8-Inch Celestial to Mar-Wan, East Side	LF	1115	$60.00	$66,900
4	New 8-Inch along Matthews	LF	690	$60.00	$41,400
5	Service Reconnects	EA	7	$800.00	$5,600
6	Hydrant Reconnects	EA	2	$1,500.00	$3,000
7	Traffic Control	LS	All	$2,000.00	$2,000
8	Testing, Disinfection, Misc.	LS	All	$5,000.00	$5,000
		Construction Total			$137,900
		Contingency			$20,685
		Engineering			$27,580
		Easement Acquisition			$20,000
		Administration			$5,516
		Project Total			**$211,681**

7.6.1.3 Priority 2 – Loop West Side

As a second priority, the dead-end pipes on the west side of Highway 99 can be eliminated by installing 8-inch pipe from the end of Matthews near sewer pump station #8 north to Pruner Road and also connecting to the end of Briggs Dr. through easements. This improvement eliminates 3 dead-ends, provides looping to create redundant flow paths for all lots east of Highway 99 and south of Pruner Road, and allows for further development of this area.

Piping Improvements - South End Priority 2					
Item	Description	Unit	Quantity	Unit Cost	Construction Cost
1	Mobilization, Bonds (10%)	LS	All	$14,000.00	$14,000
2	New 8-Inch Matthews to Pruner Rd.	LF	1135	$60.00	$68,100
4	New 8-Inch to connect Briggs Dr.	LF	520	$60.00	$31,200
5	Service Reconnects	EA	2	$800.00	$1,600
6	Hydrant Reconnects	EA	1	$1,500.00	$1,500
7	Directional Bore Under Pruner Rd.	LF	100	$175.00	$17,500
8	Testing, Disinfection, Misc.	LS	All	$5,000.00	$5,000
		Construction Total			$138,900
		Contingency			$20,835
		Engineering			$27,780
		Easement Acquisition			$15,000
		Administration			$5,556
		Project Total			**$208,071**

7.6.2 Walnut St. Area Improvement Alternatives

Piping improvements are also needed for this area to correct fire flow deficiencies. Under current conditions when the storage tanks are near the minimum normal operating depths, fire flows are limited to 230 gpm on Luke Court, 675 gpm on Rollin Court, 620 gpm at the intersection of Cook and Clark Streets, and 700 gpm along Mona Street.

7.6.2.1 Priority 1

Priority 1 Improvements in this area include replacing the existing 2-inch piping on Cooks Street between Carte and Walnut with 8-inch, installing 12-inch on Arburnia from Walnut south to tie-in to the existing 8-inch north of Rollin Court (replacing portions of 1-inch and 6-inch), installing 8-inch through an existing easement from Mona to the east end of Conrad, and replacing the existing short sections of 6-inch pipe east of Cook Street on Mona Street and Clark Street.

Piping Improvements - Arburnia Street, 12-Inch					
Item	**Description**	**Unit**	**Quantity**	**Unit Cost**	**Construction Cost**
1	Mobilization, Bonds (10%)	LS	All	$9,000.00	$9,000
2	New 12-Inch, Walnut to Mona	LF	800	$80.00	$64,000
3	Service Reconnects	EA	12	$800.00	$9,600
4	Hydrant Reconnects	EA	2	$1,500.00	$3,000
5	Testing, Disinfection, Misc.	LS	All	$4,000.00	$4,000
		Construction Total			$89,600
		Contingency			$13,440
		Engineering			$17,920
		Easement Acquisition			$5,000
		Administration			$3,584
		Project Total			**$129,544**

Piping Improvements - Cook Street					
Item	**Description**	**Unit**	**Quantity**	**Unit Cost**	**Construction Cost**
1	Mobilization, Bonds (10%)	LS	All	$5,000.00	$5,000
2	New 8-Inch, Walnut to Carte	LF	570	$60.00	$34,200
3	Service Reconnects	EA	12	$800.00	$9,600
4	Hydrant Reconnects	EA	1	$1,500.00	$1,500
5	Testing, Disinfection, Misc.	LS	All	$4,000.00	$4,000
		Construction Total			$54,300
		Contingency			$8,145
		Engineering			$10,860
		Administration			$2,172
		Project Total			**$75,477**

Piping Improvements - Clark, Mona, Conrad					
Item	**Description**	**Unit**	**Quantity**	**Unit Cost**	**Construction Cost**
1	Mobilization, Bonds (10%)	LS	All	$3,000.00	$3,000
2	New 8-Inch	LF	375	$60.00	$22,500
3	Service Reconnects	EA	2	$800.00	$1,600
4	Hydrant Reconnects	EA	1	$1,500.00	$1,500
5	Testing, Disinfection, Misc.	LS	All	$3,000.00	$3,000
		Construction Total			$31,600
		Contingency			$4,740
		Engineering			$6,320
		Administration			$1,264
		Project Total			**$43,924**

Priority 1 Improvements will correct all deficient fire flows in this area with the exception of the Luke Court area and will eliminate 3 dead-ends. The new 12-inch piping on Arburnia will also function to extend the larger diameter east loop piping. The east loop ties the storage tanks together and provides redundant connections to supply from the plant. The east loop currently extends from the central service area (plant and Aker Dr. Tank) north to Norton Lane but sections of older 6-inch piping still remain between near Clark Street at the south and Walnut Street to the north.

7.6.2.2 Priority 2

Priority 2 Improvements will correct the fire flow deficiency in the Luke Court area and essentially complete the 12-inch east looping. The proper infrastructure will also then be in place to allow further development of the large amount of undeveloped land above (east of) Luke Court and Arburnia St.

To complete the large diameter piping and provide fire flows to the Luke Court area, new 12-inch piping is needed from the existing 12-inch termination near NE Donald Terrace and Chickering to the intersection of Clark and Cook Streets. Since the fire hydrant on Luke Court is near the maximum service elevation, 12-inch piping is also needed to the fire hydrant to prevent a drop in pressure below 20 psi when this hydrant is used. The preferred 12-inch piping route is through easements near Fred Way to the south end of Allen Street and then along Allen Street up to Clark. This will eliminate 2 dead-end pipes and replace some 6-inch and 2-inch pipe. Alternatively, the route could go along Chickering Street and then along Clark Street to the intersection at Cook Street.

Piping Improvements - Donald Terrace to Allen to Clark, 12-Inch					
Item	**Description**	**Unit**	**Quantity**	**Unit Cost**	**Construction Cost**
1	Mobilization, Bonds (10%)	LS	All	$13,000.00	$13,000
2	New 12-Inch	LF	1245	$80.00	$99,600
3	Service Reconnects	EA	7	$800.00	$5,600
4	Hydrant Reconnects	EA	1	$1,500.00	$1,500
5	Testing, Disinfection, Misc.	LS	All	$7,000.00	$7,000
		Construction Total			$126,700
		Contingency			$19,005
		Engineering			$25,340
		Easement Acquisition			$5,000
		Administration			$5,068
		Project Total			**$181,113**

Piping Improvements - Luke Ct.					
Item	**Description**	**Unit**	**Quantity**	**Unit Cost**	**Construction Cost**
1	Mobilization, Bonds (10%)	LS	All	$4,000.00	$4,000
2	New 12-Inch	LF	365	$80.00	$29,200
3	Service Reconnects	EA	5	$800.00	$4,000
4	Hydrant Reconnects	EA	1	$1,500.00	$1,500
5	Testing, Disinfection, Misc.	LS	All	$2,000.00	$2,000
		Construction Total			$40,700
		Contingency			$6,105
		Engineering			$8,140
		Administration			$1,628
		Project Total			**$56,573**

Another Priority 2 Improvement recommended for the Walnut Street area is to replace the existing 4-inch pipe on Arburnia St. north of Walnut with 8-inch pipe and install a fire hydrant at the end of the street. This will bring fire protection service to the northernmost lots on Arburnia Street. With development planned in the near future for the area surrounding the Walnut Street Tank, looping of this future development into the Arburnia Street piping should be required of the development.

Piping Improvements - Arburnia Street, 8-Inch					
Item	**Description**	**Unit**	**Quantity**	**Unit Cost**	**Construction Cost**
1	Mobilization, Bonds (10%)	LS	All	$6,000.00	$6,000
2	New 8-Inch, Walnut to Carte	LF	615	$60.00	$36,900
3	Service Reconnects	EA	13	$800.00	$10,400
4	New Hydrants	EA	1	$3,000.00	$3,000
5	Testing, Disinfection, Misc.	LS	All	$4,000.00	$4,000
		Construction Total			$60,300
		Contingency			$9,045
		Engineering			$12,060
		Administration			$2,412
		Project Total			**$83,817**

7.6.3 Other Priority 1 Improvement Alternatives

Piping improvements are also needed on Ridgewood Place to provide fire protection at the cul-de-sac. The existing 4-inch pipe should be replaced with 8-inch and a fire hydrant installed near the end of the street. Without new piping, a fire hydrant at this location will only provide approximately 480 gpm. With the improvements, over 2000 gpm will be possible. No alternative exist to provide fire protection at the cul-de-sac other than obtaining multiple easements and extending pipe from Woodcrest or Aker Drive to allow placement of a fire hydrant. Such alternatives would be more difficult and costly.

Piping Improvements - Ridgewood Place					
Item	**Description**	**Unit**	**Quantity**	**Unit Cost**	**Construction Cost**
1	Mobilization, Bonds (10%)	LS	All	$8,000.00	$8,000
4	New 8-Inch	LF	805	$65.00	$52,325
5	Service Reconnects	EA	11	$800.00	$8,800
6	New Hydrant	EA	1	$3,000.00	$3,000
8	Testing, Disinfection, Misc.	LS	All	$4,000.00	$4,000
		Construction Total			$76,125
		Contingency			$11,419
		Engineering			$15,225
		Administration			$3,045
		Project Total			**$105,814**

7.6.4 Priority 3 Improvement Alternatives

Various other piping improvements would enhance system looping, replace older 2-inch piping, and potentially reduce leakage.

New piping can be installed on Back Acres Road from Jodee Street to the existing 8-inch dead-end at the newly created Back Acres Planned Community. Connection can also be made from this pipe to the existing dead-end 6-inch pipe running north off Plin Street. This piping will eliminate 2 dead-ends and allow development along Back Acres.

Piping Improvements - Back Acres					
Item	**Description**	**Unit**	**Quantity**	**Unit Cost**	**Construction Cost**
1	Mobilization, Bonds (10%)	LS	All	$8,000.00	$8,000
2	New 8-Inch	LF	1170	$60.00	$70,200
3	Testing, Disinfection, Misc.	LS	All	$3,000.00	$3,000
		Construction Total			$81,200
		Contingency			$12,180
		Engineering			$16,240
		Easement Acquisition			$2,000
		Administration			$3,248
		Project Total			**$114,868**

The existing 2-inch piping on Peacock Lane off Briggs Drive should be replaced with 6-inch. This will eliminate the undersized piping and replace it with rubber gasketed pipe less susceptible to leakage.

Piping Improvements - Peacock Lane					
Item	**Description**	**Unit**	**Quantity**	**Unit Cost**	**Construction Cost**
1	Mobilization, Bonds (10%)	LS	All	$4,000.00	$4,000
4	New 6-Inch	LF	575	$50.00	$28,750
5	Service Reconnects	EA	4	$800.00	$3,200
8	Testing, Disinfection, Misc.	LS	All	$3,000.00	$3,000
		Construction Total			$38,950
		Contingency			$5,843
		Engineering			$7,790
		Administration			$1,558
		Project Total			**$54,141**

Replacement of the dead-end 2-inch pipe at the south end of Taylor Street should be considered. By looping new 8-inch pipe from Taylor to Corwin along Pruner Road, the undersized piping can be replaced, the dead-end eliminated, and new services potentially installed to serve the south side of Pruner Road. It may then be possible to eliminate the long run of 2-inch pipe at the south side of Pruner Road. This piping will improve service and reduce leakage potential.

Piping Improvements - Taylor to Corwin Loop					
Item	**Description**	**Unit**	**Quantity**	**Unit Cost**	**Construction Cost**
1	Mobilization, Bonds (10%)	LS	All	$5,300.00	$5,300
4	New 8-Inch	LF	630	$65.00	$40,950
5	Service Reconnects	EA	5	$800.00	$4,000
8	Testing, Disinfection, Misc.	LS	All	$3,000.00	$3,000
		Construction Total			$53,250
		Contingency			$7,988
		Engineering			$10,650
		Administration			$2,130
		Project Total			**$74,018**

Several homes are currently served by a 2-inch dead-end heading south off Susan Street. Replacing this pipe with new 8-inch pipe which then loops to Taylor Street will eliminate the dead-end and promote development of the vacant land in this area and north of Glenmore Street.

Piping Improvements - Taylor to Susan Loop					
Item	**Description**	**Unit**	**Quantity**	**Unit Cost**	**Construction Cost**
1	Mobilization, Bonds (10%)	LS	All	$7,500.00	$7,500
4	New 8-Inch	LF	1030	$60.00	$61,800
5	Service Reconnects	EA	1	$800.00	$800
8	Testing, Disinfection, Misc.	LS	All	$4,000.00	$4,000
		Construction Total			$74,100
		Contingency			$11,115
		Engineering			$14,820
		Easement Acquisition			$6,000
		Administration			$2,964
		Project Total			**$108,999**

To further reduce the amount of 2-inch piping in the system, the existing 2-inch piping on Jack Court and Carriage Place can be replaced with new 6-inch piping. This should reduce leakage concerns associated with 2-inch glued joint pipe.

Piping Improvements - Jack Court					
Item	**Description**	**Unit**	**Quantity**	**Unit Cost**	**Construction Cost**
1	Mobilization, Bonds (10%)	LS	All	$2,200.00	$2,200
4	New 6-Inch	LF	240	$55.00	$13,200
5	Service Reconnects	EA	5	$800.00	$4,000
8	Testing, Disinfection, Misc.	LS	All	$3,000.00	$3,000
		Construction Total			$22,400
		Contingency			$3,360
		Engineering			$4,480
		Administration			$896
		Project Total			**$31,136**

Piping Improvements - Carriage Place					
Item	**Description**	**Unit**	**Quantity**	**Unit Cost**	**Construction Cost**
1	Mobilization, Bonds (10%)	LS	All	$4,000.00	$4,000
4	New 6-Inch	LF	445	$55.00	$24,475
5	Service Reconnects	EA	10	$800.00	$8,000
8	Testing, Disinfection, Misc.	LS	All	$4,000.00	$4,000
		Construction Total			$40,475
		Contingency			$6,071
		Engineering			$8,095
		Administration			$1,619
		Project Total			**$56,260**

Replacing the existing 4-inch piping on Irving Drive is desirable to eliminate the two dead-ends and allow installation of fire hydrants at the south end and north end of Irving Drive rather than installing new fire hydrants on Old Pacific Highway.

Piping Improvements - Irving Drive					
Item	**Description**	**Unit**	**Quantity**	**Unit Cost**	**Construction Cost**
1	Mobilization, Bonds (10%)	LS	All	$12,000.00	$12,000
4	New 8-Inch	LF	1470	$60.00	$88,200
5	Service Reconnects	EA	19	$800.00	$15,200
8	Testing, Disinfection, Misc.	LS	All	$5,000.00	$5,000
		Construction Total			$120,400
		Contingency			$18,060
		Engineering			$24,080
		Administration			$4,816
		Project Total			**$167,356**

Approximately 280 feet of 2-inch piping extends south from Klimback along public right-of-way to serve dwellings. A potential improvement is to replace this piping with 8-inch and extend south to connect to the existing 10-inch on Carte Lane. This will eliminate the dead-end, replace the undersized piping, and better serve future development in this area.

Piping Improvements - Klimback to Carte					
Item	**Description**	**Unit**	**Quantity**	**Unit Cost**	**Construction Cost**
1	Mobilization, Bonds (10%)	LS	All	$5,500.00	$5,500
4	New 8-Inch	LF	740	$60.00	$44,400
5	Service Reconnects	EA	4	$800.00	$3,200
8	Testing, Disinfection, Misc.	LS	All	$2,500.00	$2,500
		Construction Total			$55,600
		Contingency			$8,340
		Engineering			$11,120
		Easement Acquisition			$3,000
		Administration			$2,224
		Project Total			**$80,284**

7.6.5 Old Pacific Highway Improvement Alternatives

Another consideration for the Authority is the potential road-widening by the County of Old Pacific Highway. The County recently completed Phase 1 of the road widening project which required Tri-City to replace over 3600 feet of piping from just north of Chadwick to just north of Woodcrest. As the County widens the road, Tri-City will be required to relocate the water piping to a location farther away from the center of the road and outside the travel path of newly placed asphalt. Due to the large number of customers along the Highway, no alternative route exists and piping must be located along the Highway.

Douglas County has plans to conduct additional future phases of the road widening which will require additional piping relocations by Tri-City. Progress by the County has been slowed due to difficulty obtaining the additional right-of-way; however there is no indication that plans will be abandoned. Tri-City must plan for the fact that this piping may have to be replaced.

By combining the waterline replacement with the overall road work project, the cost for asphalt replacement and some trenching was not required to be paid by Tri-City. As a result, the construction costs for the water piping were less than would be seen for a typical project. The Phase 1 cost was approximately $190,000 with an average cost of $61/foot . The majority of piping installed was 12-, and 10-inch was minor quantities of 8-inch and 6-inch for connections. Bids were received in May of 2004. By increasing the price based on the ENR Construction Cost Index to current dollars, the average cost per foot for construction is estimated at $66.45 per foot.

Phase 2 of the County project is indicated to extend from the Pizza Palace 3500 feet north to near Norton Lane. If the waterline must be relocated for this entire distance the estimated construction cost would be $232,600. Adding contingency, engineering, and administrative costs brings to total estimated project cost to $312,000.

Phase 3 of the County project is planned from near Norton Lane north to the end of the Authority service area and beyond. This project could involve replacement of 2600 feet of water piping at an estimated construction cost of $173,000. Adding contingency, engineering, and administrative costs brings to total estimated project cost to $231,500.

Phase 4 is indicated to occur from just north of Woodcrest (where Phase 1 ended) north 4000 feet to near the Pizza Palace. If the waterline must be relocated for this entire distance the estimated construction cost would be $266,000. Adding contingency, engineering, and administrative costs brings to total estimated project cost to $356,000.

In summary, the remaining 3 phases of the Douglas County projects to widen Old Pacific Highway could result in waterline replacement projects for Tri-City totaling $900,000.

Capital Improvement Plan

Section 8

8.1 Supply and Treatment Improvements

The existing water supply and treatment facilities have a maximum capacity of approximately 1150 gpm. Currently, the plant is able to pump approximately 1000 gpm to the system. To meet projected water demands for a 20-year planning period, a supply and treatment capacity of at least 1740 gpm is needed. Immediate small improvements are being done now at the treatment facility to prevent water shortages in the near future. To meet the projected demand, significant improvements will be required.

8.1.1 Water Supply River Intake

The capacity of the intake structure is limited to 900 to 1200 gpm depending on how low summer water depths go in the South Umpqua River. Additionally, the intake does not contain fish screening provisions as required by the Water Resource Department and the Department of Fish and Wildlife. Requirements to immediately add fish screening are written into several of Tri-City water rights. The lowest cost option discussed in the Water System Master Plan for increasing capacity and adding screening to the existing intake has been selected as a Priority 1 project. In addition, a trailer-mounted diesel-engine emergency pump is planned to safeguard the community in the event of intake failure.

Priority 1. Project Budget = $221,000

8.1.2 Raw Water Pump Station

The existing pump station can convey the projected flow to the plant only when both of the pumps are operated simultaneously. To provide redundancy in this critical component, each of the two pumps should provide the full plant flow to prevent shortages during failure or maintenance of one pump. Since a spare pump is available and immediate capacity increases are not required, this project is considered a Priority 2 improvement. This project will be needed by the year 2010 when the necessity for plant treatment capacity expansion is anticipated.

Priority 2. Project Budget = $152,000

8.1.3 Water Treatment Facility

As discussed in the Master Plan, the plant capacity is limited to 1150 gpm due to the sedimentation basin size. It will be possible to increase plant flows above the current 1000 gpm once the ongoing improvements to add variable frequency drives to the discharge pumps is complete. Current filter area may allow flows up to 1430 gpm, however it is not known if water quality standards will be met when the higher filter rate (and shallow filter depth) is combined with an undersized sedimentation basin. Additionally, hydraulic overloading of the filter discharge piping occurs at a flow somewhere between 1300 and 1600 gpm according to staff. Essentially, plant flows will be increased gradually to determine the maximum flow that can be successfully treated. The flow rate possible will determine how long improvements can be delayed.

For the purposes of planning, it is assumed that the plant will be able to treat 1250 gpm. This will allow the existing facility to meet demands until the year 2010 unless growth outpaces the 2.5% per year projected.

For plant improvements to allow at least 1740 gpm to be treated successfully, membrane filtration has been selected. This option has a present worth cost equal to conventional rapid sand filtration and the greatest potential to meet unforeseen future regulations without the need for additional improvements. Membrane filtration will provide a greater level of treatment than currently possible with the conventional mixed-media sand filters and water quality is expected to increase. Future plant expansion will also be easier to accomplish with membranes.

Plant treatment expansion is considered a Priority 2 project due to cost and the fact that the existing filter capacity should be adequate until the year 2010.

Priority 2. Project Budget = $2,377,000

To obtain adequate disinfection contact time to comply with State and Federal regulations, baffling of the original clearwell is required. In addition, to overcome the safety hazards with the existing chlorine gas system and replace the aging equipment, on-site sodium hypochlorite generation equipment will be installed.

Priority 1. Project Budget = $261,000

8.2 <u>Storage Improvements</u>

Tri-City currently has an overall storage deficiency of 278,000 gallons. Additional storage is required in the high level service area and in the main service area.

8.2.1 High Level Storage Improvements

The high level service area, currently served by the Valley Drive Pump Station and the Back Acres Storage Tank. The lowest cost option is to remove the existing Back Acres Tank and install a new larger tank in this location. This improvement is considered an immediate need due to the extreme lack of storage in the high level system. Constructing a 430,000 gallon tank at the Back Acres site will correct the current overall system storage deficiency and delay the need for the larger main level tank until 2010.

Priority 1. Project Budget = $698,000

8.2.2 Main Level Storage Improvements

An additional 1.1 million gallons of storage is needed in the main system level for the planning period. Based on the adopted growth projections, and if the high level tank is constructed first, the 1.1 MG main level tank will be needed in the year 2010. Several tank site options are viable and the final selection should be based on land availability.

Priority 2. Project Budget = $1,179,000

8.2.3 Existing Storage Tank Improvements

To prevent long-term damage and greater refurbishments costs in the future, the Walnut Street Tank and the Aker Drive Tank should be cleaned and painted as a Priority 1 Improvement. Only the tank exteriors require painting at this time. In addition, minor piping changes should be done at both tanks to promote better mixing of the stored water to prevent chlorine residuals from dropping below required levels. Better mixing and turnover in the tanks should allow water levels to be maintained at higher settings preventing the large storage deficiency with current low level settings. Potentially, the switch to sodium hypochlorite generated at the treatment plant will also promote better chlorine residuals in the tanks.

Priority 1. Project Budget = $112,000

Note that if the Back Acres Tank is not removed to construct a larger tank, this tank will also require repainting, mixing improvements, and fencing during the planning period.

8.3 Distribution System Improvements

The various recommended piping improvements in the system are separated into 3 priority phases. Priority 1 improvements are needed to correct immediate fire flow deficiencies for larger areas. Priority 2 projects improve system looping and reliability and correct remaining fire flow deficiencies for smaller areas. Priority 3 projects replace undersized piping and potential system leakage pipes, create additional looping, and facilitate land development within the urban area.

8.3.1 Priority 1 Piping Improvements

Priority 1 Piping Improvements consist of the lowest cost options for correcting serious fire flow deficiencies in the southern portion of the service area around Arnold Lane and Matthews Lane, and the minimum improvements necessary to correct serious flow deficiencies in the area just south of Walnut Street around Mona Street, Rollin Court, Margie Court, and Cook Street. Upsizing and the addition of a fire hydrant on Ridgewood Place are also included since no fire protection exists currently.

Priority 1. Project Budget = $567,000

8.3.2 Priority 2 Piping Improvements

Priority 2 Piping Improvements consist of pipe looping from Matthews to Briggs Drive, upsizing the piping on north Arburnia Street and adding a fire hydrant (no fire protection at north end of Arburnia now), and completing the 12-inch looping and correcting the remaining fire flow problems around Luke Court.

Priority 2. Project Budget = $530,000

8.3.3 Priority 3 Piping Improvements

Priority 3 projects include various looping and undersized pipe replacement. Also included is replacement of piping along Old Pacific Highway that will be required when the Douglas County road widening projects continue. These projects can be delayed until needed due to development, until the County's road projects commence, or until various lines become problematic with unacceptable leakage or repair requirements. Generally, these projects can be considered optional at this point with the exception of the piping along Old Pacific Highway.

Priority 3. Project Budget = $1,588,000

8.4 Priority 1 Improvement Summary

Priority 1 Improvements - Immediate Need				
Item	**Description**	**Project Cost**	**% Capacity for Growth**	**Eligible SDC Cost**
1	Intake Screening Modifications	$185,735	43%	$79,601
2	Portable Emergency Supply Pump	$35,000	43%	$15,000
3	Plant Disinfection System Improvements	$260,625	43%	$111,696
4	Plant Backwash Pond Improvements	$65,000	43%	$27,857
5	High Level Storage, 0.43 MG Tank	$697,433	79%	$551,458
6	Walnut St. Tank Refurbishment	$44,758	0%	$0
7	Aker Dr. Tank Refurbishment	$43,420	0%	$0
8	Tank Mixing Improvements (Aker and Walnut)	$23,630	0%	$0
9	South Area Piping Improvements, Option B	$211,681	50%	$105,841
10	Walnut St. Area Priority 1 Piping	$248,945	50%	$124,473
11	Ridgwood Place Piping	$105,814	0%	$0
Total		**$1,922,040**		**$1,015,925**

Priority 1 Improvements are needed immediately to comply with State regulations, correct existing storage and fire protection deficiencies, and to provide basic maintenance of two storage tanks. These projects should be initiated in 2006.

8.5 Priority 2 Improvement Summary

Priority 2A Improvements - Needed in 2010				
Item	**Description**	**Project Cost**	**% Capacity for Growth**	**Eligible SDC Cost**
1	Raw Water Pump Replacement	$151,336	43%	$65,075
2	Membrane Filtration Plant Upgrade	$2,376,900	43%	$1,022,067
3	Main Level Storage, 1.1 MG Tank	$1,178,456	100%	$1,178,456
Total		**$3,706,692**		**$2,265,597**

Priority 2B Piping Improvements - Desired by 2010				
Item	**Description**	**Project Cost**	**% Capacity for Growth**	**Eligible SDC Cost**
1	South Area West Loop	$208,071	25%	$52,018
2	Walnut St. Area Priority 2 Piping	$321,503	75%	$241,127
Total		**$529,574**		**$293,145**

Priority 2A Improvements will be needed by 2010 based on projected water demand increases. These projects will bring plant capacity up to meet demand beyond the year 2010 and add the additional storage needed by that time. In order to ensure that these projects are sized for at least a 20-year design life, sizing should be updated when the projects are initiated. It is possible that the capacity will need to be slightly larger than is determined now and would be based on the year 2030 demands if initiated in 2010. It is likely that costs will also be higher than estimated now. Construction costs have risen an average of 4.8% per year since 2004.

Once the Priority 1 plant improvements are complete (specifically the disinfection improvements), flows at the plant can be increased. If treatment problems become apparent at flows less than 1250 gpm the Priority 2 plant improvements should be planned for immediately. If a flow of 1250 gpm is possible then the plant improvements can be delayed until 2010. If flows can be pushed to 1400 gpm, the Priority 2 plant improvements can be delayed until the year 2015 if growth matches the 2.5% projected.

After the Priority 1 high-level tank is completed, the high-level system storage should be adequate for build-out. However, the main service level will have a storage deficiency by 2010 and the Priority 2 1.1 MG tank will be needed.

Priority 2B Improvements are recommended in 2010 as well however they are less critical than the 2A improvements and could be delayed if funding cannot be obtained.

8.6 Priority 3 Improvement Summary

Priority 3 Piping Improvements - Desired by 2020				
Item	Description	Project Cost	% Capacity for Growth	Eligible SDC Cost
1	Back Acres Piping	$114,868	50%	$57,434
2	Peacock Lane Piping	$54,141	0%	$0
3	Taylor to Corwin Loop	$74,018	25%	$18,504
4	Taylor to Susan Loop	$108,999	75%	$81,749
5	Jack Court	$31,136	0%	$0
6	Carriage Place	$56,260	0%	$0
7	Irving Drive	$167,356	50%	$83,678
8	Klimback to Carte	$80,284	75%	$60,213
9	Old Pacific Hwy. Piping due to Road Widening	$900,000	50%	$450,000
10				
11				
Total		**$1,587,061**		**$751,579**

Priority 3 Improvements are recommended by 2020 but most could be delayed beyond that date if funds are not available and problems do not develop with the existing sections of piping. Some of these project may also be driven by development and therefore paid for by land developers rather than with public funds. The largest portion of the Priority 3 costs is the potential water piping relocations required when Douglas County proceeds with the road widening projects. The exact timeframe and extent of work required is not known at this point but the expense will not be optional.

Financing Alternatives

9.1 Improvement Costs

The total estimated cost for all improvements recommended in this Water System Master Plan is $7,746,000. Improvements to supply and treatment are planned to increase capacity from 1000 gpm currently up to at least 1740 gpm resulting in 43% of the planned capacity reserved for future users and system growth. Various other improvements range from 0% for growth (tank repainting and other maintenance items) up to 100% for growth for new main level storage. Overall, the total cost for projects and portions of projects with excess capacity to serve growth is $4,326,000. The remaining $3,419,000 of the total cost cannot be considered necessary to provide for growth but either as maintenance costs or cost to replace existing capacity.

System EDUs are projected to increase from 1731 to 2836 over the next 20 years with an average of 2170 EDUs for the planning period. Growth EDUs are therefore estimated at 1105.

Priority 1 Improvement Cost:	$1,922,000
Priority 2a Improvement Cost:	$3,707,000 (plant upgrade, raw water pumps, 1.1 MG tank)
Priority 2b Improvement Cost:	$530,000 (piping)
Priority 3 Improvement Cost:	$1,588,000

9.2 Water Rate Impacts

In Tri-City the typical household uses an average of 7075 gallons of water per month. A recent $5 rate increase which becomes effective this summer will result in an average residential water bill of $39.61 per month ($39.61 per month per EDU for 7075 gallons). Funding agencies often cite a use of 7500 gallons per month to estimate the average water bill. For 7500 gallons, the existing rate is $40.25. The rate structure consists of a base charge of $35 per month plus $1.50 per 1000 gallons used after the first 4000 gallons.

Total water system expenses including general operating expenses, maintenance, and outstanding debt payments was $746,175 for fiscal year 2005. By adding the new loan repayment of $26,000 per year for the Phase 1 Douglas County road widening project waterline relocations, the new annual total expense is expected to be approximately $772,175 not including any major capital expenditures. Yearly tax revenues are $114,888 leaving $657,290 required through water user rates. Since $683,534 in revenue was generated in FY 2005 with the previous water rate in effect, the entire $5 per month recent increase should be available for capital improvements.

Based on 2170 EDUs and a loan repayment over 20 years at 5% interest, a total of $1.62 million can be funded with the recent increase of $5 per EDU. With the total non-SDC eligible cost of $3,419,000 funded with a single 20-year loan at 5% interest, the monthly cost per EDU is $11.60 per month including a 10% reserve. This would require a second rate increase of $6.60 per month per EDU based on the current rate structure.

If the Priority 1 Improvements are initiated immediately with a 20-year loan at 5% interest, the cost per EDU is approximately $7.50 per month based on the current 1731 EDUs. If 20% grant could be obtained, the cost per EDU is $5.95 per month ($6.20 with loan at 4.375% and 10% reserve).

Priority 2A Improvements with the same loan terms result in a cost per EDU of $12.80 per month based on the year 2010 projected EDUs of 1958. Priority 2B adds another $2.00 per month per EDU. If 20% grant can be obtained, the cost per EDU is $10.13 per month for Priority 2a.

Priority 3 Improvements with the same loan result in a cost per EDU of $4.80 per month based on the year 2015 projected EDUs of 2216.

9.2.1 Required Rates Under Current Structure

To initiate Priority 1 Improvements immediately, a loan for the total Priority 1 amount would be obtained. A 20-year loan at 4.375% interest is assumed. The annual loan payment required would be $146,163 which equates to $12,180 per month. To ensure that sufficient revenue is brought in each month, an amount 10% greater than the minimum required monthly amount should be planned for to provide a 10% reserve cushion. To generate $13,400 per month with 1731 EDUs, the cost per EDU is $7.75 per month. Since the base rate was recently raised $5, a second increase of $2.75 would be needed (assuming the entire $5 increase is available for improvements). To ensure sufficient funds without relying on water quantities sold, the base rate would increase from $35 to $37.75. The average residential water bill would then be approximately $42.35. If 20% grant funding is obtained for Priority 1, the base rate could be increased from $35 to $36.20.

In 2010, Priority 2A is required. If we assume that $250,000 in SDCs are generated by 2010, the required loan amount is reduced by $250,000. The annual loan payment required for the remaining Priority 2A cost would be $262,867. Fortunately, the current GMAC bonded debt will be retired in 2010 freeing up an additional $89,168 per year. A rate increase is then needed to generate $173,700 per year. Using the projected 2010 EDUs of 1958, the resulting cost per EDU is $8.13 per month including a 10% reserve. The average bill would then need to be $48.93 per EDU. With Priority 2B added, the average bill per EDU increases to $52.60 per month. If 20% grant funding was possible, the cost per EDU for Priority 2A would be $5.50 and the average bill would be $46.30.

System Development Charges would be collected over time and used to reimburse and fund the Priority 3 Improvements.

9.2.2 Potential Water Rate Structure Changes

In FY 2005, there were 1731 EDUs in the system based on actual water consumption records. The average residential bill was $34.61 (now $39.61). Total sales revenue generated was $600,342 resulting in an average income of $28.90 per month per EDU. As discussed in Section 6.6.1, an ideal rate structure where each user is charged an equal amount for each unit of water consumed would result in revenue of $718,920 ($34.61 x 1731 x 12). The current rate structure requires an estimation of the number of EDUs for each non-single-family-residential user. What has happened is that many large water users have been allocated less EDUs than they actually are. As a result, smaller users are subsidizing the large users and these large users are not paying their fair share for the cost of water.

Based on 2005 records, the actual cost to produce and distribute water in Tri-City was $4.10 per 1000 gallons. If the new loan payment to Douglas County for Phase 1 road widening is added, the cost is $4.20 per 1000 gallons. With all of the non-SDC eligible costs for Priority 1 and 2 added, plus an increase in operating expense at the plant associated with a 75% increase in production, the cost is estimated to be approximately $5.55 per 1000 gallons. For the average single family dwelling in Tri-City, the water bill to just cover the actual cost of water would be $39.30 per month. To provide a revenue/expense ratio of at least 1.2, the charge per 1000 gallons of water should average $6.50. The resulting average bill for a single family dwelling would be $46 per month.

At this time detailed billing records showing existing meter sizes and long-term use per account are either not available or difficult to generate with the current accounting software setup. Without this information it is difficult to predict how a change in the rate structure will affect monthly or annual revenue generated by water sales. It is however recommended that Tri-City switch to an increasing block structure water rate based on consumption and meter sizes. This will eliminate the tedious and sometimes difficult task of estimating EDUs for each user. A new rate structure can also eliminate the unfair burden now placed on smaller water users.

A potential rate structure is shown below with a base rate per meter size plus an additional cost for consumption. The cost for water use up to 15,000 gallons per month increases in blocks which will tend to encourage water conservation for smaller users. The rate then drops to the cost of production for larger use (over 15,000 gallons per month) to lessen the impact to very large water users who tend to create and sustain local jobs.

Rate Structure with $46 Average Residential Bill (7075 gallons)

Meter Size	Base Rate	$ per 1000 gal. < 3000 gallons	$ per 1000 gal. 3000 to 7000 gallons	$ per 1000 gal. 7001 to 15000 gallons	$ per 1000 gal. > 15000 gallons
5/8-inch meter	$20.00	$2.50	$4.50	$6.50	$5.60
3/4-inch meter	$20.00	$2.50	$4.50	$6.50	$5.60
1-inch meter	$50.00	$2.50	$4.50	$6.50	$5.60
1-1/2-inch meter	$65.00	$2.50	$4.50	$6.50	$5.60
2-inch meter	$120.00	$2.50	$4.50	$6.50	$5.60
3-inch meter	$200.00	$2.50	$4.50	$6.50	$5.60
4-inch meter	$300.00	$2.50	$4.50	$6.50	$5.60
6-inch meter	$600.00	$2.50	$4.50	$6.50	$5.60
8-inch meter	$900.00	$2.50	$4.50	$6.50	$5.60

For the current average single-family dwelling using 7075 gallons of water per month, the monthly water bill would be $46 per month. For a household on a fixed income interested in lowering their water bill, water use could be limited to 4000 gallons per month resulting in a bill of $32 per month which is lower than the current rate.

For a larger water user with a 2-inch meter using 530,000 gallons per month (similar to the Tri City Mobile Estates), the monthly water bill would be $3082. Such a user is actually equivalent to 75 EDUs but is only paying the equivalent of 67 EDUs. With this much water consumption, a 4-inch meter should be required. With a 4-inch meter, the bill would be $3261 per month equivalent to 71 times the average single family household to better match the 75 times greater water consumption.

A user with a 2-inch meter and 88,000 gallons per month consumption, the monthly water bill would be $606.30. Based on water consumption, this user is equal to 12.4 EDUs. Based on the rate paid, the user is paying 13 times the average residential rate.

The potential rate structure shown will result in each user paying a rate much closer to their actual consumption EDUs times the rate for a typical EDU. In this way, each customer pays a fair share of the cost to operate and maintain the water system rather than placing a large burden of the cost on smaller users. The monthly bill for small families or retired couples on fixed incomes should actually be lower than current rates; however overall income to the Authority should increase.

With 1731 EDUs and each user paying an equal share for water ($46 for 7075 gallons), water sales revenue should be approximately $955,000 per year. This is $355,000 more than was generated in FY 2005. As the number of EDUs increases, revenue will increase as well. This potentially is enough to fund Priority 1 and 2a Improvements however inflationary adjustments may be required if Priority 2a is delayed.

Since accurate records of individual meter sizes and consumption are not available, the precise impact to revenue cannot be determined. The Authority can either collect the necessary data and conduct a detailed rate study, or can initiate the rate structure change and closely monitor the results and make periodic adjustments as necessary.

If 20% grant funding can be obtained for Priority 1 and 2a Improvements, the cost to produce water and run the system is estimated at $4.75 per 1000 gallons. To cover general operating costs and to pay back the various loans, including a new loan for 80% of the cost for Priority 1 and 2a, an average bill of $43 per EDU is estimated to be required. For a household on a fixed income interested in lowering their water bill, water use could be limited to 4000 gallons per month resulting in a bill of $32.80 per month which is lower than the current rate.

Rate Structure with $43 Average Residential Bill (7075 gallons)

Meter Size	Base Rate	$ per 1000 gal. < 3000 gallons	$ per 1000 gal. 3000 to 7000 gallons	$ per 1000 gal. 7001 to 15000 gallons	$ per 1000 gal. > 15000 gallons
5/8-inch meter	$22.00	$2.50	$3.30	$5.00	$4.75
3/4-inch meter	$22.00	$2.50	$3.30	$5.00	$4.75
1-inch meter	$55.00	$2.50	$3.30	$5.00	$4.75
1-1/2-inch meter	$71.50	$2.50	$3.30	$5.00	$4.75
2-inch meter	$132.00	$2.50	$3.30	$5.00	$4.75
3-inch meter	$220.00	$2.50	$3.30	$5.00	$4.75
4-inch meter	$330.00	$2.50	$3.30	$5.00	$4.75
6-inch meter	$660.00	$2.50	$3.30	$5.00	$4.75
8-inch meter	$990.00	$2.50	$3.30	$5.00	$4.75

9.3 System Development Charges (SDC)

Out of the total project costs $4,326,000 is that portion providing capacity for future users and is eligible for inclusion in SDC determinations. Currently the system contains 1731 equivalent dwelling units (EDU). System EDUs are projected to increase by 1105 over the 20-year planning period for a total of 2836 EDU. By dividing the eligible costs by the number of added EDUs, a cost of $3,915 per EDU results. If the projected estimate of 1105 new EDUs added to the system by the year 2025 is realized, a total of $4,326,000 will be collected to construct capacity building projects or to reimburse for capacity building projects already completed.

In addition, as discussed in Section 6.6.2, the cost for the Valley Drive pump station ($45 per future EDU) and the Water System Master Plan ($45 per future EDU) can be added to the SDC computations as well.

The total maximum water SDC that can be justified for these improvements is therefore $4005 per EDU. If rates are effectively increased to pay for improvements, credits should be awarded to future users so that they do not have to pay SDC fees and increased user rates. To determine the credit, the present worth value of any rate increase needs to be determined based on the point in time that the new user joins the water system. For example, if rates are increased by $6 to construct improvements and a new user joins the system in year 1 out of 20, the present worth value of $6 per month over 20 years is $1071 assuming 3% inflation. The effective SDC such a user would pay is then $2934 ($4005 - $1071). If 20% grant funding is obtained and Priority 1 and 2a improvements are initiated with a $3 per month per EDU rate increase, the present worth value for 20 years of the rate increase is $536. Then a user joining the system in year 1 would pay an effective SDC of $3469. If the user joined the water system in year 10 out of the 20-year improvements they would only be paying 10 years of the rate increase and the effective SDC would be $3698.

To facilitate future adjustments in SDC methodologies and to have a single methodology for all utility services provided by the Authority (water and sanitary sewer), and since exact SDC charges and credits will depend on projects selected, rate structures, the level of grant funding obtained, and the terms of any loans acquired, a separate stand-alone SDC Methodology Study is recommended.

9.4 Loan and Grant Sources

The current State average non-metropolitan median household income is $41,230. According to the 2000 Census Tri-City has a median household income (MHI) of $33,306 which is 80.8% of the state average. The percent of low/moderate income persons in Tri-City is unknown without a special income survey. Since Myrtle Creek currently has less than 50% low/mod persons, it is likely that Tri-City has less than 50% as well. The current average residential water bill is $39.61 for the average use per EDU of 7075 gallons, and $40.25 for use of 7500 gallons.

Based on the information at this time, Tri-City does not qualify for a Block Grant from the Oregon Economic and Community Development Department (OECDD) since the low/mod income requirements are not met. Loans are available from State and Federal sources. The only major grant potential appears to be through the USDA Rural Utilities Service (RUS). Grant funding for a portion of the water intake (60% of cost up to $75,000) is possible through the ODFW Cost Share Grant Program.

The RUS Water and Waste Disposal Program provides funding to communities of 10,000 or less when the community is unable to obtain needed funds from commercial sources at reasonable rates and terms. Grants may also be provided when necessary to reduce user costs to reasonable levels. Clem Singer with RUS was contacted to discuss Tri-Cities needs. According to Mr. Singer, "reasonable user cost" is currently around $38 per month per EDU. Since Tri-City recently increased rates such that the rate per EDU is now slightly over $39 per month, grant funding is possible. Tri-City qualifies for the intermediate loan term interest rate of 4.375%. Mr. Singer indicated that there is a good possibility for Tri-City to receive through RUS a 20% grant. The remaining 80% of project costs could be funded with a loan at 4.375%.

Potentially, a loan could also be obtained through the Safe Drinking Water State Revolving Loan Fund. Typically an existing compliance problem or health risk is required however with capacity problems and a potential compliance problem with turbidity and/or contact time violations if flows are increased without improvements, funding might occur. This loan program could be considered if the terms are better than the RUS loan.

At this point, the funding assistance available from the USDA Rural Utilities Service (RUS) appears to be the most attractive option. A one-stop meeting should be conducted as soon as possible to ensure that all options are considered and allow Tri-City to choose between the best State and/or Federal sources.

9.5 Recommended Financing Plan

Step 1 – Conduct preliminary engineering to further develop intake screen details and drawings. Apply for ODFW Cost Share Grant Program and COE/DSL Permit for intake screening project. Summer 2006.

Step 2 – Schedule a One-Stop Meeting with funding agencies to select best loan options and maximum grant possible. Summer 2006.

Step 3 – Based on funding, select projects to be initiated. If 20% grant can be obtained it is recommended that Priority 1 and Priority 2a projects be undertaken simultaneously. This will ensure that the plant improvements are completed when needed in 2010 and will avoid the inflationary increases in the plant project that will occur if delayed another 4 or 5 years. Priority 1 + 2A with a 20% grant and a 20 year loan at 4.375% will required an annuity payment of $253,000 and average bill of $43 per EDU. If Priority 1 alone is selected with same funding, the annuity will be $116,930.

Step 4 – With final project funding and schedule known, adopt new rate structure as soon as possible. Enact new rates by January 2007. Average bill of $46 is required to fund all non-SDC eligible improvements for the planning period. At minimum a structure that provides an average bill of $43 per EDU is recommended.

Step 5 – Conduct SDC Methodology Study in Fall 2006. Increase SDC as applicable. Budget approximately $15,000 for Water and Sewer SDC Study.

Step 6 – Evaluate impact to sales revenue due to rate structure change after 3 months of new rate. Ensure sufficient reserve funds are being generated to pay for any loans obtained. Adjust rates in mid 2007 if required.

Step 7 – Begin design of selected projects that have been funded. Begin as soon as possible after funding approved.

Step 8 – Use collected SDC funds to initiate Priority 2b improvements. Goal is to complete project by the year 2010. Initiate as soon as funds are available.

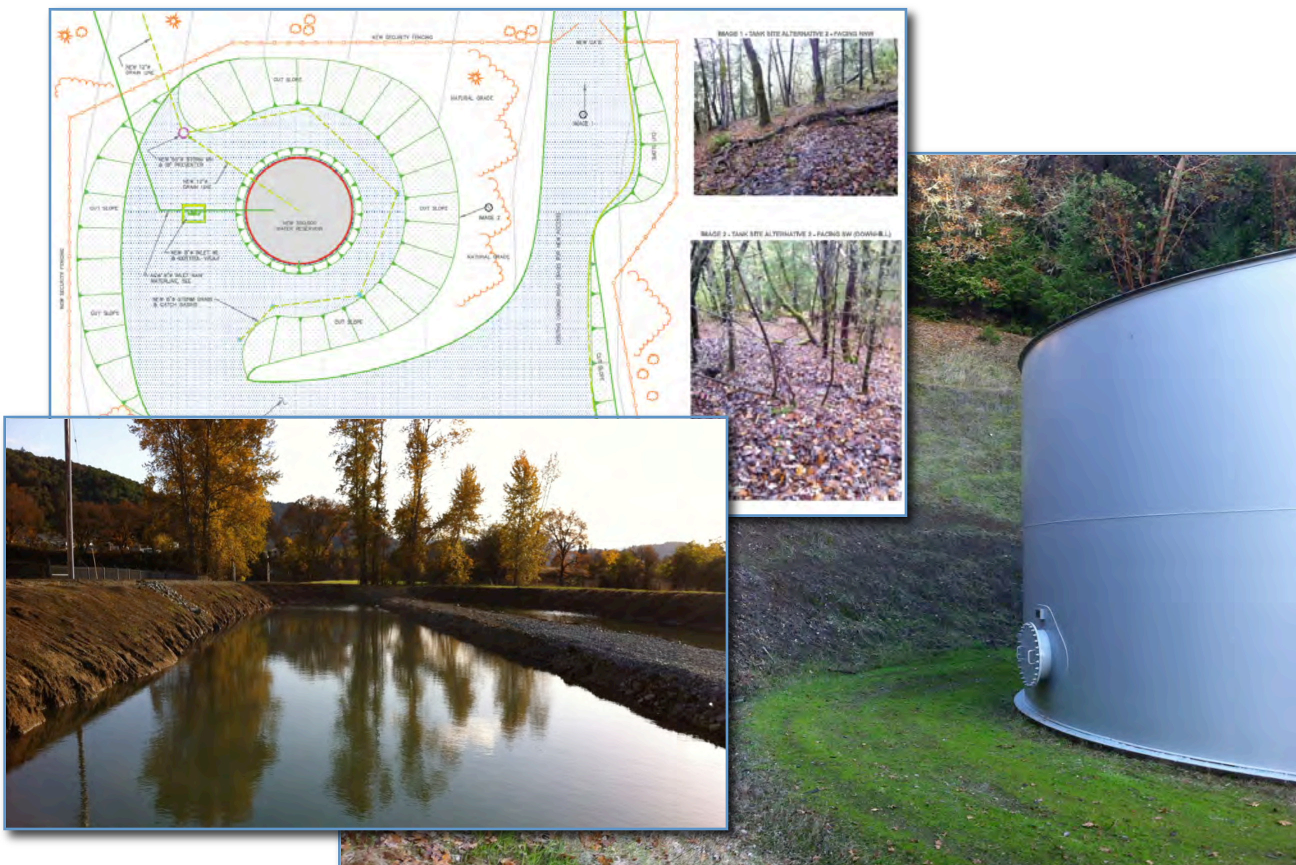
Step 9 – Conduct Priority 3 projects as funds allow with collected SDC funds. If Douglas County begins the subsequent phases of their road widening project, this work may have to be initiated before sufficient SDC funds are available. In this case, a small rate increase may be required.

Step 10 – Review water user rates annually. At minimum, make inflationary adjustments to avoid the need for large rate increases in the future.

Tri City Water
& Sanitary Authority

Back Acres Preliminary Engineering Report

JUNE 2013



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Project No. 100.01

Tri City Water & Sanitary Authority
Douglas County, Oregon

Back Acres Preliminary Engineering Report

June 2013

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ABBREVIATIONS

AAD = Average Annual Demand
ADD = Average Daily Demand
MMD = Maximum Monthly Demand
MDD = Maximum Daily Demand
PHD = Peak Hourly Demand

WMP = Water Master Plan (HBH, 2006)
WRS = Water Rate Study (2007)

Executive Summary

Tri City Water & Sanitary Authority (Tri City, Authority) has diligently pursued the improvement of its water system in order to ensure excellent service to its customers. This study affirms the Authority's commitment to the community and answers key questions concerning various aspects of the water system. These aspects include the current status of water system characteristics, updated projections for future water system needs, conservation measures, required capital projects, and other important needs.

The need for this study became apparent over time, as the Authority has worked to prioritize improvements, address perceived system deficiencies, and accommodate potential growth. An important potential driver of water system demand could include development of the industrial park on the west side of Interstate 5. Development groups have asked the Authority to provide estimates of the level of utility service (water and wastewater) that can currently be provided, as well as those improvements required to accommodate a significant utility consumer. The answers to these questions requires a detailed analysis of current water system characteristics, and a comprehensive understanding of the current status of the water system. This analysis also forms the basis for answering other questions concerning the water system, including a new high-level pressure zone inline water storage tank (Back Acres), and conservation measures that could practically be implemented in the water system. This analysis provides a practical milestone-based framework that will guide the efforts of the Authority in the coming years, including the level of service that could be provided to the industrial park should a developer become seriously engaged. The required demand characteristics will trigger important improvements. Although each specific type of development has its own marginal demand characteristics that must be considered during the development phase, the current study will provide the necessary tools to support the development discussion.

This study was funded in part (50% matching) through a grant awarded by the Oregon Water Resources Department (Grant Agreement No. 0056 13). In addition to the objectives above, a requirement of the grant funding includes statutory tasks that must be considered in the study. This discussion is developed in Appendix A.

Study Objectives

The primary objective of the this study is to provide a preliminary engineering analysis for an inline water storage tank within the high-level pressure zone of Tri City's water system. Additional objectives of the study are summarized in the bulleted list below including the sections of relevant work.

- Compile necessary data, including the validation of the Water Master Plan data (Section 4.1). Ensure that water requirements are adequate to meet water system needs (Section 4.2).
- Identify potential alternatives to improve current deficiencies in the water system (Section 7).
- Evaluate each alternative for the Back Acres inline water storage tank, including preliminary engineering cost estimates for each (Section 7).
- Provide recommendations for the most feasible and cost effective alternative to meet water system needs (Section 7).
- Provide recommendations for water conservation and efficiency measures (Appendix A).

In addition to the objectives above, grant funding requirements call for specific statutory tasks to be considered in the study. These tasks are regarded in more detail in Appendix A.

ES1 Historic Water System Characteristics

The foundation of this study was created through a comprehensive analysis of the current status of the water system. This was accomplished through the review of a significant body of previous work, and analysis of recent water system data. The last three years of water system records were analyzed in detail and compared to the Water Master Plan (WMP) (2006, HBH), water rate study (2007), and other previous work. The primary goal of this analysis is to understand current water system characteristics as a means to ensure that excellent service can continue, and that future needs can be met. Reflection of this analysis upon the overall future needs of the community provides perspective on improvement needs as the community grows. Planning for future capital projects, conservation, and system efficiency projects are provided.

Figure ES1 provides a graphical representation of the Average Daily Demand (ADD) of water for the analysis period of 2001 – 2012. The blue bars represent actual historic water system data, while the red bars represent projections developed in the WMP. It is important to note that the blue bars for recent water system demand (2010 – 2012) are much less than the projected values developed in the WMP. The key value from this analysis shows that timeframes for expensive infrastructure improvements called for in the WMP can be deferred well into the future. Complex relationships drive water system demand, though the differences between projected and actual demand are the direct result of actions taken by the Authority. These direct actions include the reduction of water system losses through distribution system repairs, and the implementation of a new water rate structure in 2007. The new water rate structure encouraged conservation. In addition, the current population is significantly less than the projection developed in the WMP. The WMP projected a 2010 population of 4,592, while the 2010 census proved an actual population of 3,931.

Figure ES1 – Historical Average Daily Demand (2001 – 2012)

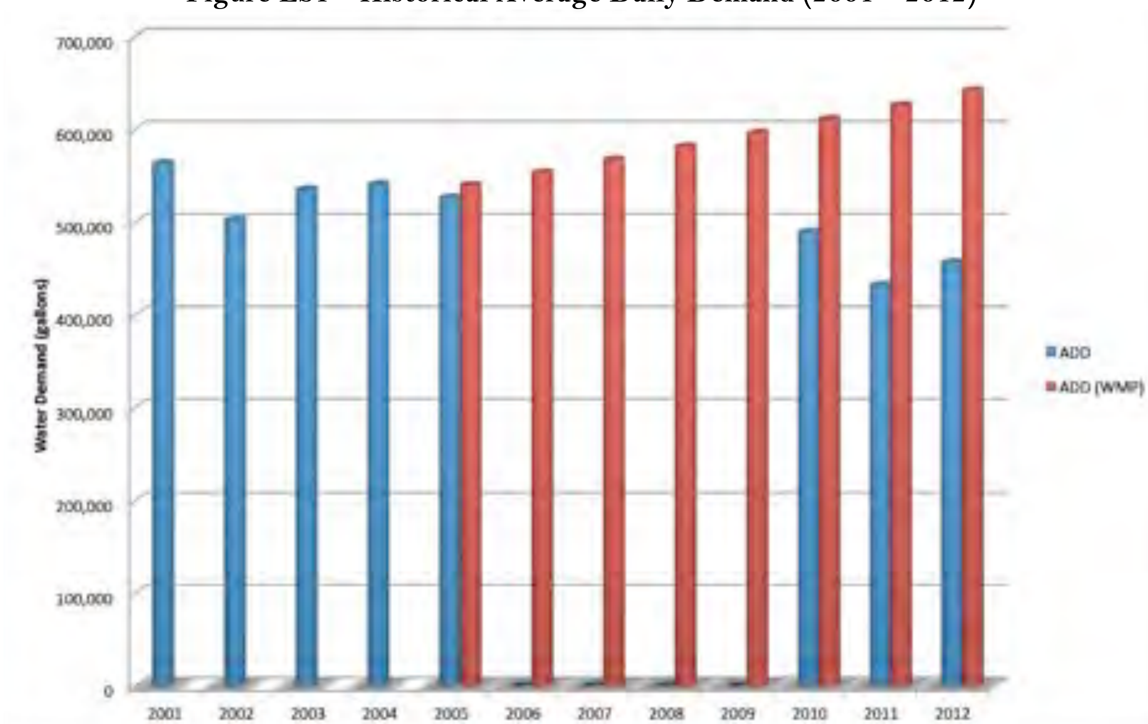


Table ES1 provides a summary of actual historic water system data, as well as projections developed in the WMP from 2001 - 2012. Abbreviation definitions are provided on page iii of the Table of contents.

Table ES1 – Historic Water Demand (2001 – 2012)
(Units in gallons per day unless otherwise noted)

	AAD (gallons)	ADD* (WMP)	ADD	MMD	MDD	PHD	Notes
2001	205,583,800		563,243	901,189	1,464,433	2,816,216	WMP
2002	183,322,900		502,255	803,607	1,305,862	2,511,273	WMP
2003	195,308,600		535,092	856,147	1,391,239	2,675,460	WMP
2004	197,397,000		540,814	865,302	1,406,116	2,704,068	WMP
2005	192,009,737	540,000	526,054	841,687	1,367,741	2,630,270	WRS
2006	202,027,500	553,500	-	-	-	-	WMP
2007	207,078,370	567,338	-	-	-	-	WMP
2008	212,255,165	581,521	-	-	-	-	WMP
2009	217,561,535	596,059	-	-	-	-	WMP
2010	178,499,353	610,960	489,039	782,463	1,271,502	2,445,197	This Study
2011	157,746,490	626,234	432,182	691,491	1,123,674	2,160,911	This Study
2012	166,653,345	641,890	456,585	730,535	1,187,120	2,282,923	This Study

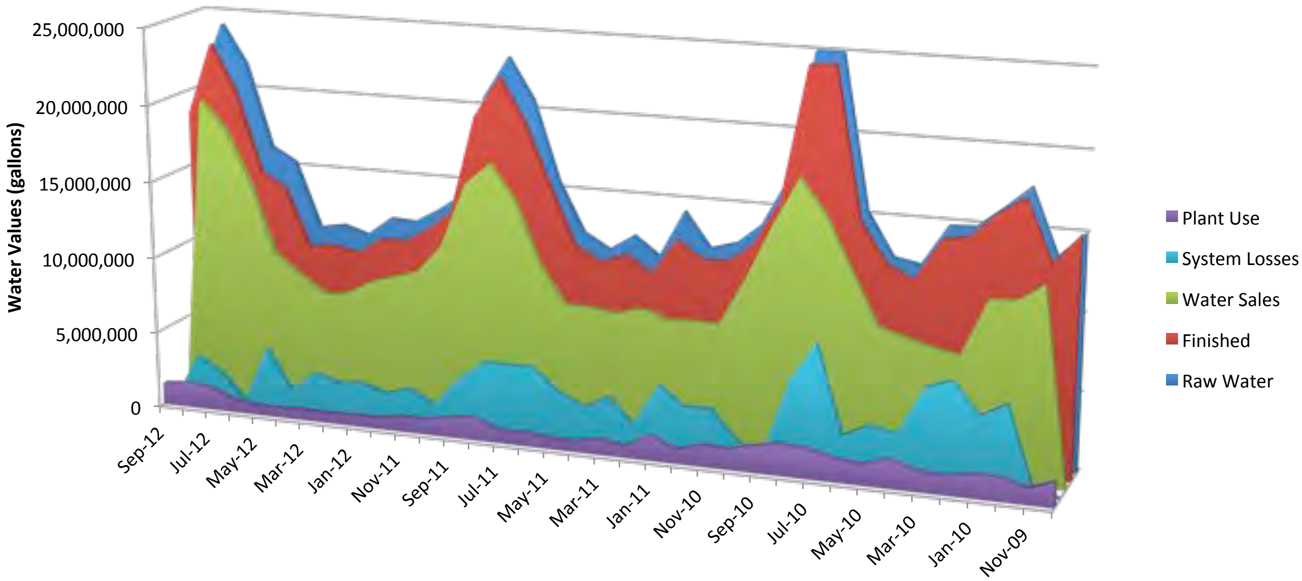
* This column reflects the future projection developed in the WMP.

Table ES1 provides a comprehensive picture of the previous decade of water system demand. Key aspects to note are table cells highlighted in orange and green. The orange cells show that current ADD (2012) is in excess of 80,000 gallons per day less than the ADD developed in the WMP (2005). The green cells show that the current Average Annual Demand (AAD) is nearly 29 million gallons less than that developed in the WMP. Conservation-encouraging policies, operational efficiency improvements, system leak mitigation, and lower than projected population have played important roles in water system demand reduction. This has provided valuable time for the Authority to continue cost-effective infrastructure development projects and future planning efforts.

Figure ES2 provides a comprehensive illustration of water system characteristics for the period of 2010-2012. The figure represents all major aspects of the water system from the raw water intake, to the end consumer. Some key qualitative observations from Figure ES2 include:

- Please note that the horizontal axis historically progresses from right to left.
- Monthly water system demand growth has remained flat, or slightly decreased, over the period. This validates the findings of the study, especially related to the milestone-based planning guidelines, as well as the current water system demand projections.
- Water system losses have decreased historically, proving that actions to repair leaks in the water system have resulted in significant reductions in overall losses. This may also have contributed to the slight reduction seen in overall demand over the period.
- Water plant usage has remained consistent and efficient over the period.
- Water conservation encouraging water rate structure has continued to prove effective.
- Water sales increased in 2012, yet overall system demand remained flat. The reasons for this are not entirely understood, however, Tri City efficiently delivered this water to users.

Figure ES2 – Historic Monthly Water System Characteristics (2010 – 2012)



ES2 Projected Water System Characteristics

This study has developed a comprehensive evaluation of the previous decade of water system characteristics and produced a milestone-based demand tool that will guide future water system needs. The population growth rate developed in the WMP has been utilized in this study, yet the updated future population projection developed in this study shows the 2010 US Census value as the new baseline. Table ES2 provides this milestone-based tool, including comparative values to those developed in the WMP, and the values developed in this study. Maximum Monthly Demand (MMD) and Maximum Daily Demand (MDD) are important water system characteristics that directly affect the design of water infrastructure improvements. The cells highlighted in blue illustrate an important key finding. The 2013 MDD projected in the WMP will not actually occur until sometime after 2025, unless significant development occurs. This table can be utilized as an infrastructure-planning tool. Demand characteristics of new development can be added to the current status of water system demand at any time, in order to project which improvements could be required. Figure ES2 clearly shows that infrastructure improvements called for in the WMP may be deferred long into the future, as long as existing water infrastructure is well maintained and efficiently operated.

Table ES2 – Comparison of Projected Demand*

	Water Master Plan Projection		Study Projection	
Year	MMD	MDD	MMD	MDD
2013	1,053,919	1,711,856	748,799	1,216,798
2014	1,080,266	1,754,652	767,519	1,247,218
2015	1,107,273	1,798,518	786,707	1,278,398
2016	1,134,954	1,843,481	806,374	1,310,358
2017	1,163,328	1,889,568	826,534	1,343,117
2018	1,192,412	1,936,807	847,197	1,376,695
2019	1,222,222	1,985,228	868,377	1,411,112
2020	1,252,777	2,034,858	890,086	1,446,390

2021	1,284,097	2,085,730	912,338	1,482,550
2022	1,316,199	2,137,873	935,147	1,519,614
2023	1,349,104	2,191,320	958,526	1,557,604
2024	1,382,832	2,246,103	982,489	1,596,544
2025	1,417,403	2,302,255	1,007,051	1,636,458
2026	-	-	1,032,227	1,677,369
2027	-	-	1,058,033	1,719,303

* Values are compared between the projections in the Water Master Plan (HBH, 2006) and this preliminary engineering report.

ES3 Current Status of the Water System

Tri City's water system was well characterized and presented in the WMP. The focus of the present study is to verify the current status of this water system, and re-characterize its various aspects. This will help Tri City to provide reliable and cost-effective water service for the community well into the future.

The bulleted list below is a chronological summary of recent efforts relating to the water system.

- The Authority developed a **Water Master Plan** (HBH, 2006), which provided infrastructure assessment and planning for the coming twenty years (2006-2026).
- In 2007, the Authority commissioned a **water rate study** and implemented its recommended water rate structure. The goal of the new structure was to encourage maximum conservation within the system through a purely consumptive model. Water demand fell significantly, which is supported by the analysis provided in this study.
- Authority installed an **on-demand hypochlorite disinfectant generation system** in 2008, which reduces the potential of disinfection byproduct risk, enhances disinfection effectiveness and efficiency, and reduces chlorine gas system health and safety risk. This also reduces cost and supply chain risks associated with chlorine gas systems.
- Tri City procured the **Pruner Road Hotel Impact Study** (Civil West, 2008) to study the resources and improvements that would be required to service a hotel in the industrial park.
- **Clearwell baffling** was installed in 2010. This ensures greater disinfection effectiveness by enhancing mixing and minimizing short-circuiting of finished water. Treatment effectiveness is enhanced through greater log reduction of pathogens prior to the first customer.
- In 2010, Tri City constructed **Raw Water Intake improvements**, including the installation of a fish-friendly Johnson Tee Screen (fish screen), airburst system, and related equipment.
- The Authority commissioned a **Water System Risk Failure Analysis** (HBH, 2011) in order to assess risk within the water system, and prioritize recommended improvements based upon the potential risks and outcomes, should various types of failure occur. This study reinforced the relative importance of various improvements called for in the Water Master Plan, including the Back Acres water storage tank, which is developed in the present study.
- In 2011, the Dyer Partnership was consulted to assist the Authority with the **analysis of potential operational improvements and conservations measures** that could be implemented in the water treatment plant (WTP). A copy of the technical letter can be found in Appendix D. The recommended improvements included modifications to the backwash cycle for improved efficiency, installation of a variable frequency drive (VFD) for the backwash pump, the installation of electrically actuated valves, and the future consideration of improvements of the backwash filter system and underdrains with an air scour system.
- In 2012 Midea Development provided recommendations to **improve the operational effectiveness of the current backwash treatment system**. The Authority implemented the

recommended improvements in 2012 during normal backwash basin cleaning operations. This will result in significant improvements to backwash water quality and treatment efficiency.

- In 2012 the Authority commissioned Midea Development to answer specific questions concerning the capacity and upgradeability of the water treatment system. The goal was to **identify the level of service that could be provided to the industrial park** on the west side of Interstate 5. The results of the technical letter provided general information, yet a more detailed, multi-year analysis would be required to answer the question with sufficient certainty. A secondary question was concerned with the level of improvements that would be required to provide higher levels of water service, should a larger development become a possibility. A copy of this letter can be found in Appendix D.
- In 2012, the Authority began planning for the **installation of a VFD** on the backwash pump, per the Dyer Partnership's recommendation. This will allow improved operational flexibility, performance, and efficiencies. The system will be integrated into the computer control system of the WTP, which will enable the Authority to optimize treatment, while minimizing waste and power usage. It could further reduce electrical loading and costs to the WTP. This improvement is to be completed prior to the publishing date of this study.
- The Authority **replaced existing lighting in its facility with high-efficiency lighting**, which is anticipated to significantly reduce electricity demand. The majority of existing lighting fixtures were replaced in approximately 2009, and then the remainder were replaced in 2012 and 2013. Older incandescent, halogen, and T12 florescent fixtures were replaced with more efficient T8 fixtures.
- The efforts above defined the need for this present study. Tri City developed the funding application that lead to the present grant. The overarching goal is to provide a recent analysis that answers important questions for the future, including the high-level pressure zone water storage needs, water system capacity, water conservation and efficiency. The Authority was **awarded grant funding for this study**, which supports this study, in part.

ES4 Water System Needs

The WMP provides detailed information concerning justifiable need for improvements to the water system. The Water System Risk Failure Analysis (HBH, 2011) reinforces and expands upon the recommendations called for in the WMP, with specific focus on a risk assessment framework. The improvement needs developed in these studies are specifically related to the demand projections developed in the WMP. This study developed an updated water system analysis, which includes actual recent demand characteristics and population values from the 2010 census. The present analysis validates the impacts resulting from thoughtful actions taken by Tri City, which has encouraged significant levels of conservation.

It is the general consensus of the above studies, and this study, that specific improvements are required for Tri City's water system, depending upon various factors, including actual water system demand. This is true, with some significant exceptions. Population and water demand are significantly less than projected in the WMP. Other exceptions will require additional analysis and consideration in order to develop practical and cost-effective improvement alternatives.

Water Rights Recommendations

Current water rights are adequate for future needs at current levels of usage and growth. However, this could change significantly should the level of demand increase due to development in the Industrial Park. Demand in excess of 1,000 gpm also creates additional required improvements to the water treatment facility, including the raw water pumps, and water treatment processes, which are discussed below. The raw water pump station should continue to be developed and improved in order to ensure that the full water rights can be utilized, which would enhance the discussion toward water right certification. Concerns with the raw water intake and pump station are limiting factors to the utilization of additional water storage rights. Water storage rights are presently available in Galesville reservoir. Tri City should pursue water rights as a practical and forward-looking part of their overall water system development strategy.

Intake & Pump Stations Recommendations

The mitigation of raw water intake and pumping concerns should be investigated in greater detail. During low river level periods, drawing the necessary flow rate has proven difficult. The infrastructure lacks automation, redundancy, and commonality of equipment. Various potential alternatives are provided in Section 7, which enables a beginning point for further analysis. The goal should be to provide cost effective solutions that progress full water right development, minimize operational risks, and ensure that water can be delivered even during low river level periods.

Water Treatment Facility Recommendations

Viable filtration alternatives that adequately consider the specific circumstances in Tri City must be developed. The alternatives developed in the WMP are inadequate to properly compare viable alternatives, and do not provide consideration for actual construction phasing requirements. Viable alternatives must consider operational and demand needs of the community. This dramatically affects the cost estimates and feasibility of the proposed projects developed in the WMP. Re-developed alternatives would include preliminary engineering cost estimates, and would provide valuable information to help Tri City to make informed decisions. A detailed investigation should be implemented to further consider the most practical and cost-effective alternatives that would meet the treatment goals of Tri City, and the needs of the community.

One New Potential Alternative

One new alternative that could be considered is the phased upgrading of the facility to the type of facility currently being constructed to treat the City of Sutherlin's Cooper Creek Reservoir water rights. The system Sutherlin selected is a Siemen's Trident HS (high solids) packaged conventional mixed-media system. This type of facility is more efficient and effective than conventional mixed-media filtration equipment, and is capable of treating surface water that is impossible to treat with membrane facilities. The system was compared to various membrane technologies and proved to be the most cost-effective and reliable alternative for Sutherlin. The Sutherlin WTP is designed to treat 5 cfs, which is similar to the needs of Tri City (4.87 cfs). This packaged equipment has dual treatment trains with tube settler modules capable of providing settlement and flocculation capacity prior to entering absorption clarification, and mixed-media filtration with media-retention underdrains, and an air scour system. Electrically actuated control valves can be utilized rather than pneumatic control valves, which enhances control and reliability of the facility, while also minimizing energy usage associated with pneumatically-actuated valve systems.

A major component to membrane treatment facility costs is the additional cost required for pre-treatment of raw water, prior to the membrane equipment. Tri City's WTP is currently deficient in settling and flocculation capacity. The WMP calls for an expensive improvement to the sedimentation / flocculation basins, yet this new alternative may be designed to negate the need for new basins. This alternative would utilize the existing sedimentation basin as a pre-settling and pre-flocculation basin, with secondary flocculation and settling occurring in the new treatment equipment.

The existing WTP would continue to operate while the new equipment is constructed. Once commissioned, the existing WTP would be decommissioned. The facility and mechanical piping would be sized to handle growth for the next 10 years, when the existing equipment would be demolished and an additional Trident treatment train would be installed in the existing building. This alternative would create a viable, simple and scalable water system, while utilizing as much of the existing equipment and infrastructure as possible. It is further recommended that Tri City staff tour the Sutherlin facility in order to gain a greater understanding of what is involved with the construction of a new facility, while continuing to operate an existing water treatment facility.

Summary
The most cost-effective medium-term solution for the water treatment facility is to to expand the capabilities of the existing facility in order to maximize the operational life of the existing plant. This could include the installation of additional mixed-media filters, while also installing new underdrains, electrically-actuated valves, filter media, air-scour systems, and repairing existing operational and mechanical deficiencies within the existing treatment facility. Additional settling and flocculation capability will be required in order to significantly enhance water treatment facility capacity; therefore additional filtration alone will not solve the existing operational and capacity deficiencies in the facility. Improvement of the existing facility could be combined with the alternative mentioned above. However, this functionally equates to the operation of two separate treatment facilities with different water treatment capacities and capabilities, which would result in significant operational difficulties. Finally, expenditures toward the existing facility should be carefully considered, as this capital could be invested toward the future needs of the water treatment system. The future water system will be considerably different than the current facility. Investments made toward the improvement of overall long-term deficiencies could prove to be more prudent than investments toward the enhancement of the existing facility. This topic must be considered in greater detail in order to enable an informed decision.

Back Acres Water Storage Tank Recommendations

The alternative chosen by the Authority for the Back Acres high-level pressure zone inline storage tank should meet long-term strategic requirements for the water system. This improvement will bring the overall storage system, and the high-level pressure zone within water system storage requirements. The high-level pressure zone is currently deficient in storage highlighting the need for this improvement. Table ES3 summarizes projected water system storage needs. The current minimum water storage deficiency is in excess of 250,000 gallons, but a larger storage tank is recommended to enhance fire suppression capability. A 350,000 gallon tank would bring the water storage system under compliance approximately beyond the year 2020, unless a large water user connects to the water system.

Table ES3 – Projected Water Storage Requirements

	Study Projection		Study Projected Storage Requirements		
Year	MMD	MDD	Storage	Need	Deficient
2013	748,799	1,216,798	2,017,300	2,060,998	(43,698)
2015	786,707	1,278,398	2,017,300	2,137,998	(120,698)
2020	890,086	1,446,390	2,017,300	2,347,988	(330,688)
2025	1,007,051	1,636,458	2,017,300	2,585,572	(568,272)
2030	1,139,386	1,851,502	2,017,300	2,854,377	(837,077)

Tank Site Alternative 2 is the recommended alternative for consideration. Other alternatives developed in this study are equally viable and have their own advantages and disadvantages. This affords Tri City excellent flexibility during the actual siting process for the project. Figure ES3 illustrates the recommended alternative, including alternatives for supply pipeline routing. A GPS survey was conducted in order to assess these alternatives and their viability. This incorporated various factors, including easy access, and excellent site characteristics, including drainage. Two possible routings are possible to the tank site, which provides flexibility during land acquisition negotiations and geotechnical studies. It could be possible with this alternative to leverage the improved fire flows, and improved service for development owners toward favorable land acquisition terms. In an ideal situation, the property owner would donate land for the new tank, and consider the tax benefits, and the future added value to the land as an excellent tradeoff. Alternative 2 provides the best combination of benefits, for only a 16% premium over Alternative 1. Figure 7.4.3 provides a conceptual engineering drawing of the water tank site for Alternative 2; however this conceptual drawing could be applied to any site during the actual design.

Table ES4 provides a summary of water storage tank alternatives that were considered in this study. Although Alternative 2 is modestly more expensive than Alternative 1 and Alternative 3, significant additional benefits can be realized, including enhanced serviceability characteristics and fire flows to the high-level pressure zone. The existing water tank would remain in operation, which would allow operations staff to service either tank without affecting the continuity of service to the high-level pressure zone. This alternative also enhances fire flows and minimizes risks to service to the existing community, as well as to new development that is likely to occur to the north and west of the current pressure zone.

Figure ES3 – New Tank Site Alternatives 2A / 2B / 3

Table ES4 – Back Acres Tank Site Alternatives Summary

Back Acres Inline Water Storage Tank Alternatives Summary Table			
Alternative No.	Total Alt. Cost	% Alt 1 Difference	Notes
Alternative 1A	$883,100	-	Least Cost, Residence Impacts, No Fire Flow Increase, No Supply Redundancy, No Growth Incentive.
Alternative 1B	$909,400	+ 3 %	Residence Impacts, No Fire Flow Increase, No Supply Redundancy, No Growth Incentive.
Alternative 2	$1,024,300	+ 16 %	Fire Flow Increase, Supply & Maintenance Redundancy, Growth Incentive.
Alternative 3	$1,110,500	+ 26 %	Fire Flow Increase, Supply & Maintenance Redundancy, Growth Incentive.

Please note that land acquisition costs are not included in these estimates, as this is impossible to determine at this time (other than the fair market value). Also note that the assumed tank size is 340,000 gallons, per the Water Master Plan (2006).

Study Analysis and Conservation Strategy

The development strategy for the present study was first to perform a comprehensive analysis of recent water system data in order to assess the current status of water system characteristics. The results of this analysis was contrasted and compared to a significant body of previous work. A brief summary of previous work is included in Section ES3, and is referred to in a multitude of locations throughout this study. The analysis and comparison work laid the groundwork for the development of the following discussion concerning the topic of conservation.

The study consultant presupposed that the WMP framework was acceptable, but that current water system characteristics, and demand projections, were very different. The study analysis clearly shows this to be true. Not only are current population characteristics very different than those called for in the WMP, but significant conservation has also been encouraged through the implementation of a new water rate structure in 2007. Midea used various sources of data, including the 2010 US Census, previous studies, water treatment system data, and other previous work to determine the current status of the water system.

The current status of the water system characteristics creates a unique opportunity to investigate additional means of conservation. This study develops the discussion in a comprehensive fashion. Examples of this include the deferral of major water system improvements, such as the diversion of righted water for a period of time. The Authority can utilize the present study as a tool, or guide, for when to develop specific water system needs on demand-based milestone criteria. Essentially, as demand reaches certain pre-established criteria, the Authority can act toward the development of needed improvements. Tri City can now plan for these improvements proactively.

The efficient use of energy and water resources maximizes the level of cost-effective service that can be provided to the community and defers necessary improvements into the future. This study develops a comprehensive list of projects that Tri City has implemented since 2006, and provides recommendations for additional conservation measures. The work presented proves that Tri City has actively pursued conservation projects as a means to minimize costs, as well as impacts to the environment. These projects include the installation of high-efficiency lighting fixtures and pump motor equipment, variable frequency pump drives, backwash system and operational improvements. The present study discusses other resource conservation topics such as backwash wastewater recycling.

<u>*Summary*</u>

One of the most effective conservation activities Tri City can pursue is the reduction of energy and resource use. Tri City is pursuing this as a part of its overall strategy, which includes the goal to maximize the useful life of the existing facility. This can only be accomplished if conservation-encouraging measures are adopted, such as the updated water rate structure. The strategy is further supported through capital projects and operational improvements that further reduce energy and water use. These projects include high efficiency pump motors and lighting, and other improvements discussed in this study. The actions taken by Tri City have enabled the water system to meet the needs of the community well into the near future. The inline water storage tank improvement developed in this study provides improved water service to the community, while also enabling operational flexibility and buffering of raw water diversion. Tri City is committed to taking further action toward conservation as discussed throughout this study, including the optimization of raw water pumping and treatment facilities, and operational improvements as well. Finally, this work will allow Tri City to defer the full development of water rights into the future. This includes the future improvement needs of the water treatment facility. The highly efficient facility will include state-of-the-art control and monitoring technologies, which will maximize efficiency and conservation. The demand-based tools developed in this study will serve to guide the future capital improvements needs for the community.

Section
1

Introduction

1.1 Background and Objective

Tri City Water & Sanitary Authority (Tri City, Authority) has diligently pursued the improvement of its water system in order to ensure excellent service to its customers. This study affirms the Authority's commitment to the community and answers key questions concerning various aspects of the water system. These aspects include the current status of water system characteristics, updated projections for future water system needs, conservation measures, and other important needs.

The need for this study became apparent over time, as the Authority has worked to prioritize improvements needs, address perceived system deficiencies, and accommodate potential growth. An important potential driver of water system demand could include development of the industrial park on the west side of Interstate 5. Development groups have asked the Authority to provide estimations of the level of utility service (water and wastewater) that can currently be provided, as well as what improvements would be required to accommodate a significant utility consumer. The answers to these questions require a detailed analysis of current water system characteristics, and a comprehensive understanding of the current status of the water system. The analysis forms the basis for answering other questions concerning the water system, including a new high-level pressure zone inline water storage tank (Back Acres), and conservation measures that could be implemented practically into the water system. This study provides a functional, milestone-based framework that will guide the efforts of the Authority in the coming years, including the level of service that could be provided to the industrial park should a developer become seriously engaged. The required demand characteristics will trigger the development planning for important improvements.

This study was funded in part (50% matching) through a grant awarded by the Oregon Water Resources Department (Grant Agreement No. 0056 13). In addition to the objectives above, a requirement of the grant funding includes statutory tasks that must be considered in the study. This discussion is developed in Appendix A.

1.2 Study Objective

The primary objective of the present study is to provide a preliminary engineering study for an inline water storage tank within the high-level pressure zone of Tri City's water system. Other objectives of the study are summarized in the bulleted list below. The analysis is supported by a comprehensive analysis of recent water system data and previously published work.

- Compile necessary data, including the validation of the Water Master Plan data. Ensure that water requirements are adequate to meet water system needs.

- Identify potential alternatives to improve current deficiencies in the water system.

- Evaluate each alternative, including preliminary engineering cost estimates for each.

- Provide recommendations for the most feasible and cost effective alternative to meet water system needs.

- Provide recommendations for water conservation and efficiency measures.

In addition to the objectives above, grant funding requirements call for specific statutory tasks to be considered in the study. These tasks are regarded in more detail in Appendix A.

1.3 Scope of Study

This study mirrors the format provided in the WMP in order to ensure that the context of the material can be properly connected to specific sections. The study should be considered an updated investigation of the current status of the water system, with revised recommendations for improvement milestones, conservation measures, and a detailed analysis for the requirements of the high-level Back Acres pressure zone.

Planning Period

The planning period for the WMP provides the overall framework for the needs of the Authority's water system through the year 2026. This study is supplementary to the WMP, and provides more recent analyses and important updates to specific topics of the plan.

1.4 Authorization

Tri City Water & Sanitary Authority contracted with Midea Development, LLC on July 6, 2012 to develop the Back Acres Preliminary Engineering Report. The scope of this Plan was based on a Scope of Engineering Services included in the contract with the City, and within the grant agreement.

1.5 Acknowledgements

The development of this preliminary engineering report would not have been possible without the combined efforts of many individuals and agencies. The participation of these parties in collecting data, answering questions, reviewing drafts, and providing guidance for this report is greatly appreciated. The efforts and expertise of Paul Wilborn, Bill Thomas, Tri City office staff, and the Tri City Board of Directors are of particular importance.

The assistance and cooperation of the Oregon Department of Human Services, the Drinking Water Program, and the Oregon Department of Water Resources in the development of the Plan is appreciated, especially the participation of Mr. Scott Curry of the Drinking Water Program and Mr. Bill Fujii of the Department of Water Resources. This project would have been difficult to fund and to develop without the support Mr. Fujii provided.

Section

2

Study Area Characteristics

The Tri City study area characteristics are well developed in the Water Master Plan (WMP) (HBH, 2006). Specific topics are included below to provide additional or updated information to their current status when appropriate for the development of this study.

2.1 Socioeconomic Environment

Population

The WMP developed current and projected population through the utilization of a disclosed methodology. Population projections are typically developed through the utilization of census data, population research data, and with the coordinated population projection data in the comprehensive plan of Douglas County. This study presented an opportunity to compare the recent 2010 census data with the projection developed in the WMP. This can be used as a tool by the Authority to ensure that adequate levels of service are provided to actual users, as compared to the generic projections.

Figure 2.1.1 illustrates the comparison between the population projections provided in the WMP, and the current status of population in the area based upon the 2010 US Census.

Figure 2.1.1 – Population Comparison – WMP vs. 2010 Census

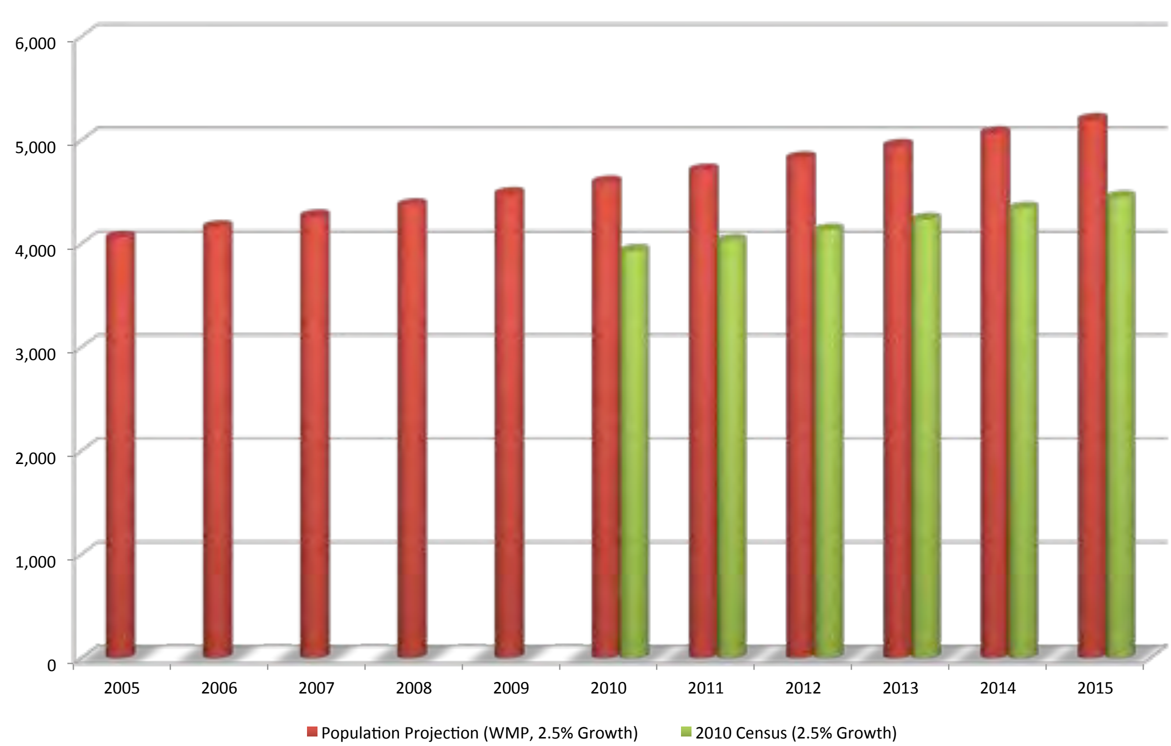
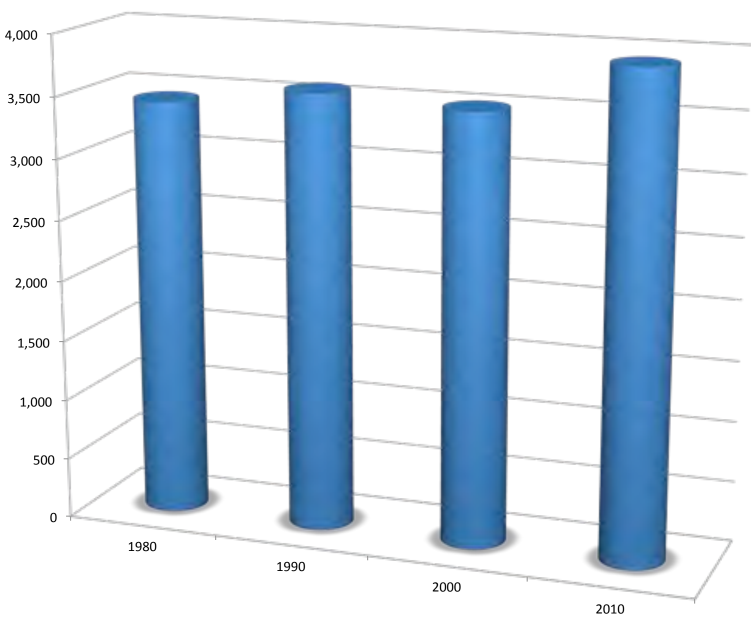


Figure 2.1.1 illustrates an updated population projection (green bars) with the growth assumptions developed in the WMP (red bars). The 2010 census population is actually less than the WMP projection for 2005. This results in a significant gap between the WMP developments of needs as compared to the actual population needs. Population is the critical driver of demand for a water system. The actual needs of the community, rather than generic population projections, should be thoroughly considered in the planning of infrastructure projects. The analysis conducted in this study shows that the current population lags behind that shown in the WMP by approximately seven years.

Historic Population

Historic census population data can be used for a general understanding of growth in the Tri City area. Figure 2.1.2 shows historic census data from 1980 – 2010. The population remained fairly stable in the earliest decades, at just below 3,500 residents. The total population grew 11.7% from 2000 – 2010. Although this level of growth is not necessarily expected to continue, it can be used as a general tool to verify our understanding of population in the area.

Figure 2.1.2 – Historic Census Population Data



Future Population

If we assume the same level of growth illustrated between 2010 – 2020 and between 2000 - 2010, then the 2020 population would be approximately 4,390. For the sake of comparison, the WMP projects a 2020 population of 5,879, which is nearly 1,500 residents in excess of our analysis performed above (11.7% growth for the decade). This is a significant deviation. However, it should be noted that the analysis developed in the WMP is based upon required methodologies, and has been reviewed by the Oregon Health Authority.

Section 3

Regulatory Environment

Regulatory requirements were well developed in the Water Master Plan (HBH, 2006). This study does not update the work performed.

Water Use and Projected Demands

Section

4

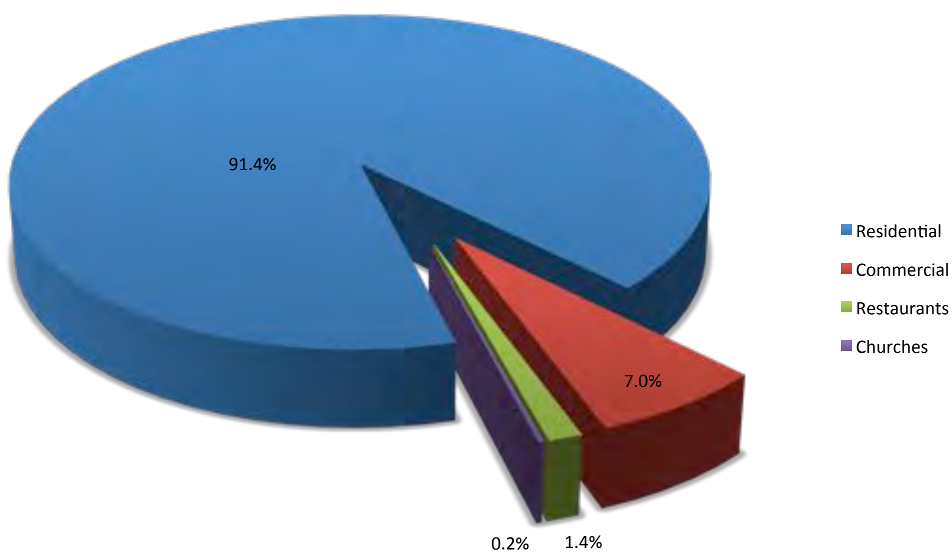
Tri City Water & Sanitary Authority (Tri City, Authority) has diligently pursued the improvement of its water system in order to ensure excellent service to its customers. This study analyzes the past decade of water system data in order to understand the current status of the water system. The last three years of data are analyzed in detail and compared to previous work, including the Water Master Plan (WMP) (2006, HBH), and the water rate study (2007). The goal of this analysis is to understand current water system characteristics as a means to ensure that excellent service can continue.

4.1 Current Water Demand

Current water system demand has been thoroughly evaluated, including water consumed, water treated, and water diverted. The development of this analysis enhances a comprehensive understanding of water system characteristics. The characterization of the current water system will procure an understanding of water system efficiency, including a reasonable potential explanation of unaccounted water losses.

Residential sources account for approximately 91 percent of all water consumed within Tri City. Commercial, churches, and restaurants account for the remaining 9 percent of water consumed, as shown in Figure 4.1.1. It is important to note that actual billed water consumption by consumers does not include losses from the water system, including non-accounted water.

Figure 4.1.1 – Average Water Consumption by Type (2010 – 2012)



Equivalent Dwelling Units

This study does not redefine the EDU as developed in the Water Master Plan. Although water consumption has decreased in the recent years, for various reasons, the current EDU consumption value will result in a conservative planning criteria.

Water Treated

A detailed explanation of metered consumption is critical for the understanding of financial and user-type characteristics within a water system. Water system planning requires a thorough look at water production, to ensure that water leakage and unaccounted water are considered. The methodology makes certain that water demands from the system can be accommodated. Treated water produced, pumped to the water distribution system, and utilized for backwash and wasting cycles are discussed below.

Water Treatment Plant Production

Tri City operations staff maintains detailed records concerning various aspects of water treatment plant (WTP) operations. Treated water is conveyed to the water distribution system for user consumption. This treated water produced by the WTP is typically the difference between water diverted from the source, and the water utilized within the boundaries of the WTP. The water used by the WTP includes backwash water, wastewater, and other water consumed at the site.

Water production characteristics were well defined in the Water Master Plan (WMP). This study assumes that the peaking factors developed in the WMP continue to be valid. This assumption will result in the conservative analysis. These peaking factors are applied to the water system characteristics developed in this study from water records. The definition of the water system characteristics are illustrated in the WMP, and include Average Daily Demand (ADD), Maximum Monthly Demand (MMM), Maximum Daily Demand (MDD), and Peak Hourly Demand (PHD). These water system characteristics are important to properly assess, plan, and design water system infrastructure as discussed in the WMP.

This study investigated water system characteristics over the last decade, from 2001 – 2012. This was accomplished with the thorough analysis of water system records for the years 2010 – 2012, and the review of previous water system planning documents, including the WMP, and the Water Rate Study (2007). Figure 4.1.1 summarizes water system characteristics between 2001 – 2012.

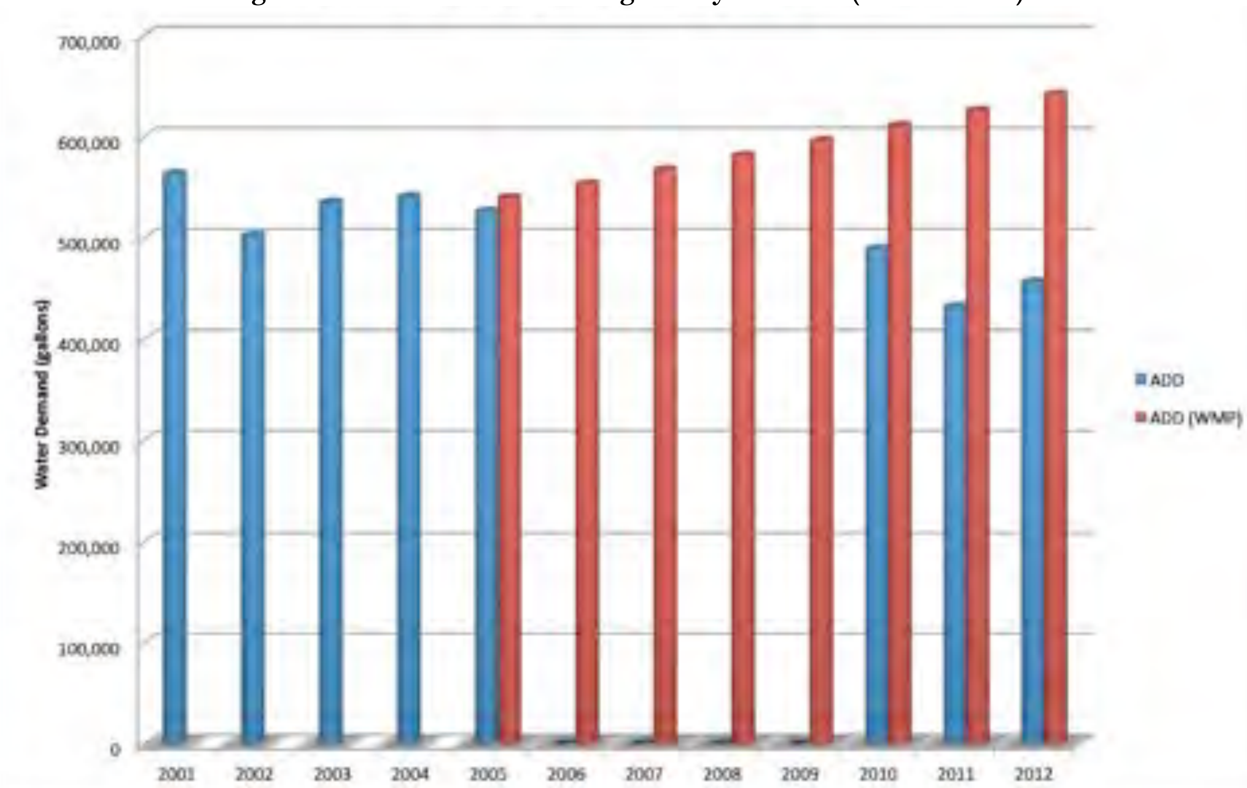
Table 4.1.1 – Historic Water Demand (2001 – 2012)
(Units are in gallons per day unless otherwise noted)

	AAD (gallons)	ADD* (WMP)	ADD	MMD	MDD	PHD	Notes
2001	205,583,800		563,243	901,189	1,464,433	2,816,216	WMP
2002	183,322,900		502,255	803,607	1,305,862	2,511,273	WMP
2003	195,308,600		535,092	856,147	1,391,239	2,675,460	WMP
2004	197,397,000		540,814	865,302	1,406,116	2,704,068	WMP
2005	192,009,737	540,000	526,054	841,687	1,367,741	2,630,270	WRS
2006	202,027,500	553,500	-	-	-	-	WMP
2007	207,078,370	567,338	-	-	-	-	WMP
2008	212,255,165	581,521	-	-	-	-	WMP
2009	217,561,535	596,059	-	-	-	-	WMP
2010	178,499,353	610,960	489,039	782,463	1,271,502	2,445,197	This Study
2011	157,746,490	626,234	432,182	691,491	1,123,674	2,160,911	This Study
2012	166,653,345	641,890	456,585	730,535	1,187,120	2,282,923	This Study

* This column reflects the future projection developed in the WMP.

Figure 4.1.2 shows a compilation of data from the Authority's water system for the past decade. Data from 2001 – 2005 was utilized to develop the WMP. WMP ADD demand projections can be seen in the figure. Population projection comparisons between the WMP and this current study can be viewed in Section 2. The new water rate structure implemented in 2007 is based on a purely consumptive model, which typically encourages conservation by consumers. The data presented for the years 2010 – 2012 illustrate the comparison between the WMP and the analysis performed by the consultant (only for ADD). It is clear that the current demand characteristics in the service area are very different than those called for in the WMP. Since the same peaking factors are assumed in this study, as developed in the WMP, other factor comparisons would reflect the similar trend shown in this figure.

Figure 4.1.2 – Historical Average Daily Demand (2001 – 2012)



Nonaccount Water

Water volume sold is always less than the treated water conveyed to the distribution system. This is due to leaks and other unmetered water, including some water used at the WTP. It is not entirely clear how much miscellaneous water is used at the WTP, but this amount is very small, and does not affect this analysis. Water system losses have been characterized in this study for the years 2010 – 2012, which are summarized in Figure 4.1.3. These losses are basically the difference between the amount of water conveyed to the distribution system, and the water actually sold. It should be noted that a significant drop in water losses occurred in 2012. This may be attributed to a significant water leak discovered and repaired by Tri City staff near the water treatment facility. This leak was located after leaving the water treatment facility, but prior to the first customer. This could be a significant improvement, yet one or more additional years of WTP data should be evaluated to ensure that no other factor could have caused the improvement. Previous to the improvement, water system losses were within a similar level as those developed in the WMP. Table 4.1.2 summarizes water system losses for each year as well as the three-year average.

Figure 4.1.3 – Historic Water System Losses (2010 – 2012)

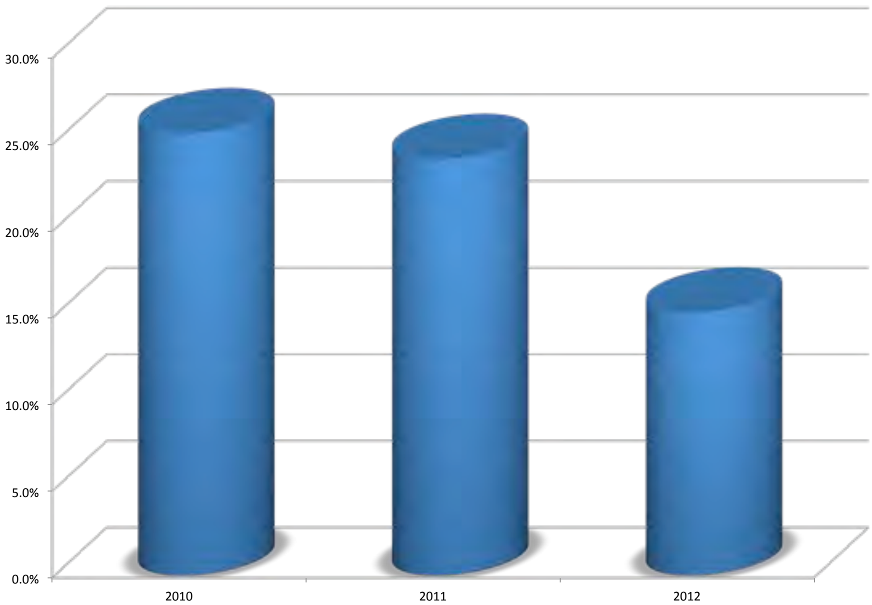


Table 4.1.2 – Historic Water System Losses (2010 – 2012)

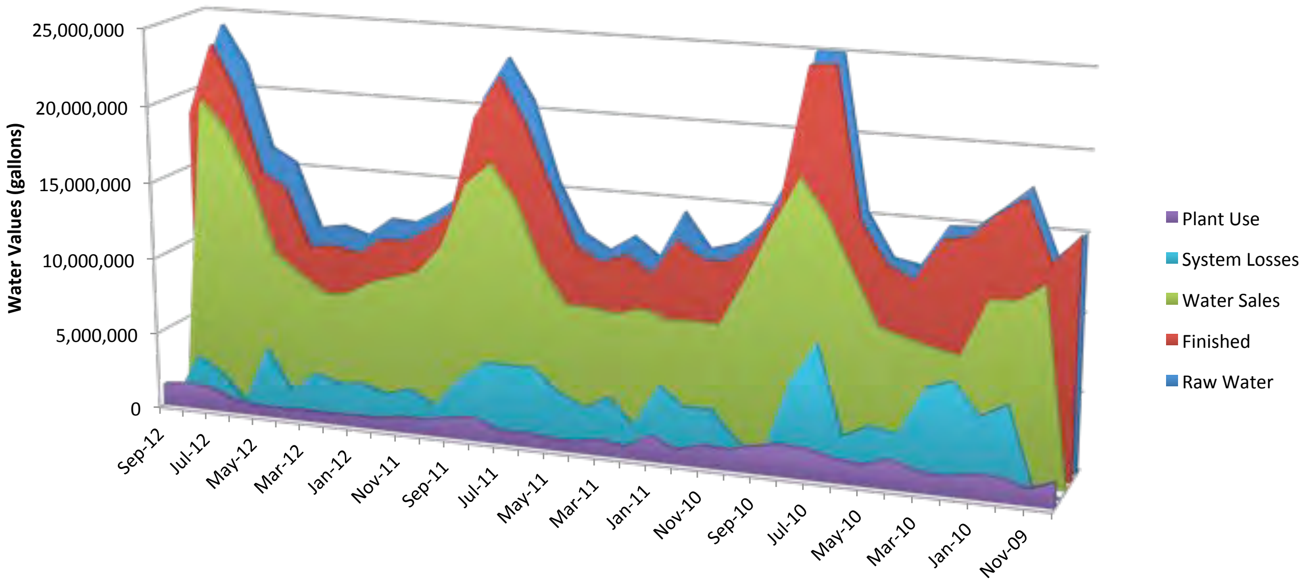
Average 3 Year System Loss	21.8%
2009-10 Actual System Loss	25.5%
2010-11 Actual System Loss	24.0%
2011-12 Actual System Loss	15.2%

Other potential sources of water losses could include leakage, inaccurate flow meters, unauthorized connections, and other unmetered water such as hydrant flushing, fire suppression, or water tank flushing. Tri City should continue to identify and eliminate unaccounted water. Reductions in water losses can result in increased revenue, conservation of source water, and the longevity of the water system to meet future water needs.

Water System Characteristics

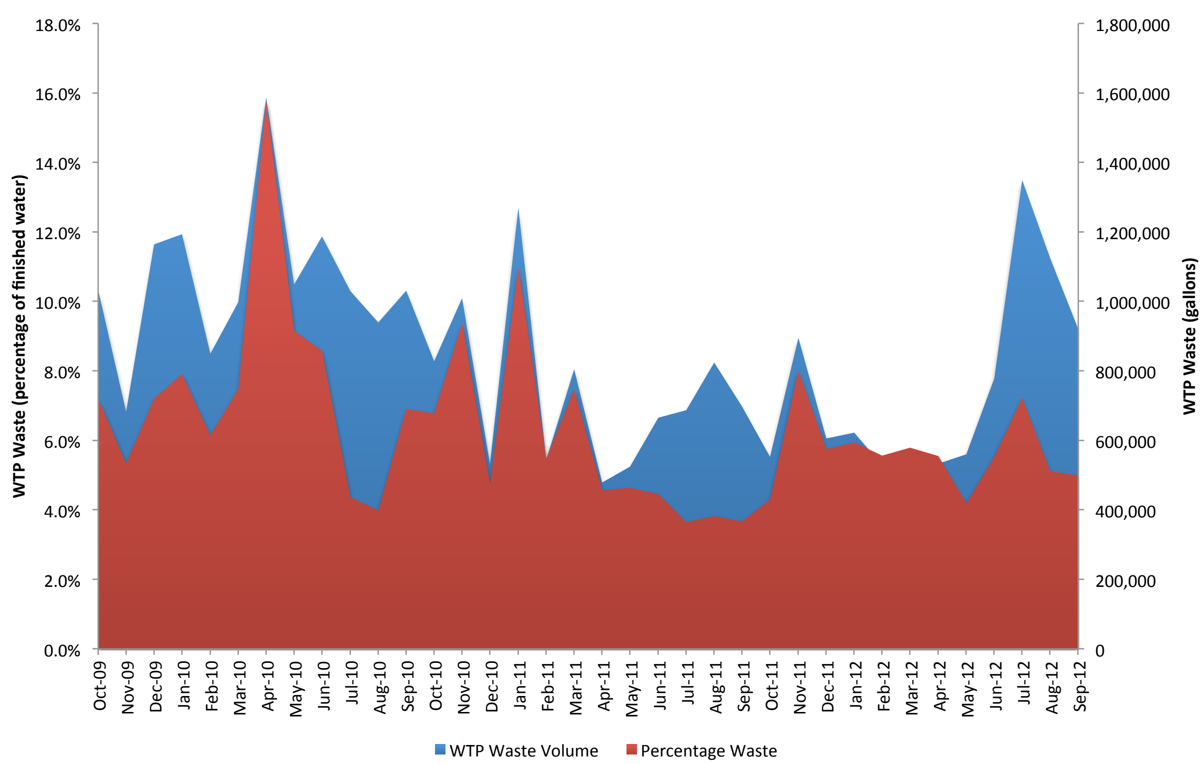
The overall performance of Tri City's water system can be summarized in one efficient graphic as shown in Figure 4.1.4. The graphic illustrates total monthly values over the study period for raw water diverted, finished water produced, water sold, plant use, and system losses. *It is important to note that the horizontal axis represents time from right to left.* Raw water diverted is shown in the deepest background layer (dark blue), with a peak monthly diversion of approximately 25 million gallons. The next layer is finished water produced (red). The difference between these layers is equivalent to the WTP waste. This can be seen as the dark blue band that is visible above the red profile. Water sales are illustrated in green. The difference between the red and green profiles results in the visualization of water system losses, which is also individually illustrated with the light blue profile. Plant use is shown in the foreground in violet. It should be noted specifically that one clear trend is apparent, which is that water system losses have been reduced over the study period. As noted previously, Tri City staff located and repaired a water leak in the system after the master meter, and prior to the first customer. This could have potentially contributed to this reduction.

Figure 4.1.4 – Historic Monthly Water System Characteristics (2010 – 2012)



The overall performance of Tri City's water treatment plan can also be summarized in one efficient graphic as shown in Figure 4.1.5. The graphic illustrates total monthly values over the study period for actual WTP waste on one axis, and the percentage of total treated water produced on a secondary axis.

Figure 4.1.5 – Historic Monthly Water Treatment Plant Waste (2009 – 2012)



It should be noted that during times of poor raw water quality (late summer and winter), WTP efficiencies and total waste water volumes increase. This is to be expected. It should be also noted that waste volumes have decreased and treatment efficiency has increased over the study period, with the exception of the summer period of 2012. This is expected as Tri City is working to improve its overall efficiencies through focused operational and equipment improvements.

Summary

Current water system characteristics were analyzed in detail for 2010 – 2012, and the entire previous decade of data was reviewed in order to create a comprehensive picture of the current status of the water system. This data was utilized to assess the system, evaluate future requirements, and investigate potential conservation measures that can be implemented to ensure quality service to customers. The results of this analysis are presented throughout this study.

4.2 Projected Water Demand

Future water demand can be difficult to accurately project. The WMP developed a 20-year planned projection, which also calls for specific capital infrastructure improvements that would be required to meet the demand. Although these projections are not valid temporally, they continue to be valid on a milestone basis. For example, the plan calls for a specific level of demand in the future, driven by projected population growth, which would require specific improvements to the WTP capacity. Then it provides alternatives for how to meet the demand with these improvements. Tri City should periodically review water system characteristics in order to track actual demand milestones, and then reflect to the WMP, and this study, for guidance. It is reasonable to anticipate the actual timing of needed projects, rather than simply applying generic timelines. This analysis can also be utilized to assess the water right needs of Tri City in the future, which is discussed in Section 7.

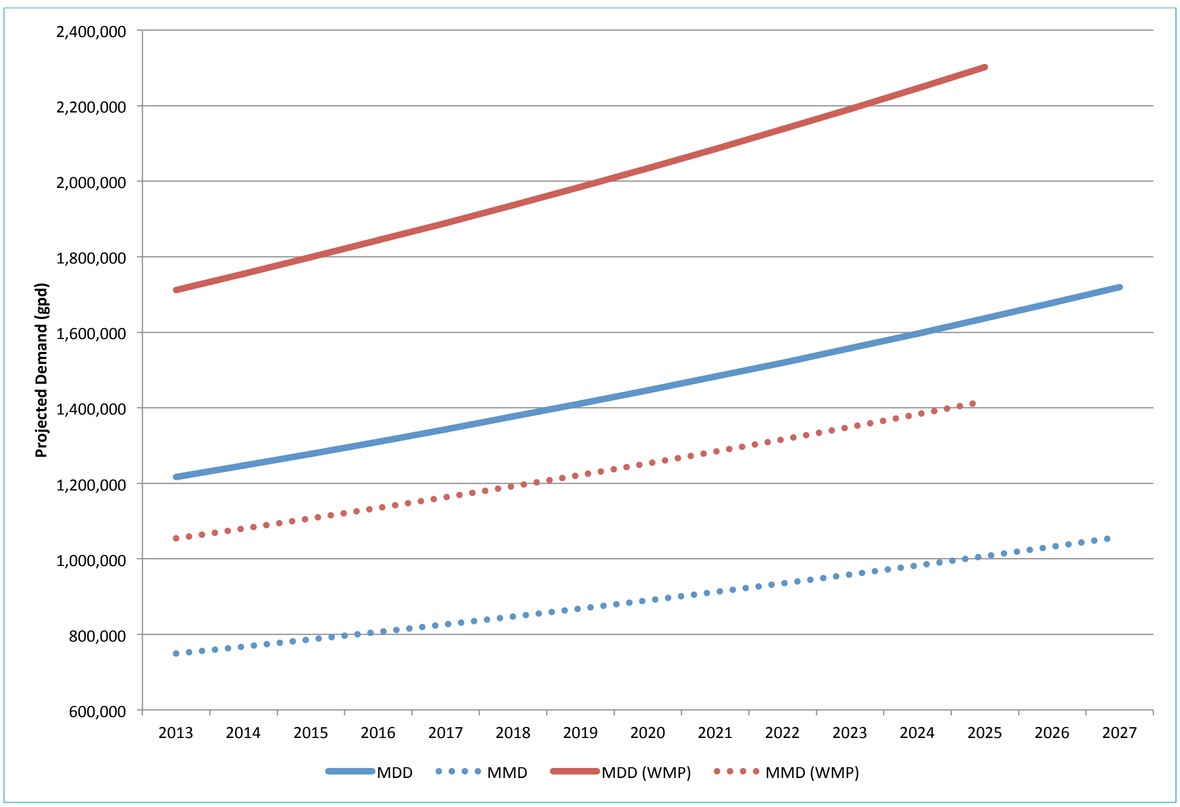
Projected demand values developed in this study assume the equivalent level of growth as provided in the WMP (2.5%) beginning from 2012 data. Table 4.2.1 summarizes the comparison between the WMP projection, and the projection developed in this study. Table 4.2.1 can be used as a tool during the planning efforts for needed capital improvement projects. This tool enables a demand milestone-based capability, which relates back to the improvements called for in the WMP. It should be noted that the new projected level of demand does not equal the 2013 projected value from the WMP, until beyond the 20-year planning period of the WMP. This is noted in the table in red and in blue. Figure 4.2.1 is a graphical illustration of Table 4.2.1.

Table 4.2.1 – Comparison of Projected Demand*

	Water Master Plan Projection		Study Projection	
Year	MMD	MDD	MMD	MDD
2013	1,053,919	1,711,856	748,799	1,216,798
2014	1,080,266	1,754,652	767,519	1,247,218
2015	1,107,273	1,798,518	786,707	1,278,398
2016	1,134,954	1,843,481	806,374	1,310,358
2017	1,163,328	1,889,568	826,534	1,343,117
2018	1,192,412	1,936,807	847,197	1,376,695
2019	1,222,222	1,985,228	868,377	1,411,112
2020	1,252,777	2,034,858	890,086	1,446,390
2021	1,284,097	2,085,730	912,338	1,482,550
2022	1,316,199	2,137,873	935,147	1,519,614
2023	1,349,104	2,191,320	958,526	1,557,604
2024	1,382,832	2,246,103	982,489	1,596,544
2025	1,417,403	2,302,255	1,007,051	1,636,458
2026	-	-	1,032,227	1,677,369
2027	-	-	1,058,033	1,719,303

* Values are compared between the projections in the Water Master Plan (HBH, 2006) and this preliminary engineering report.

Figure 4.2.1 – Comparison of Projected Demand*



Section

5

Design Criteria and Service Goals

This section is adequately developed in the Water Master Plan (HBH, 2006).

Section

6

Existing Water System

Tri City's water system is well characterized and presented in the Water Master Plan (WMP) (HBH, 2006). The focus of the current study is to verify the status and re-characterize various aspects of the water system, for quality assurance purposes.

6.1 Current Status of the Water System

The bulleted list below is a chronological summary of recent efforts relating to the water system.

- The Authority developed a **Water Master Plan** (HBH, 2006), which provides infrastructure assessment and planning for the coming twenty years (2006-2026).
- In 2007, the Authority commissioned a **water rate study** and implemented its recommended water rate structure. The goal of the new structure was to encourage maximum conservation within the system through a purely consumptive model. Water demand fell significantly, which is supported by the analysis provided in this study.
- Authority installed an **on-demand hypochlorite disinfectant generation system** in 2008, which reduces the potential of disinfection byproduct risk, enhances disinfection effectiveness and efficiency, and reduces chlorine gas system health and safety risk. This also reduces cost and supply chain risks associated with chlorine gas systems.
- Tri City procured the **Pruner Road Hotel Impact Study** (Civil West, 2008) to study the resources and improvements that would be required to service a hotel in the industrial park.
- **Clearwell baffling** was installed in 2010. This ensures greater disinfection effectiveness by enhancing mixing and minimizing short-circuiting of finished water. Treatment effectiveness is enhanced through greater log reduction of pathogens prior to the first customer.
- In 2010, Tri City constructed **Raw Water Intake improvements**, including the installation of a fish-friendly Johnson Tee Screen (fish screen), airburst system, and related equipment.
- The Authority commissioned a **Water System Risk Failure Analysis** (HBH, 2011) in order to assess risk within the water system, and prioritize recommended improvements based upon the potential risks and outcomes, should various types of failure occur. This study reinforced the relative importance of various improvements called for in the Water Master Plan, including the Back Acres water storage tank, which is developed in the present study.
- In 2011, the Dyer Partnership was consulted to assist the Authority with the **analysis of potential operational improvements and conservations measures** that could be implemented in the water treatment plant (WTP). A copy of the technical letter can be found in Appendix D. The recommended improvements included modifications to the backwash cycle for improved efficiency, installation of a variable frequency drive (VFD) for the backwash pump, the installation of electrically-actuated valves, and the future consideration of improvements of the backwash filter system and underdrains with an air scour system.
- In 2012 Midea Development provided recommendations to **improve the operational effectiveness of the current backwash treatment system**. The Authority implemented the recommended improvements in 2012 during normal backwash basin cleaning operations. This will result in significant improvements to backwash water quality and treatment efficiency.

- In 2012 the Authority commissioned Midea Development to answer specific questions concerning the capacity and upgradeability of the water treatment system. The goal was to **identify the level of service that could be provided to the industrial park** on the west side of Interstate 5. The results of the technical letter provides general information on this matter, yet a more detailed, multi-year analysis would be required to answer the question with sufficient certainty. A secondary question warranted the level of improvements that would be required to provide higher levels of water service, should a larger developer become interested. A copy of this letter can be found in Appendix D.
- In 2012, the Authority began planning for the **installation of a VFD** on the backwash pump, per the Dyer Partnership's recommendation. This will allow improved operational flexibility, performance, and efficiencies. The system will be integrated into the computer control system of the WTP, which will enable the Authority to optimize treatment, while minimizing waste and power usage. It could further reduce electrical loading and costs to the WTP. This improvement is to be completed prior to the publishing date of this study.
- The Authority **replaced existing lighting in its facility with high-efficiency lighting**, which is anticipated to significantly reduce electricity demand. The majority of existing lighting fixtures were replaced in approximately 2009, and then the remainder were replaced in 2012 and 2013. Older incandescent, halogen, and T12 florescent fixtures were replaced with more efficient T8 fixtures.
- The efforts above define the need for this present study. Tri City developed the funding application that lead to the present grant. The overarching goal is to provide a recent analysis that answers important questions for the future, including the high-level pressure zone water storage needs, water system capacity, and water conservation and efficiency. The Authority was **awarded grant funding for this study**, which supports this study, in part.

Improvement Needs

Section
7

7.1 Background

The improvement needs developed in the Water Master Plan (WMP) (HBH, 2006) provide detailed information concerning the water system. The Water System Risk Failure Analysis (HBH, 2011) reinforces and expands upon the recommendations called for in the WMP, with specific focus on a risk assessment framework. The improvement needs called for in these studies specifically relate to the demand projections developed in the WMP.

This present study develops an updated water system analysis, which includes actual recent demand characteristics and population values from the 2010 census. Tri City's restructured water rates were implemented following the Water Rate Study (WRS) (2007). The present analysis validates the impacts resulting from thoughtful actions of Tri City, which has encouraged significant levels of conservation.

It is the general consensus of the above studies, and this study, that specific improvements are required for Tri City's water system. This is true, with some significant exceptions. Population and water demand are significantly less than projected in the WMP. Section 4 provides an updated evaluation of water demand and provides comparisons to the projections developed in the WMP. This study provides a milestone-based tool to help with the development of improvement planning, including practical and conservation-encouraging improvements, while also minimizing risk to the water system.

This section develops additional information concerning the need and milestone-based criteria for specific improvements.

7.2 Raw Water Sources and Water Rights

This section will provide updated information concerning raw water sources and water rights.

7.2.1 Water Rights

Tri City's water rights analysis was properly developed in the WMP, although the demand characteristics are currently much different than illustrated in the WMP. The current total water right can provide adequate water supply throughout the study period of the WMP. The primary risks associated with the water rights include the relatively frequent reduction of Tri City's junior water rights during low stream flows. This is further complicated by difficulties experienced with the raw water intake and pump station, as discussed below. This analysis will focus on low stream flow periods, with the goal of providing a quantitative tool for assessing water supply needs for the future.

The analysis from Section 4 shows that the current water rights in Galesville reservoir should be adequate throughout the study period of the WMP. The analysis from page 7-1 of the WMP, applied to the current analysis, shows that during the low flow period, with water right reductions, the Galesville water right will allow approximately 100 days of supplemental flow for the MMD of approximately 865,000 gallons. The present analysis shows this should not occur prior to the year 2019. It should be noted that this circumstance could change for various reasons, including the scenario that significant development occurs in the industrial park.

Recommendations

The current water rights are adequate for the needs into the near future at current levels of usage and growth. However, this could change significantly should the level of demand increase due to development in the Industrial Park. Demand in excess of 1,000 gpm also creates additional required improvements to the water treatment facility, including the raw water pumps, and water treatment processes, which are discussed below. The raw water pump station should continue to be developed and improved in order to ensure that the full water rights can be utilized, which would enhance the discussion toward water right certification. Finally, Tri City should pursue additional water storage rights in Galesville reservoir as soon as possible in order to ensure that future needs can be met.

7.2.2 Intake and Pump Station

The existing water treatment system effectiveness is reduced as diverted flows extend beyond approximately 1,150 gpm, due to limitations of the sedimentation / flocculation system. This will be discussed below, but is mentioned here to provide context for the raw water intake and pump station discussion. When future demand requires flows in excess of 1,150 gpm, both raw water pumps will need to simultaneously operate in order to meet the demand as well. During low stream level periods, the raw water intake capacity is limited to between 900 – 1,200 gpm, which creates a third limiting factor for the water treatment system. This is the most critical and important deficiency to mitigate. Tri City installed a new intake fish screen system in 2010. The capacity of the system is more than adequate to meet future demand through at least 2025. However, during low river level periods, the capacity of the intake system is significantly reduced due to low water levels. The raw water intake is hydraulically capable of supporting flow of 1,740 gpm. This is more than adequate to support growth through the year 2025, so long as a major industrial source of water is not added to the system. The addition of one additional fish screen and piping should allow for hydraulic flows in excess of the current water rights of 2,186 gpm.

Tri City could mitigate the low water flow rate deficiency in a number of ways, which should be considered in detail beyond this study. Here are some potential alternatives for further investigation:

- Reconstruct the intake to ensure full submersion of equipment, specifically during a low river level period.
- Construct a raw water storage tank that would provide buffering to the system. This tank could be filled during off-peak periods, or when the water treatment facility is not operating. The system could be easily controlled in an automated fashion.
- Change the point of diversion downstream to a deeper site, more suitable to ensure full submersion. This would require the construction of a new intake, pipework, and potentially more significant equipment.
- Construct a third pumping system that can provide suction pumping capabilities. This would enable adequate flows beyond the gravity-based hydraulic capacity of the intake to the wetwell, which is specifically only a problem during low river level periods.
- Construct active PID control with level sensing of the wetwell to ensure that raw water pumps are paced to water replenished into the wetwell.

The intake pump station is currently comprised of a newer Flygt Model NP3171.180 Type HT (installed 2004) and a Fairbanks Morse Model D5433MV (installed 2000). The Fairbanks pumps originally installed had various and persistent problems, which led to multiple rebuilds and significant costs and man-hours to resolve. This led to the installation of the Flygt pump in 2004, which has operated well. The old Fairbanks Morse pump has been rebuilt and now functions as a standby pump for the pump that is still operational.

The full water right cannot currently be delivered to the treatment process by the pump station. With both pumps operating, less than 2,000 gpm can be delivered. Note that the current capacity of the raw water intake is only 1,740 gpm. Not only is this less than the current water right, but this also leaves no redundancy for the pumping operations. Here are some potential alternatives to consider further:

- Replace pumps and select the proper pump to individually deliver the full water right of 2,185.8 gpm. This would require a significantly larger pump than the current pump. Two of these pumps could also be costly, and driving the pumps near 50% pumping capacity could sacrifice the benefit of the high efficiency pump motors.
- Replace pumps and select three identical pumps, two of which would deliver the full water right, and one pump could deliver the current approximately 1,000 gpm. This alternative would allow additional operational flexibility and reliability. In the event a pump failed, the third pump could be swapped out with the pump that requires service. The existing wetwell is unlikely to support installation of a third pump, due to size restrictions.
- A phased alternative could also be viable, and could improve the pumping situation toward an automated solution. When maximum daily demand approaches the current water right, and the intake requires improvements to accommodate the demand, a second wetwell could be constructed and tied together with the existing wetwell. The third pump could be installed in the new wetwell, which would not only accommodate the necessary demand, but could also switch and alternate in an automated fashion. The automation would function to evenly wear pumps and to automatically switch should another pump fail, or trigger a service alarm.

Recommendations

Alternatives should be pursued to mitigate the low water level condition, and the lack of redundancy. The various alternatives discussed above should serve as a starting place for this discussion.

7.3 Water Treatment Facility

The WMP develops a discussion concerning operational characteristics and deficiencies of the water treatment facility, including flocculation / sedimentation, filtration, disinfection, and finished water conveyance. Each section then provides the justification of need for various improvements, including alternatives. The water treatment facility has various process bottlenecks, which could be addressed in any number of ways. The following subsections will discuss the most practical solutions concerning the current status of the system, and also consider updated demand projections.

7.3.1 Flocculation / Sedimentation

Flocculation and sedimentation currently limit the capability of the WTP to deliver greater volumes of treated water without sacrificing finished water quality. If the existing WTP technology continues to be utilized, additional flocculation and sedimentation capacity will be required. Water quality begins to decrease at a flow rate of approximately 1,150 gpm. This flow rate is also greater than a single raw water pump can deliver from the raw water pump station. Although two pumps can be operated simultaneously, this also reduces operational redundancy, and raw water pumping capacity becomes the process bottleneck, as discussed in Section 7.2.2.

7.3.2 Filtration

The WMP develops the limitations of the current WTP, and provides various potential alternatives for expanding filtration capacity. These include expansion of the current plant with similar technology, a packaged membrane microfiltration system, and an immersed membrane ultrafiltration system. A review of the alternatives reveals that each alternative is limited by a lack of understanding of water treatment facility construction processes, and robust engineering cost estimation. The current facility must continue to operate, or only cease to operate for short periods of time, during construction of upgraded facilities. It is beyond the scope of this study to fully develop cost estimates and alternatives to the water treatment facility, but the goal of this discussion is to provide information that could help Tri City develop plans to successfully address the concerns discussed below.

7.3.2.1 Option A – Conventional Granular Media Filtration

The expansion of the existing WTP with conventional media filtration equipment is the most viable and cost-effective alternative, though various difficulties could arise from the improvement as shown. The cost estimate is inadequate, and it does not consider the importance of continuity of treatment operations during construction. It calls for the removal of the flocculation and sedimentation equipment internal to the building to provide adequate room for the new mixed-media filters. This would require significant demolition and modifications to the existing building. The existing treatment processes would be inoperable for a considerable period of time during this construction. A new flow splitting and influent piping structure would be necessary, and would need to be constructed prior to the demolition of the current flocculation and sedimentation structure, as it currently integrates the flow splitter and influent piping. The piping could also require significant earthwork below the ground, which requires additional expansion of demolition activities. Modifications of restrictions in the current piping are also problematic to continued operation of the facility during construction, with potentially significant impacts to treatment operations. The WMP states that virtually all of the mechanical piping will require significant modifications, without providing a viable plan for how treatment operations will continue to operate during the improvements. The majority of the building will need to be removed, and rebuilt, which was not accounted for in the discussion and cost estimation. All of these factors increase the costs of these facility improvements, and add significant risk to this alternative.

7.3.2.2 Option B – Package Membrane Microfiltration

Conversion of the existing WTP to a packaged membrane microfiltration system holds many of the challenges mentioned above, but additional and more serious concerns are prevalent. These similar challenges include significantly insufficient cost estimates for the improvement, continuity of treatment operations, demolition scope and scale, to name a few. The most important oversight is that the existing facility must continue to operate during construction of the new equipment. The WMP shows the new equipment in place of the existing equipment, in the same building. This alone makes this alternative unviable unless the design and estimations are significantly modified. The alternative also neglects important elements of a membrane treatment facility, including significant tankage and mixing systems, compression air systems, storage of additional chemicals, and other significant oversights. This results in an infeasible alternative and insufficient engineering cost estimates. Figure 7.3.1 shows a Pall membrane WTP in Cottage Grove, Oregon. It should be noted that the plant requires Amiad filtration at the front of the plant, and a significant level of additional tankage and other equipment, including a large volume of chemical mixing tanks and infrastructure. The equipment is shown without the membrane modules installed.

Figure 7.3.1 – Cottage Grove Pall Membrane WTP

The WMP also states that there would be significant operational and chemical cost saving with the operation of the membrane facility. Depending upon the water quality and age of the membrane facility, and the type of chemical exposure membranes go through during operations, could require significant maintenance. This includes opening membrane modules and plugging failed membrane fibers, and providing additional chemical treatment. The chemicals are expensive and also reduce the life of the membranes. Membranes require that relatively clean water be delivered to them, and do not handle flashy surface water conditions well. There are also significant costs associated with compressed air and blowers with membrane technologies. Although the filtrate from a membrane can be superior, this also depends upon how well the membranes are maintained, including how many fibers have failed. The WMP also states that less operator skill and attention is required. This is certainly not true, as any facility requires skilled operators. In addition, Tri City staff is already familiar with conventional treatment technologies, and would require significant training in order to become comfortable operating a membrane facility. This training is available, yet the time investment is significant. Finally, the WMP states that an additional exterior sedimentation basin would not be required for membrane filtration. Conventional mixed-media filtration is very capable of filtering sludge upset and floc carryover, which occurs at current flows. This would only increase with higher flow, which would be a problem for membrane technology. A minimum requirement would be for additional screens to be installed ahead of the membrane system. These costs are not reflected in the WMP analysis. This alternative as developed in the WMP does not account for a viable development plan, which would cost significantly more than illustrated. A membrane plant would most likely require construction of an entirely separate facility, which would then be switched over to the new facility, once prepared for commissioning.

7.3.2.3 Option C – Immersed Membrane Ultrafiltration

This alternative is considered to be unviable, and will not be discussed in this study, yet the major concerns with this alternative are similar to Section 7.3.2.2, above.

7.3.3 Disinfection

Tri City installed a hypochlorite on-demand disinfectant generation system in 2008, which diminishes the potential of disinfection byproduct risk, enhances disinfection effectiveness and efficiency, and reduces chlorine gas system health and safety risk. Cost and supply chain risks associated with chlorine gas systems are also lowered. The system is capable of delivering 60 pounds per day of equivalent chlorine gas. This method is scalable as the water system is expanded with additional tankage, generation, pumping and other equipment as required.

7.3.4 Water Treatment Facility Recommendations

Viable filtration alternatives that adequately consider the specific circumstances in Tri City must be developed. The options developed in the WMP are inadequate for proper comparison of viable alternatives, and do not provide consideration for actual construction phasing requirements. Viable alternatives must consider operational and demand needs of the community. This dramatically affects the cost estimates and feasibility of the proposed projects developed in the WMP. Redeveloped alternatives would include preliminary engineering cost estimates, and would provide valuable information to help Tri City to make informed decisions. A detailed investigation should be implemented to further consider the most practical and cost-effective alternatives that would meet the treatment goals of Tri City, and the needs of the community.

One New Potential Alternative
One new alternative that could be considered is the phased upgrading of the facility to the type of facility currently being constructed to treat the City of Sutherlin's Cooper Creek Reservoir water rights. The system Sutherlin selected is a Siemen's Trident HS (high solids) packaged conventional mixed-media system. This type of facility is more efficient and effective than conventional mixed-media filtration equipment, and is capable of treating surface water that is impossible to treat with membrane facilities. The system was compared to various membrane technologies and proved to be the most cost-effective and reliable alternative for Sutherlin. The Sutherlin WTP is designed to treat 5 cfs, which is similar to the needs of Tri City (4.87 cfs). This packaged equipment has dual treatment trains with tube settler modules, capable of providing settlement and flocculation capacity prior to entering adsorption

clarification, and mixed-media filtration with media-retention underdrains and air scour system. Electrically-actuated control valves can be utilized rather than pneumatic control valves, which enhances control and reliability of the facility, while also minimizing energy usage associated with pneumatically-actuated valve systems.

A major component to membrane treatment facility costs is the additional cost required for pre-treatment of raw water, prior to the membrane equipment. Tri City's WTP is currently deficient in settling and flocculation capacity. The WMP calls for an expensive improvement to the sedimentation / flocculation basins, yet this new alternative may be designed to negate the need for new basins. This option would utilize the existing sedimentation basin as a pre-settling and pre-flocculation basin, with secondary flocculation and settling occurring in the new treatment equipment.

The existing WTP would continue to operate while the new equipment is constructed. Once commissioned, the existing WTP would be decommissioned. The facility and mechanical piping would be sized to handle growth for the next 10 years, when the existing equipment would be demolished, and an additional Trident treatment train would be installed in the existing building. This alternative would create a viable, simple and scalable water system, while utilizing as much of the existing equipment and infrastructure as possible. It is further recommended that Tri City staff tour the Sutherlin facility in order to gain a greater understanding of what is involved with the construction of a new facility, while continuing to operate an existing water treatment facility.

Summary
The most cost-effective, medium-term solution for the water treatment facility is to expand the capabilities of the existing facility in order to maximize its operational life. This could include the installation of additional mixed-media filters, the addition of new underdrains, electrically actuated valves, filter media, air-scour systems, and the repair of operational and mechanical deficiencies within the existing treatment facility. Additional settling and flocculation capability will be required in order to significantly enhance water treatment facility capacity. Therefore, additional filtration alone will not solve the existing operational and capacity deficiencies in the facility. Improvement of the existing facility could be combined with the alternative mentioned above. However, this functionally equates to the operation of two separate treatment facilities with different water treatment capacities and capabilities, which would result in significant operational difficulties. Finally, expenditures toward the existing facility should be carefully considered, as this capital could be invested toward the future needs of the water treatment system. The future water system will be considerably different than the current facility. Investments made toward the improvement of overall long-term deficiencies could prove to be more prudent than investments toward the enhancement of the existing facility. This topic must be considered in greater detail in order to enable an informed decision.

7.3.5 WTP Backwash Waste Water Reuse

One potential water conservation alternative is the reuse of backwash water, which is discharged from the WTP into the recently improved backwash pond adjacent to the WTP. Water from the primary backwash pond is discharged into a secondary backwash pond prior to the supernatant being discharged back into the South Umpqua River. During peak water demand periods (June through September) the amount of plant total wastewater was recently between 3.5 - 4.0 million gallons, or approximately less than 1 million gallons per month. Total plant waste is well within typical levels for a well-operated facility at approximately 3.5 – 5.5%.

Concerns from operations staff regarding the use of recycled wastewater include the introduction of filter water, with viruses and other microbiological pathogens, as well as accumulated algae from the ponds. The WTP is well operated, and is capable of treating various water qualities, but the introduction of varying recycled wastewater quality could affect the treatment processes in unpredictable ways. This also makes troubleshooting of treatment processes more difficult, as treatment issues are more difficult to narrow by source. Although less than 1 million gallons of wastewater is discharged from the plant per month during the peak demand period, this amount of water would not be available for retreatment. Evaporation and seepage from the backwash basins significantly reduce the amount available.

Plant wastewater recycling would require the installation of a new pumping facility, with an approximate 4-inch diameter system. The EPA Filter Backwash Recycle Rule requires that all plant wastewater must undergo complete treatment through the WTP with introduction at the head of the facility. A control system would be required in order to minimize water quality impacts, and to ensure adequate levels are maintained in the facilities.

Backwash recycling would occur primarily in the dry season from June through October. Assuming that approximately 30 percent is lost through evaporation and seepage, and some water would be needed to maintain operation of the backwash ponds, very little water would be available. Backwash waste would be discharged into the backwash pond, where it would travel around the entire loop to ensure maximum water quality, prior to being pumped back to the facility. This would enable approximately 50% of backwash water to be available for recycling. This equates to approximately 0.4 – 0.6 million gallons of water per month. This is approximately 2.5% of finished water volumes. Additional analysis will be required in order to understand whether the costs and risk associated with the construction of wastewater improvement are justifiable.

7.4 Treated Water Storage

The WMP develops the need for water storage through the study period under the assumption of specific population growth. The current study helps illustrate the need for the projects once the actual system water demand justifies the improvements. In general, the required storage goal is to provide 1.25 times the maximum daily demand (MDD), plus 540,000 gallons for fire reserve storage. Additional information is presented concerning the current status of the water storage system in Tri City, including the development of pre-design criteria for a new water storage tank for the Back Acres pressure zone.

7.4.1 Existing Storage and Projected Storage Need

The current MDD is 1.217 mgd, which would require 2,062,000 gallons of storage. Tri City currently has 2,017,300 gallons of storage, so is at this time only slightly deficient in storage on a system-wide basis. Should Tri City install an additional storage tank to the high-level Back Acres pressure zone between 350,000 – 500,000 gallons, it will no longer be deficient in storage. Although this is true, the Back Acres pressure zone is deficient in storage, including fire reserve. Once the tank is constructed, the storage system will be well within compliance at least through the year 2020, and potentially through the year 2025 (depending upon the tank volume installed). Table 7.4.1 summarizes the projected demands in the water system, the current storage, storage need, and the projected deficiency, assuming no additional tankage is constructed. This can be used as a tool to guide the timeframe and desired impacts of the tank improvements to the Back Acres pressure zone. It should be noted that if a major industrial user connects to the system in the industrial park, this analysis should be revisited in order to ensure that adequate fire flows could be delivered to the new user. This topic is considered in some detail in Tri City's Pruner Road Hotel Impact Study (Civil West, 2008). Additional analysis would be needed to address the requirements surrounding the specific type of development.

Table 7.4.1 – Projected Water Storage Requirements

	Study Projection		Study Projected Storage Requirements		
Year	MMD	MDD	Storage	Need	Deficient
2013	748,799	1,216,798	2,017,300	2,060,998	(43,698)
2015	786,707	1,278,398	2,017,300	2,137,998	(120,698)
2020	890,086	1,446,390	2,017,300	2,347,988	(330,688)
2025	1,007,051	1,636,458	2,017,300	2,585,572	(568,272)
2030	1,139,386	1,851,502	2,017,300	2,854,377	(837,077)

7.4.2 Back Acres Tank Improvements

A major driving component to this study is the need for additional storage in the high-level pressure zone currently serviced by the Back Acres tank. Presently the tank is capable of storing 87,000 gallons of water. The pressure zone is currently deficient approximately 250,000 gallons, and additional storage to accommodate future growth is recommended. A minimum of 350,000 gallons of additional storage is recommended. The WMP states that the most cost-effective alternative is to remove the existing tank, and reconstruct a new tank on the existing site. The following alternatives are presented, including cost estimates for a new tank at each of four potential sites, and a discussion of the positive and negative features of each alternative.

7.4.2.1 Tank Site Alternatives 1A & 1B

The existing Back Acres tank site could potentially be utilized for a new additional water storage tank. This analysis has shown that the high-level pressure zone is deficient by approximately 250,000 gallons. There are two potential new tank site alternatives directly adjacent to the current water tank site; one directly to the south of the existing tank across a natural drainage way; a second directly to the north of the existing tank. Please see Figure 7.4.1, below. The figure shows the alternatives, including GPS survey waypoints. Re-routing of the current access road was also considered from the south.

Site 1 alternatives would utilize the existing access roadway, but a private residence would be significantly impacted by construction, and roadway improvements will be required. The access road would require de-limbing of mature vegetation, and realignment and repaving of the access road. A strong relationship will be required with the property owner. Acquisition of additional land will be required for either alternative. Finally, this alternative does not provide redundancy of water tank supply to the community. Should maintenance be required on the supply line or the tank itself, high-level users would be without service (including fire service). Excellent drainage is available to the site. It should be noted that a geotechnical investigation will be required as part of the design process. If any geotechnical issues are found, one alternative could be favored over another.

Figure 7.4.1 – Existing Tank Site and New Tank Alternatives 1A / 1B

Alternative Advantages:
Least expensive alternatives.
Utilizes existing access roadway and utilities to the site.

Alternative Disadvantages (Risks):
Lack of redundant water supply.
Will not improve fire flow rates to high-level pressure zone.
Will not encourage future housing developments.
Significant impacts to private residence.
Significant access road improvements required.

7.4.2.1 Tank Site Alternative 2

Tank Site Alternative 2 has two potential access road routings, as shown in Figure 7.4.2, below.

Route 2A is along an existing logging road, which would require development (gravel and pavement). The access is in excellent condition, but a steep section exists at the bottom. This routing is advantageous, since the existing 8-inch water system is easily accessed. A blowoff for the existing distribution system is readily accessible, and an existing easement extends Back Acres Lane through the east boundary of existing property boundaries.

Route 2B could also be easily developed, due to existing logging roads. It extends to the same tank site as Route 2A, and is slightly longer than Route 2A. This route can also be readily connected to the existing water distribution system.

It should be noted that the water utility routing could be constructed along one route, while the access road itself is constructed along the other route. This could have some advantage for cost savings. Excellent drainage is available to the site.

Alternative Advantages:
Enhanced fire flows to Westridge, and future development to the north.
Created redundancy of water storage and supply.
Provided capability to maintain water distribution and storage tanks.

Alternative Disadvantages (Risks):
Slightly higher costs.

7.4.2.3 Tank Site Alternative 3

Tank Site Alternative 3 is the longest and most expensive alternative, yet has many of the advantages of Alternative 2, as shown in Figure 7.4.2, below. An existing logging road exists for the first portion of the access to the tank site, but a new access would require development for the remainder. The tank site is also somewhat steeper than that for the other tank sites. Excellent drainage is available to the site.

Alternative Advantages:
Enhanced fire flows to Westridge, and future development to the north.
Created redundancy of water storage and supply.
Provided capability to maintain water distribution and storage tanks.

Alternative Disadvantages (Risks):
Most expensive alternative.

Figure 7.4.2 – New Tank Site Alternatives 2A / 2B / 3

Recommendations

The alternative chosen by the Board should support a long-term strategy for development and service. Table 1 summarizes all considered alternatives, including cost, cost difference, and lists pros and cons for each alternative. Each of these alternatives is feasible. If cost is deemed the most important factor, then Alternative 1 is the clear choice. If enhanced fire flow rates, encouragement of long-term residential development, and supply/maintenance redundancy are desired, then Alternatives 2 and 3 should be considered.

Please note that land acquisition costs are not included in these estimates, as this is impossible to determine at this time (other than the fair market value). Also note that the assumed tank size is 340,000 gallons, per the Water Master Plan (2006). The minimum tank size required is approximately 165,000 gallons, although it is recommended that a larger tank volume be constructed in order to enhance fire flow and service capability.

My professional opinion is to pursue Alternative 2. Two possible routings are possible to the tank site, which provides flexibility during the land acquisition negotiations and geotechnical studies. It might be possible with this alternative to leverage the improved fire flows, and improved service for development owners, toward favorable land acquisition terms. In an ideal situation, the property owner would donate land for the new tank, and consider the land donation tax and development benefits as a fair trade. Alternative 2 provides the best combination of benefits, for only a 16% premium over Alternative 1. Figure 7.4.3 provides a conceptual engineering drawing of the water tank site for Alternative 2, though this conceptual drawing could be applied to any site during actual design.

Figure 7.4.3 – Alternative 2 Conceptual Engineering Drawing

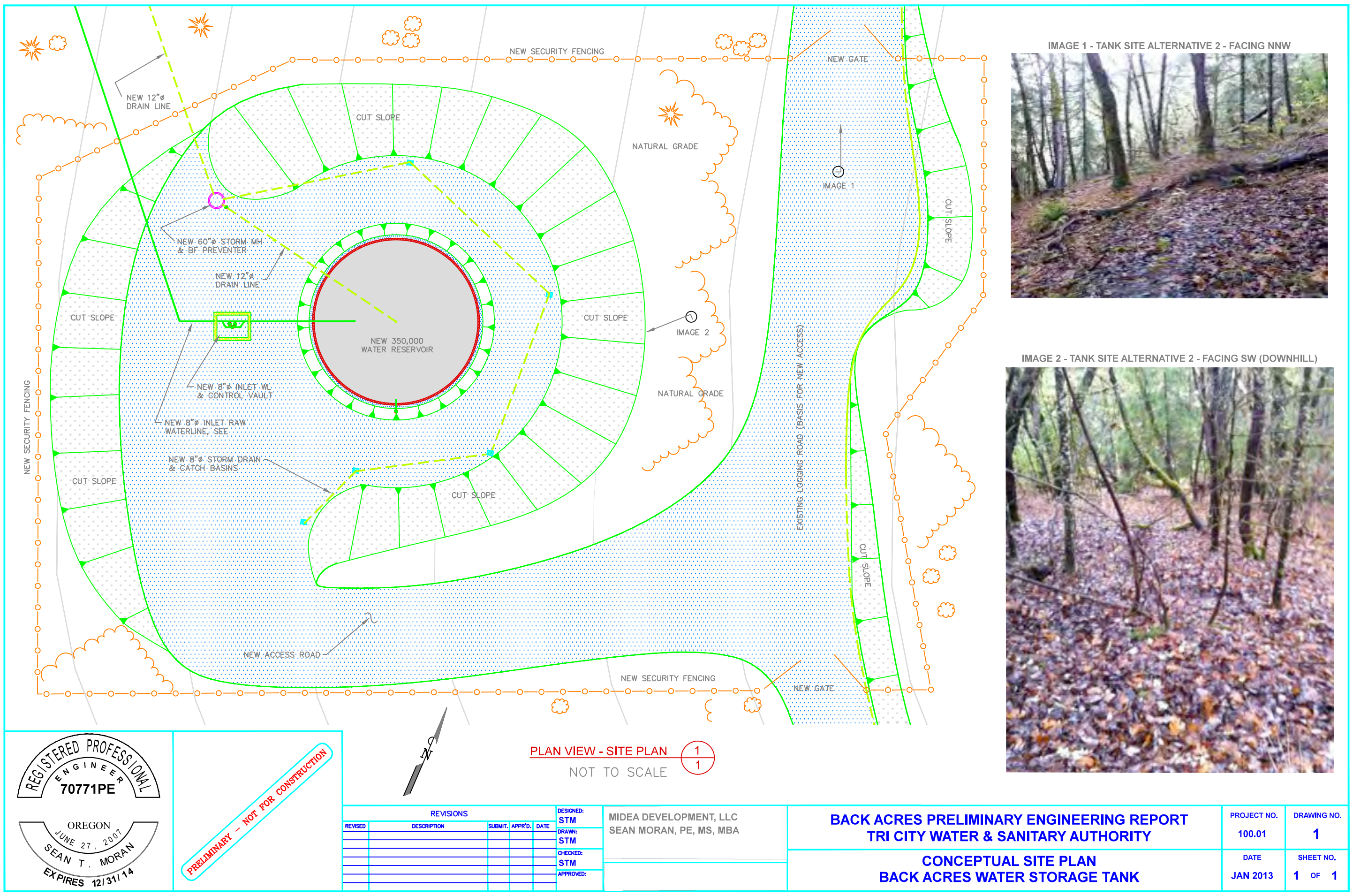


Table 7.4.2 – Back Acres Tank Site Alternatives Summary* **

Back Acres Inline Water Storage Tank Alternatives Summary Table			
Alternative No.	Total Alt. Cost	% Alt 1 Difference	Notes
Alternative 1A	$883,100	-	Least Cost, Residence Impacts, No Fire Flow Increase, No Supply Redundancy, No Growth Incentive.
Alternative 1B	$909,400	+ 3 %	Residence Impacts, No Fire Flow Increase, No Supply Redundancy, No Growth Incentive.
Alternative 2	$1,024,300	+ 16 %	Fire Flow Increase, Supply & Maintenance Redundancy, Growth Incentive.
Alternative 3	$1,110,500	+ 26 %	Fire Flow Increase, Supply & Maintenance Redundancy, Growth Incentive.

* 340,000 gallon water tank has been assumed. Minimum tank size is approx. 165,000 gallons.
** Land acquisition costs are not included in these preliminary engineering estimates.

7.5 Water Distribution Piping System Deficiencies

The condition and deficiencies of the water distribution system have been well developed in the WMP. Tri City should seriously consider which alternatives present the highest risk for interruption of service, and fire flow deficiencies, and address those improvements in a strategic and cost-effective manner.

Section

8

Capital Improvement Plan

The Capital Improvement Plan (CIP) provided in the Water Master Plan (WMP) (HBH, 2006) is adequately developed with the exceptions noted in this study. These exceptions are primarily associated with the filtration system portion of the water treatment plant (WTP), the Back Acres high-level pressure zone, and the water intake and pump station. These exceptions are developed and discussed in Section 7.

This study explicitly developed an updated analysis of water demand and system characteristic, and validated many of the assumptions of previous work, including the WMP, the Water Rate Study (2007), Water System Risk Failure Analysis (HBH, 2011), and other studies. Additional investigation and analysis may be required in order to fully assess and update alternatives to some of the operational issues and deficiencies currently identified in this study.

This study can be utilized as a tool for future planning of projects called for on a milestone basis of water system demand. The current study provides recommendations and guidelines for this application.

References

These References are in chronological order, rather than alphabetic order.

Tri City Joint Water & Sanitary Authority, Water System Master Plan; HBH Consulting Engineers; May 2006.

Tri City Joint Water & Sanitary Authority, Water Rate Study; 2007.

Tri City Joint Water & Sanitary Authority, Pruner Road Hotel Impact Study; Civil West Engineering Services, Inc.; Project No. 2901-001; September, 2008.

Tri City Joint Water & Sanitary Authority, Water Intake Improvements; HBH Consulting Engineers; Project No. 2006-019; August 2009.

Tri City Joint Water & Sanitary Authority, Water System Improvements (Clearwell Baffling Improvements); HBH Consulting Engineers; December 2009.

Tri City Joint Water & Sanitary Authority, Fish Screen Operations & Maintenance Manual; Johnson Screens; 2010.

U.S. Census Bureau; Profile of General Population & Housing Characteristics: 2010; 2010 Demographic Profile Data; Tri City CDP, Oregon.

Tri City Joint Water & Sanitary Authority, Water System Risk Failure Analysis; HBH Consulting Engineers; Project No. 2010-025; February 2011.

Tri City Joint Water & Sanitary Authority, Technical Letter: Water Treatment Facility Assessment, Facility Improvements Recommendations; Dyer Partnership Engineers & Planners, Inc.; April 6, 2011.

Tri City Joint Water & Sanitary Authority, Technical Letter: Industrial Park Development, Utility Capacities & Costs; Midea Development, LLC.; April 16, 2012.

Tri City Joint Water & Sanitary Authority, Raw Water Intake Survey; SHN Consulting Engineers & Geologists, Inc.; September 2012.

Appendix

A

Statutory Requirements

This project is funded in part (50% matching) through a grant awarded by the Oregon Water Resources Department (Grant Agreement No. 0056 13). The primary goal of the study is to provide a preliminary engineering design study for an inline water storage tank within the high-level pressure zone of Tri City Water & Sanitary Authority's (Authority) water system. A requirement of the grant funding includes statutory tasks that must be considered in the study. These tasks are outlined in Table A-1, below. Additional goals include a comprehensive evaluation of the current status of the water system, and the investigation of potential water conservation and efficiency measures that could be practically implemented by the Authority.

Table A1 – Required Statutory Tasks

Statutory Task	Description of Progress (include methodology)*	Description of Proposed analysis (include methodology)*
(a) Analyses of by-pass, optimum peak, flushing and other ecological flows of the affected stream and the impact of the storage project on those flows;	Continue to report water use under OAR 690 Div 85. The use of water is reduced during typical peak flow needs.	Review current water rights on record to understand any issues for priority date.
(b) Comparative analyses of alternative means of supplying water including but not limited to the costs and benefits of conservation and efficiency alternatives and the extent to which long-term water supply needs may be met using those alternatives;		Provide recommendations for water conservation and efficiency measures.
(c) Analyses of environmental harm or impacts from the proposed storage project;	Recently installed new fish screens	Review existing fish screen specifications to understand the current requirements of the Oregon Fish And Wildlife Department
(d) Evaluation of the need for and feasibility of using stored water to augment in-stream flows to conserve, maintain and enhance aquatic life, fish life and any other ecological values; and		The amount of storage on hand will be less than 1% of the natural stream flow in a 24 hour period; flow augmentation could not be accomplished if water is being supplied. The grantee will analyze if there will be any greater flexibility to withdraw water at a reduced rate or less often with increased in line storage.

Analysis and Conservation Strategy

The development strategy for the present study was first to perform a comprehensive analysis of recent water system data, in order to assess the current status of water system characteristics. The results of this analysis were compared and contrasted to a significant body of previous work. A brief summary of previous work is included in Section 6, and is referred to in a multitude of locations throughout this study. The analysis and comparisons laid the groundwork for the development of the following discussion concerning the topic of conservation.

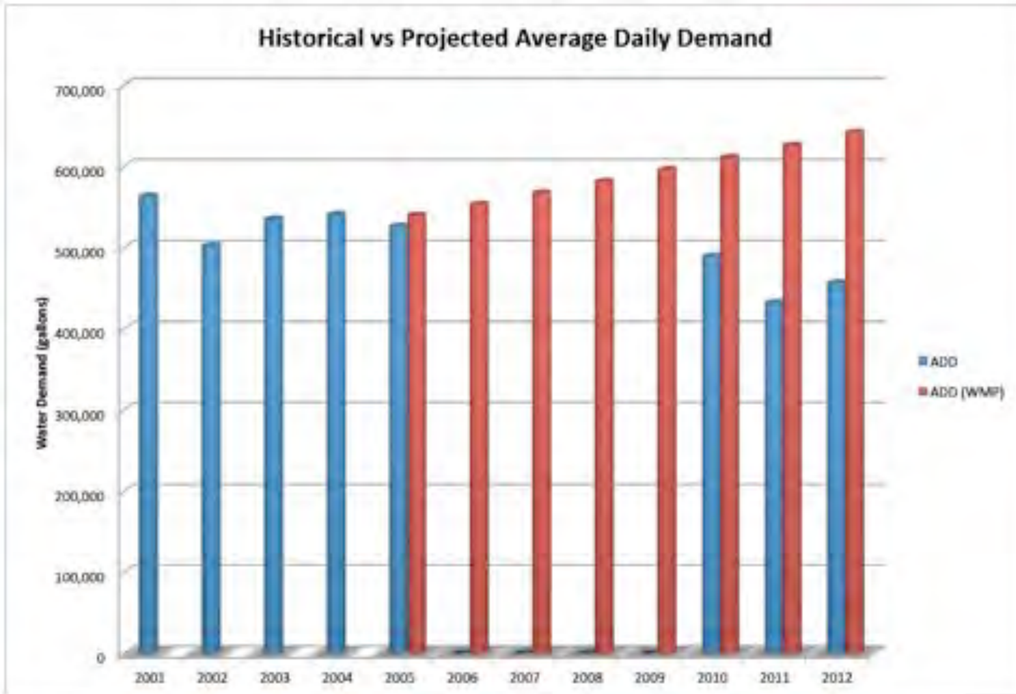
The study consultant presupposed that the WMP framework was acceptable, but that current water system characteristics, and demand projections were very different. The study analysis clearly shows this to be true. Not only are current population characteristics very different than called for in the WMP, but significant conservation has also been encouraged through the implementation of a new water rate structure in 2007. Midea used various sources of data, including the 2010 US Census, previous studies, water treatment system data, and other previous work to determine the current status of the water system.

The current status of water system characteristics creates a unique opportunity to investigate additional means of conservation. This study develops the discussion in a comprehensive fashion. Examples of this include the deferral of major water system improvements, such as the diversion of righted water for a period of time. The Authority can utilize the present study as a tool, or guide, for developing specific water system needs on demand-based milestone criteria. Essentially, as demand reaches certain pre-established criteria, the Authority can act toward the development of needed improvements. Tri City can now plan for these improvements proactively.

The efficient use of energy and water resources maximizes the level of cost-effective service that can be provided to the community, and defers necessary improvements into the future. This study develops a comprehensive list of projects that Tri City has implemented since 2006, and provides recommendations for additional conservation measures. The work presented proves that Tri City has actively pursued conservation projects as a means to minimize costs and impacts on the environment. These projects include the installation of high-efficiency lighting fixtures and pump motor equipment, variable frequency pump drives, a backwash system, and operational improvements. The present study discusses other resource conservation topics such as backwash wastewater recycling.

The analysis developed in this study provides a solid basis for the development of the statutory tasks. An example of this data is illustrated in Figure A1, which shows a compilation of data for the past decade regarding the Authority's water system. Data from 2001 – 2005 was utilized to develop the WMP. A larger version of this figure is presented in Figure ES1 in the Executive Summary. Required population and demand projections can be seen in red in the figure. The new water rate structure implemented in 2007 is based on a purely consumptive model, which typically strongly encourages conservation by consumers. The data presented for the years 2010 – 2012 illustrate the comparison between the WMP and the analysis performed by Midea (only for ADD). It is clear that the current demand characteristics in the service area are very different than those called for in the WMP. This creates a unique opportunity to investigate further conservation alternatives, which could take many forms. This study develops these discussions in a comprehensive fashion. Examples of this include the deferral of major improvements to the water system, including the diversion of righted water for a period of time. The Authority can utilize the present study as a tool, or guide, for developing specific water system needs on demand-based milestone criteria. Essentially, as demand reaches certain pre-established criteria, the Authority can act toward the development of needed improvements.

Figure A1



Discussion of Statutory Tasks

Midea has worked closely with the Authority and Mr. Bill Fujii of the Oregon Department of Water Resources Department (WRD) to thoroughly consider the statutory tasks called for under the grant agreement. Once Midea fully developed the current status of the water system, Midea had a detailed conversation with the WRD on February 22, 2013 to specifically consider the approach concerning the tasks. Midea intentionally deferred this discussion until this time to ensure that the necessary underlying analysis and water tank siting alternatives were sufficiently developed.

The findings of this study will be implemented by the Authority per the recommendations. Some additional study and analysis may need to occur for specific improvements, including improvements to the water treatment facility. The Authority has, and will continue to pursue, the responsible operation and improvement of its facilities. The findings of this study do not create issues concerning impacts to endangered species. Some parts of the project recommended in this study could require their own studies of potential impacts, depending upon the actual scope of those improvements. The Authority plans to pursue funding for the recommended project through an intelligent and thoughtful strategy, while also considering the financial impacts to its community.

Please note that the development of the discussion in statutory task one hold many common elements to other statutory tasks. This will be noted in each discussion, but typically not repeated for the sake of creating an efficient discussion.

Statutory Task a)
Review of previous work, including the WMP, has shown that the analysis of by-pass, optimum peak, flushing, and other ecological flows from permitted water rights is adequately developed. Current water rights are adequate to meet demand well into the near future, including through the study period of the WMP. The Authority has not currently fully developed existing permitted water rights. This study has developed a framework for when additional water rights could be needed. The current water system is incapable of treating the current water rights until significant improvements are completed. These

improvements are well developed in this study, and include water intake, intake pump station, water treatment facility, and other improvements. During low water periods, the Authority is sometimes required to divert water from Galesville reservoir, in order to meet peak demand. This discussion is well developed in the WMP. In addition, a fish friendly intake screen was recently constructed, as discussed in this study.

The proposed water tank in the high-level Back Acres pressure zone is an important improvement for the water system. This improvement will allow for additional operational and diversion flexibility. Operations staff can fill the water tank during off-peak demand periods, which will flatten the diverted water curve (by serving as a demand buffer). Water tank circulation equipment will ensure adequate tank mixing, which will produce excellent water quality. The water tank will end the current water storage deficiency for the water system, allow for improved fire suppression flows and volume, and will enable tank serviceability while continuing water service (from the second tank), since the existing water tank will continue to operate. During a drought period, many water users will be required to implement water conservation measures. The current water tank improvement will enable some operational flexibility that could mitigate potential effects.

It is important to note that the proposed project in this preliminary engineering report is for a high-pressure zone inline water storage tank. The project will NOT divert any additional water from the source stream. No permit will be required for this project to impound additional water.

This study has shown that water demand from the water system is significantly less than projected in the water master plan, significantly in part due to direct conservation-encouraging actions, including the restructuring of water rates.

Finally, should this statutory section be read independently of the study, please note the Executive Summary, and other parts of the study, which discuss the significant efforts the Authority has devoted to conservation, and the responsible diversion of its water rights. The Authority will continue to pursue this high level of care as discussed in this study, which will include such improvements to enhance water treatment facility efficiency.

Statutory Task b)
This study did not develop a discussion concerning means for alternative water supply. Various alternatives have been developed for specific improvement needs, as required, including the new inline storage tank. Alternative water supply includes water from Galesville reservoir as discussed above. Water supply from other communities is entirely cost-prohibitive due to the distance to the adjacent water systems in Riddle or Myrtle Creek. Water conservation measures are discussed above, as well as throughout this study.

Statutory Task c)
This statutory task is concerned with the potential environmental harm, or impacts from the proposed project. The current inline storage project is not anticipated to have any adverse impact. The construction site will be sited in the surrounding hills of the area in order to service the Authority's only high-level pressure zone. This construction will follow all applicable permitting and regulatory requirements, including the execution of applicable erosion control measures, should they be required. Many of the factors discussed in statutory task a, above, also apply here, including the recent construction of a fish friendly intake screen.

Statutory Task d)
It is important to note that this project calls for the development of an inline water storage tank, not a water impoundment or reservoir. Therefore, this statutory task does not apply to this project. This project does not create an opportunity to enhance, or augment, in-stream flows.

Appendix

B

Water Use and Projected Demand Documentation

Water System Characteristics							
				Metered Sales		Water	
Month	Raw Water	Finished	Plant Use	3 Day M.A.	Metered Sales	System Loss	% Loss System
Sep-12	20,071,200	18,424,900	1,421,239		16,547,856		
Aug-12	24,142,700	23,105,889	1,577,129	19,691,097	20,051,657	2,962,552	13.1%
Jul-12	21,577,100	20,032,728	1,468,968	17,962,319	22,473,779	1,950,869	9.8%
Jun-12	16,207,500	14,869,107	815,877	14,583,924	11,361,520	248,413	1.7%
May-12	15,312,400	13,905,612	606,742	9,921,784	9,916,473	3,936,798	28.4%
Apr-12	11,058,700	10,155,183	610,913	8,724,595	8,487,358	1,353,858	13.4%
Mar-12	11,362,300	10,387,582	683,362	7,592,014	7,769,955	2,680,588	26.1%
Feb-12	10,864,600	10,033,184	638,804	7,756,495	6,518,730	2,166,769	21.8%
Jan-12	12,098,600	11,089,451	702,931	8,678,606	8,980,801	2,330,565	21.2%
Dec-11	12,009,300	11,115,611	714,171	9,158,551	10,536,287	1,848,200	16.8%
Nov-11	12,936,200	12,062,047	959,567	9,684,407	7,958,564	2,313,120	19.3%
Oct-11	14,085,200	13,371,686	973,286	11,564,821	10,558,371	1,386,065	10.7%
Sep-11	20,634,700	19,657,520	1,288,850	15,679,804	16,177,527	3,385,786	17.8%
Aug-11	23,279,800	22,339,013	1,478,753	17,111,159	20,303,514	4,572,814	21.1%
Jul-11	20,604,700	19,541,028	833,498	14,791,693	14,852,437	4,603,365	23.7%
Jun-11	15,499,600	15,578,084	855,882	10,755,539	9,219,128	4,632,355	30.1%
May-11	12,298,300	11,854,992	698,412	8,432,620	8,195,052	3,249,152	27.8%
Apr-11	11,368,900	10,935,690	634,220	8,411,842	7,883,680	2,368,418	22.0%
Mar-11	12,412,800	11,585,474	980,364	8,191,574	9,156,793	3,219,110	28.2%
Feb-11	11,208,000	10,481,798	706,138	8,695,201	7,534,249	1,624,757	15.7%
Jan-11	14,280,300	12,814,843	1,614,613	8,109,575	9,394,561	4,360,198	35.0%
Dec-10	12,033,100	11,715,656	917,686	8,200,374	7,399,914	3,129,852	27.6%
Nov-10	12,515,000	11,768,522	1,390,472	8,169,034	7,806,647	3,218,938	28.3%
Oct-10	13,872,900	13,038,431	1,316,781	11,218,139	9,300,541	1,331,842	10.6%
Sep-10	16,951,300	15,966,090	1,743,080	14,711,877	16,547,229	542,323	3.6%
Aug-10	25,285,400	24,441,667	2,020,307	17,820,572	18,287,860	5,539,855	23.7%
Jul-10	25,002,900	24,597,711	1,900,101	15,602,286	18,626,628	8,124,535	34.2%
Jun-10	15,261,200	15,014,733	1,506,013	12,262,761	9,892,371	2,433,492	16.6%
May-10	12,512,100	12,497,789	1,313,079	8,981,363	8,269,284	3,252,136	26.6%
Apr-10	12,201,900	11,650,322	1,845,702	8,488,393	8,782,435	2,904,049	25.5%
Mar-10	14,780,400	14,346,992	1,308,812	8,028,009	8,413,461	6,008,063	42.8%
Feb-10	14,872,500	14,661,160	1,193,800	7,696,537	6,888,130	6,620,783	46.2%
Jan-10	16,194,500	16,260,871	1,488,251	11,360,147	7,788,020	4,605,544	28.8%
Dec-09	17,641,600	17,338,958	1,521,098	11,451,539	19,404,290	5,530,479	32.6%
Nov-09	13,497,500	13,462,280	1,021,810	12,637,030	7,162,306	485,820	3.7%
Oct-09	15,068,600	15,355,609	1,621,659		11,344,494		
	561,003,800	535,458,213	42,372,370	382,125,681	409,791,902	108,921,463	22.2%

DP-1

Profile of General Population and Housing Characteristics: 2010

2010 Demographic Profile Data

NOTE: For more information on confidentiality protection, nonsampling error, and definitions, see http://www.census.gov/prod/cen2010/doc/dpsf.pdf.

Geography: Tri-City CDP, Oregon

Subject	Number	Percent
SEX AND AGE		
Total population	3,931	100.0
Under 5 years	229	5.8
5 to 9 years	225	5.7
10 to 14 years	270	6.9
15 to 19 years	269	6.8
20 to 24 years	190	4.8
25 to 29 years	184	4.7
30 to 34 years	241	6.1
35 to 39 years	223	5.7
40 to 44 years	229	5.8
45 to 49 years	268	6.8
50 to 54 years	304	7.7
55 to 59 years	305	7.8
60 to 64 years	284	7.2
65 to 69 years	200	5.1
70 to 74 years	175	4.5
75 to 79 years	154	3.9
80 to 84 years	122	3.1
85 years and over	59	1.5
Median age (years)	42.8	(X)
16 years and over	3,142	79.9
18 years and over	3,044	77.4
21 years and over	2,892	73.6
62 years and over	882	22.4
65 years and over	710	18.1
Male population	1,980	50.4
Under 5 years	116	3.0
5 to 9 years	118	3.0
10 to 14 years	150	3.8
15 to 19 years	128	3.3
20 to 24 years	104	2.6
25 to 29 years	104	2.6
30 to 34 years	108	2.7
35 to 39 years	116	3.0
40 to 44 years	123	3.1
45 to 49 years	127	3.2
50 to 54 years	154	3.9
55 to 59 years	140	3.6
60 to 64 years	143	3.6
65 to 69 years	94	2.4
70 to 74 years	84	2.1
75 to 79 years	81	2.1
80 to 84 years	60	1.5
85 years and over	30	0.8

Subject	Number	Percent
Median age (years)	41.5	(X)
16 years and over	1,568	39.9
18 years and over	1,524	38.8
21 years and over	1,440	36.6
62 years and over	440	11.2
65 years and over	349	8.9
Female population	1,951	49.6
Under 5 years	113	2.9
5 to 9 years	107	2.7
10 to 14 years	120	3.1
15 to 19 years	141	3.6
20 to 24 years	86	2.2
25 to 29 years	80	2.0
30 to 34 years	133	3.4
35 to 39 years	107	2.7
40 to 44 years	106	2.7
45 to 49 years	141	3.6
50 to 54 years	150	3.8
55 to 59 years	165	4.2
60 to 64 years	141	3.6
65 to 69 years	106	2.7
70 to 74 years	91	2.3
75 to 79 years	73	1.9
80 to 84 years	62	1.6
85 years and over	29	0.7
Median age (years)	44.1	(X)
16 years and over	1,574	40.0
18 years and over	1,520	38.7
21 years and over	1,452	36.9
62 years and over	442	11.2
65 years and over	361	9.2
RACE		
Total population	3,931	100.0
One Race	3,789	96.4
White	3,608	91.8
Black or African American	2	0.1
American Indian and Alaska Native	119	3.0
Asian	24	0.6
Asian Indian	7	0.2
Chinese	2	0.1
Filipino	9	0.2
Japanese	0	0.0
Korean	2	0.1
Vietnamese	0	0.0
Other Asian [1]	4	0.1
Native Hawaiian and Other Pacific Islander	6	0.2
Native Hawaiian	1	0.0
Guamanian or Chamorro	0	0.0
Samoan	0	0.0
Other Pacific Islander [2]	5	0.1
Some Other Race	30	0.8
Two or More Races	142	3.6
White; American Indian and Alaska Native [3]	96	2.4
White; Asian [3]	4	0.1
White; Black or African American [3]	8	0.2
White; Some Other Race [3]	7	0.2
Race alone or in combination with one or more other races: [4]		
White	3,740	95.1
Black or African American	18	0.5
American Indian and Alaska Native	232	5.9

Subject	Number	Percent
Asian	35	0.9
Native Hawaiian and Other Pacific Islander	18	0.5
Some Other Race	44	1.1
HISPANIC OR LATINO		
Total population	3,931	100.0
Hispanic or Latino (of any race)	176	4.5
Mexican	137	3.5
Puerto Rican	7	0.2
Cuban	2	0.1
Other Hispanic or Latino [5]	30	0.8
Not Hispanic or Latino	3,755	95.5
HISPANIC OR LATINO AND RACE		
Total population	3,931	100.0
Hispanic or Latino	176	4.5
White alone	110	2.8
Black or African American alone	0	0.0
American Indian and Alaska Native alone	8	0.2
Asian alone	0	0.0
Native Hawaiian and Other Pacific Islander alone	1	0.0
Some Other Race alone	30	0.8
Two or More Races	27	0.7
Not Hispanic or Latino	3,755	95.5
White alone	3,498	89.0
Black or African American alone	2	0.1
American Indian and Alaska Native alone	111	2.8
Asian alone	24	0.6
Native Hawaiian and Other Pacific Islander alone	5	0.1
Some Other Race alone	0	0.0
Two or More Races	115	2.9
RELATIONSHIP		
Total population	3,931	100.0
In households	3,931	100.0
Householder	1,526	38.8
Spouse [6]	775	19.7
Child	1,013	25.8
Own child under 18 years	711	18.1
Other relatives	288	7.3
Under 18 years	129	3.3
65 years and over	38	1.0
Nonrelatives	329	8.4
Under 18 years	47	1.2
65 years and over	21	0.5
Unmarried partner	166	4.2
In group quarters	0	0.0
Institutionalized population	0	0.0
Male	0	0.0
Female	0	0.0
Noninstitutionalized population	0	0.0
Male	0	0.0
Female	0	0.0
HOUSEHOLDS BY TYPE		
Total households	1,526	100.0
Family households (families) [7]	1,077	70.6
With own children under 18 years	387	25.4
Husband-wife family	775	50.8
With own children under 18 years	220	14.4
Male householder, no wife present	105	6.9
With own children under 18 years	63	4.1
Female householder, no husband present	197	12.9
With own children under 18 years	104	6.8

Subject	Number	Percent
Nonfamily households [7]	449	29.4
Householder living alone	329	21.6
Male	173	11.3
65 years and over	60	3.9
Female	156	10.2
65 years and over	85	5.6
Households with individuals under 18 years	477	31.3
Households with individuals 65 years and over	506	33.2
Average household size	2.58	(X)
Average family size [7]	2.93	(X)
HOUSING OCCUPANCY		
Total housing units	1,633	100.0
Occupied housing units	1,526	93.4
Vacant housing units	107	6.6
For rent	38	2.3
Rented, not occupied	0	0.0
For sale only	25	1.5
Sold, not occupied	2	0.1
For seasonal, recreational, or occasional use	4	0.2
All other vacants	38	2.3
Homeowner vacancy rate (percent) [8]	2.2	(X)
Rental vacancy rate (percent) [9]	8.1	(X)
HOUSING TENURE		
Occupied housing units	1,526	100.0
Owner-occupied housing units	1,096	71.8
Population in owner-occupied housing units	2,721	(X)
Average household size of owner-occupied units	2.48	(X)
Renter-occupied housing units	430	28.2
Population in renter-occupied housing units	1,210	(X)
Average household size of renter-occupied units	2.81	(X)

X Not applicable.

[1] Other Asian alone, or two or more Asian categories.

[2] Other Pacific Islander alone, or two or more Native Hawaiian and Other Pacific Islander categories.

[3] One of the four most commonly reported multiple-race combinations nationwide in Census 2000.

[4] In combination with one or more of the other races listed. The six numbers may add to more than the total population, and the six percentages may add to more than 100 percent because individuals may report more than one race.

[5] This category is composed of people whose origins are from the Dominican Republic, Spain, and Spanish-speaking Central or South American countries. It also includes general origin responses such as "Latino" or "Hispanic."

[6] "Spouse" represents spouse of the householder. It does not reflect all spouses in a household. Responses of "same-sex spouse" were edited during processing to "unmarried partner."

[7] "Family households" consist of a householder and one or more other people related to the householder by birth, marriage, or adoption. They do not include same-sex married couples even if the marriage was performed in a state issuing marriage certificates for same-sex couples. Same-sex couple households are included in the family households category if there is at least one additional person related to the householder by birth or adoption. Same-sex couple households with no relatives of the householder present are tabulated in nonfamily households. "Nonfamily households" consist of people living alone and households which do not have any members related to the householder.

[8] The homeowner vacancy rate is the proportion of the homeowner inventory that is vacant "for sale." It is computed by dividing the total number of vacant units "for sale only" by the sum of owner-occupied units, vacant units that are "for sale only," and vacant units that have been sold but not yet occupied; and then multiplying by 100.

[9] The rental vacancy rate is the proportion of the rental inventory that is vacant "for rent." It is computed by dividing the total number of vacant units "for rent" by the sum of the renter-occupied units, vacant units that are "for rent," and vacant units that have been rented but not yet occupied; and then multiplying by 100.

Source: U.S. Census Bureau, 2010 Census.

Water System Demand Tables							
2003 - 2005 Comparative Analysis							
Comparative Annual	Assume Average Loss Percentage for 2009-10						Appx. Actual Demand
2005						152,574,910	
ADD						**418,013**	526,054
Minimum						7,574,720	
Maximum						24,666,080	
2004						147,000,988	
ADD						**402,742**	402,742
Minimum						7,474,374	
Maximum						20,691,480	
2003						150,471,814	
ADD						**412,252**	412,252
Minimum						7,596,680	
Maximum						24,304,780	

Water System Demand Tables
2010

Month	Customers Billed	Commercial	Churches	Restaurants	Residential	Water Usage	Total Water Amount	Appx. System Loss	Appx. Loss %
Sep-10	1,507	1,478,667	31,110	197,215	14,840,237	16,547,229	$ 37,231	542,323	3.6%
Aug-10	1,514	1,719,106	31,190	200,626	16,336,938	18,287,860	$ 41,148	5,539,855	23.7%
Jul-10	1,501	1,198,589	29,230	165,155	17,233,654	18,626,628	$ 41,910	8,124,535	34.2%
Jun-10	1,502	624,306	24,360	168,555	9,075,150	9,892,371	$ 22,258	2,433,492	16.6%
May-10	1,490	497,918	19,020	153,350	7,598,996	8,269,284	$ 18,606	3,252,136	26.6%
Apr-10	1,481	575,154	26,110	171,320	8,009,851	8,782,435	$ 19,760	2,904,049	25.5%
Mar-10	1,490	551,870	26,580	155,030	7,679,981	8,413,461	$ 18,930	6,008,063	42.8%
Feb-10	1,495	456,500	16,100	128,630	6,286,900	6,888,130	$ 15,498	6,620,783	46.2%
Jan-10	1,492	463,040	17,490	131,700	7,175,790	7,788,020	$ 17,523	4,605,544	28.8%
Dec-09	1,500	646,220	44,930	186,090	18,527,050	19,404,290	$ 43,660	5,530,479	32.6%
Nov-09	1,508	427,580	13,660	157,920	6,563,146	7,162,306	$ 16,115	485,820	3.7%
Oct-09	1,506	974,980	52,950	148,430	10,168,134	11,344,494	$ 25,525	0	0.0%
Sep-09	1,503	1,829,690	37,060	208,750	14,904,500	16,980,000	$ 38,205	0	0.0%
Average	1,499	830,413	28,223	164,630	10,796,674	11,819,940	$ 26,595	3,792,063	25.8%
Minimum	1,481	427,580	13,660	128,630	6,286,900	6,888,130	$ 15,498	0	
Maximum	1,514	1,829,690	52,950	208,750	18,527,050	19,404,290	$ 43,660	8,124,535	
Total Annual	-						**$ 319,138**	**45,504,756**	
Overall Annual Usage		9,964,953	338,680	1,975,556	129,560,090	**141,839,279**			
ADD						388,601			

Water System Demand Tables
2011

Month	Customers Billed	Commercial	Churches	Restaurants	Residential	Water Usage	Total Water Amount	Appx. System Loss	Appx. Loss %
Sep-11	1,490	1,603,510	37,850	188,160	14,348,007	16,177,527	$ 36,399	3,385,786	17.8%
Aug-11	1,502	1,359,990	36,510	239,780	18,667,234	20,303,514	$ 45,683	4,572,814	21.1%
Jul-11	1,498	1,105,550	31,520	174,680	13,540,687	14,852,437	$ 33,418	4,603,365	23.7%
Jun-11	1,504	556,680	18,140	132,710	8,511,598	9,219,128	$ 20,743	4,632,355	30.1%
May-11	1,500	635,400	16,220	131,179	7,412,253	8,195,052	$ 18,439	3,249,152	27.8%
Apr-11	1,502	526,513	16,920	136,141	7,204,106	7,883,680	$ 17,738	2,368,418	22.0%
Mar-11	1,504	659,631	30,230	149,751	8,317,181	9,156,793	$ 20,603	3,219,110	28.2%
Feb-11	1,496	496,694	15,220	118,792	6,903,543	7,534,249	$ 16,952	1,624,757	15.7%
Jan-11	1,490	513,938	19,710	147,580	8,713,333	9,394,561	$ 21,138	4,360,198	35.0%
Dec-10	1,497	437,930	16,230	161,715	6,784,039	7,399,914	$ 16,650	3,129,852	27.6%
Nov-10	1,491	464,741	29,030	148,712	7,164,164	7,806,647	$ 17,565	3,218,938	28.3%
Oct-10	1,496	703,873	19,770	156,453	8,420,445	9,300,541	$ 20,926	1,331,842	10.6%
Average	1,498	755,371	23,946	157,138	9,665,549	10,602,004	$ 23,855	3,308,049	24.0%
Minimum	1,490	437,930	15,220	118,792	6,784,039	7,399,914	$ 16,650	1,331,842	
Maximum	1,504	1,603,510	37,850	239,780	18,667,234	20,303,514	$ 45,683	4,632,355	
Total Annual	-						**$ 286,254**	**39,696,587**	
Overall Annual Usage		9,064,450	287,350	1,885,653	115,986,590	**127,224,043**			
ADD						348,559			

Water System Demand Tables
2012

Month	Customers Billed	Commercial	Churches	Restaurants	Residential	Water Usage	Total Water Amount	Appx. System Loss	Appx. Loss %
Sep-12	1,486	1,457,159	43,164	149,427	14,898,106	16,547,856	$ 37,233	0	0.0%
Aug-12	1,490	1,555,948	44,133	186,890	18,264,686	20,051,657	$ 45,116	2,962,552	13.1%
Jul-12	1,483	1,017,209	32,960	149,557	21,274,053	22,473,779	$ 50,566	1,950,869	9.8%
Jun-12	1,484	864,387	26,195	130,609	10,340,329	11,361,520	$ 25,563	248,413	
May-12	1,486	826,215	22,589	132,697	8,934,972	9,916,473	$ 22,312	3,936,798	28.4%
Apr-12	1,484	585,121	18,219	157,923	7,726,095	8,487,358	$ 19,097	1,353,858	13.4%
Mar-12	1,478	550,725	15,380	137,929	7,065,921	7,769,955	$ 17,482	2,680,588	26.1%
Feb-12	1,483	502,988	13,934	124,849	5,876,959	6,518,730	$ 14,667	2,166,769	21.8%
Jan-12	1,478	501,507	14,185	142,720	8,322,389	8,980,801	$ 20,207	2,330,565	21.2%
Dec-11	1,477	548,910	14,700	135,159	9,837,518	10,536,287	$ 23,707	1,848,200	16.8%
Nov-11	1,487	169,031	16,068	124,072	7,649,393	7,958,564	$ 17,907	2,313,120	19.3%
Oct-11	1,483	1,227,111	25,470	150,880	9,154,910	10,558,371	$ 23,756	1,386,065	10.7%
Average	1,483	817,193	23,916	143,559	10,778,778	11,763,446	$ 26,468	1,931,483	18.1%
Minimum	1,477	169,031	13,934	124,072	5,876,959	6,518,730	$ 14,667	0	
Maximum	1,490	1,555,948	44,133	186,890	21,274,053	22,473,779	$ 50,566	3,936,798	
Total Annual	-						**$ 317,613**	**23,177,797**	
Overall Annual Usage		9,806,311	286,997	1,722,712	129,345,331	**141,161,351**			
ADD						386,743			

Basic Water Rate Analysis - 2012
Water Rates Effective October 1, 2007
Base monthly rate by meter size:

	Current No. Meters	Meter Size	Meter Rate	Monthly Base Revenue	Annual Base Revenue
	1451	5/8 X 3/4"	$ 26	$ 37,726	$ 452,712
	18	1"	$ 65	$ 1,170	$ 14,040
	5	1.5"	$ 130	$ 650	$ 7,800
	14	2"	$ 208	$ 2,912	$ 34,944
	1	6"	$ 390	$ 390	$ 4,680
Total	1489			$ 42,848	$ 514,176
Total 2012 Consumption Charges					$ 317,613
Approximate 2012 Water Reveue					$ 831,789

All water is billed @ $2.25 per 1,000 gallons.

Appendix

C

Analysis and Improvement Alternatives Calculations

THE DYER PARTNERSHIP
ENGINEERS & PLANNERS, INC.

April 6, 2011

Tri City Water & Sanitary Authority
215 North Old Pacific Highway
Myrtle Creek, OR 97457

Attn: Ms. Vicki Howren
General Manager

RE: Water Treatment Facility Assessment
Facility Improvements Recommendations

Dear Vicki:

Please accept the following assessment of various operational and equipment issues at your water treatment facility.

Introduction & Background

Dyer visited the water treatment facility and met with TCWSA Staff on January 18th and March 2nd to observe the facility. The observed issues primarily comprise the backwash / filter cycle and related equipment. Finished water treatment does not appear to be affected by these operational issues. The operational issues include:

1. Columnar surging of backwash water through the filter media at the beginning of the backwash cycle for all filters.
2. Poor filter water level control during backwash cycle. This requires manual adjustment of the filter-to-waste valve for each filter.
3. All 4 filters can no longer be backwashed in sequence. This was possible prior to the installation of new clearwell baffling.

Tri City Staff drained a filter and manually exposed the underdrain system and settled media. Staff observed that there appeared to be no problems with the underdrain or media.

The current backwash equipment consists of a single backwash pump and related piping and appurtenances. Backwash water is supplied from finished water in the clearwell. Pneumatic control valves provide control of the backwash cycle. Flow control is currently accomplished with a manually actuated knife valve immediately downstream of the backwash pump and a manually actuated filter-to-waste valve to the waste piping. A Griswold automatic flow control valve was utilized in the past to control flow. The core elements of the flow control valve were removed in the past to allow increased flow rate during the backwash cycle.

759 WEST CENTRAL AVENUE
SUTHERLIN, OREGON 97479
TELEPHONE: (541) 459-4619
FAX: (541) 459-4679
WEB: www.dyerpart.com

Discussion of Recommendation

The objectives of the improvement recommendations are to improve operational automation, flexibility and performance of the backwash cycle.

We believe that the columnar surging through the filter media at the beginning of the backwash cycle may be caused by poor control of backwash flow rate and the rate at which the on-off valve opens. The surge does not appear to contain air or to affect treatment. This could potentially cause, or contribute to, the columnar surging.

Installation of a variable frequency drive (VFD) to control the backwash pump could help operations in various ways. The VFD would allow operators to ramp up backwash flows, which could mitigate the filter surge. The VFD would also allow for excellent operational flexibility for flow and filter tank level control. The existing equipment in the filter tanks includes level transducers that measure water level. The signal is used to control finished water operations. We believe that this signal can also be used to provide tank level control for the backwash cycle. If a modulating valve is installed in place of the existing filter-to-waste control valve, the level signal from the filter tanks could be relayed to a Programmable Logic Controller (PLC) that would control the fluid level in the filter tanks during the backwash cycle. That same signal can be used for the PLC to control backwash pump output. This provides redundancy in flow control if a modulating valve malfunctions. This could be controlled regardless of the actual flow rate of the backwash pump. The VFD would be set to a flow profile and fluid level in the tank would be controlled by the modulating valve by the PLC. This would eliminate the existing problems created through manual actuation of the filter-to-waste valve during backwash operations. These improvements would create improved levels of operational flexibility. The modulating valve should be the same style and size as the existing filter-to-waste valve and have a new modulating actuator. The actuator could be either electric or pneumatic. The selection of the actual actuator style could be affected by project costs or space constraints in the vicinity of the valve. A simple paddlewheel flow meter should also be installed in the backwash supply pipeline. This would not be used to control the backwash cycle, but would indicate actual flow rates and help to provide operational information during the backwash cycle.

The new clearwell baffling appears to have created hydraulic constraints that no longer allow adequate supply of water to the backwash pump to provide for sequentially backwashing all four filters. Only two filters can be sequentially backwashed before clearwell hydraulic capacity is exhausted. It may be possible to improve backwash efficiency through trial and error with the recommended improvements and allow for sequential backwash of all four filters again. This is yet to be determined. We do not recommend any modifications to the clearwell baffling at this time until all other alternatives have been exhausted. It is further recommended that the backwash flush of the waste piping be ceased at the beginning of each backwash cycle, as it is unnecessary. This will save water and help improve backwashing operations.

Finally, we noted that the water treatment facility backwash cycle may be improved if a new underdrain and air scour system were to be installed on the existing equipment. This would consist of the installation of low pressure air blowers, air piping and media retention underdrain system. A backwash channel baffle would need to be installed to ensure filter media is retained in the filter tanks during the air scour cycle. There is adequate space in the facility to accommodate this improvement. The primary surface wash equipment could be removed as it would no longer be necessary. This improvement would increase the cleaning efficiency of the backwash cycle and reduce the amount of required backwash water.

Recommendation Summary

Dyer recommends a phased approach to improvements to mitigate operational issues and improve operational flexibility at the water treatment facility. These improvements include:

1. Install a VFD on the backwash pump, PLC and new modulating filter-to-waste control valve to control the backwash cycle. The existing level sensors in the filter tanks can provide signals that could be used to control filter tank water level during the backwash cycle. A paddlewheel flow meter should be installed to indicate flow rates and to support operational improvement and monitoring efforts. Terry Nelson of Camtronics is highly recommended for this improvement
2. Remove waste piping backwash flush (currently 30 seconds) from backwash cycle. I believe operational staff has already applied this improvement as a result of the discussions during the March 2nd.
3. Install a new air scour backwash system in the future to increase backwash efficiency and to improve filter cleaning.
4. Reevaluate the need for clearwell baffle modifications after the backwash system has been updated.

If you have any questions, please feel free to call me.

Sincerely,
THE DYER PARTNERSHIP
ENGINEERS & PLANNERS, INC.

Steve Major, PE
Sean Moran, PE, MS, MBA

Back Acres Tank Water Storage Requirement Calculation					
Alternative 1 @ 1,000 gpm				**Buildout**	
Storage Category	**Relevant Info.**	**Flow (gpm)**	**Duration (h)**	**Total (gal)**	
Fire Flow		1,000	2	120,000	
Equalization	0.25 MDD			26,018	
Emergency	MDD			104,073	
Appx. Service EDUs	180				
2012 EDU	2,053				
2012 MDD (gpd)	1,187,120				
2012 MDD / EDU (gpd/	578				
Buildout MDD (gpd)	104,073				
Total Storage Requirement				250,091	
Current Storage (gal)				87,000	
Storage Need (gal)				163,091	
Alternative 2 @ 1,500 gpm				**Buildout**	
Storage Category	**Relevant Info.**	**Flow (gpm)**	**Duration (h)**	**Total (gal)**	
Fire Flow		1,500	2	180,000	
Equalization	0.25 MDD			26,018	
Emergency	MDD			104,073	
Appx. Service EDUs	180				
2012 EDU	2,053				
2012 MDD (gpd)	1,187,120				
2012 MDD / EDU (gpd/	578				
Buildout MDD (gpd)	104,073				
Total Storage Requirement				310,091	
Current Storage (gal)				87,000	
Storage Need (gal)				223,091	

Pipe Loss Calculations						
Back Acres Water Tank Site Alternatives						
Fire Flow Event Asssumed						
	Pipe Flow Calculations					
	Pipe Size (inch dia.)	8	10	12		
	Fire Flow (gpm)	1,500	1,500	1,500		Assumes all flow from new tank.
	Fire Flow (ft3/s)	3.34	3.34	3.34		
	Pipe Area (ft 2)	0.349	0.545	0.785		
	Flow Velocity (ft/s2)	9.6	6.1	4.3		9.6 ft/s flow rate ok
	Pressure Loss (psi)	13.6	4.5	1.8	Per 1000 feet, Assume = old PVC	
	Alt. Total Length (ft)					
1A	150	2.0	0.7	0.3	Total Alt 1A psi loss	
1B	300	4.1	1.4	0.5	Total Alt 1B psi loss	
2	900	12.2	4.1	1.6	Total Alt 2 psi loss	12.2 psi OK for fire flow loss
3	1,500	20.4	6.8	2.7	Total Alt 3 psi loss	20.4 psi not OK, needs to be 10-inch minimum

PRELIMINARY COST ESTIMATE					
Back Acres Water Reservoir - Preliminary Engineering Design Report					
Tri City Water & Sanitary Authority					
Alternative 1A - Immediately North to Existing Tank					
Project No. 100.01					January 6, 2013
No.	Item	Quantity	Unit	Unit Price ($)	Total Price ($)
1	Constr. Facilities & Temp. Controls	1	LS	$75,000	$ 75,000
2	Demolition & Site Preparation	1	LS	$50,000	$ 50,000
3	Foundation Stabilization	200	CY	$30	$ 6,000
4	8-inch Waterline	150	LF	$50	$ 7,500
5	8-inch Gate Valve	6	Each	$800	$ 4,800
6	8-inch Tees	2	Each	$600	$ 1,200
7	Misc. fittings, elbows	1	LS	$5,000	$ 5,000
8	12-inch Gate Valve	1	Each	$1,200	$ 1,200
9	12-inch Drain Line	80	LF	$60	$ 4,800
10	Flap Valves	2	Each	$800	$ 1,600
11	Altitude Valve & Appurtenances	1	Each	$10,000	$ 10,000
12	0.34 MG Reservoir	1	LS	$300,000	$ 300,000
13	Concrete Base	2200	SF	$15	$ 33,000
14	AC Pavement	200	Ton	$110	$ 22,000
15	Aggregate Base	400	Ton	$40	$ 16,000
16	Import Granular Fill	1500	CY	$20	$ 30,000
17	Precast Modular Retaining Wall	500	SF	$30	$ 15,000
18	8' Security Fencing	420	LF	$25	$ 10,500
19	24' Double Swing Gate	1	Each	$1,500	$ 1,500
20	Site & Roadway Excavation	1500	CY	$15	$ 22,500
21	Solar Powered Mixer	1	LS	$15,000	$ 15,000
22	Solar / Battery Alarm SCADA	1	LS	$20,000	$ 20,000
23	Landscaping	1	LS	$6,000	$ 6,000
			Construction		**$ 658,600**
			Contingency		**$ 65,900**
			Engineering		**$ 118,600**
			Geotechnical		**$ 20,000**
			Admin / Legal		**$ 20,000**
			TOTAL ALTERNATIVE COST		**$ 883,100**

PRELIMINARY COST ESTIMATE					
Back Acres Water Reservoir - Preliminary Engineering Design Report					
Tri City Water & Sanitary Authority					
Alternative 1B - South of Existing Tank					
Project No. 100.01					January 6, 2013
No.	Item	Quantity	Unit	Unit Price ($	Total Price ($)
1	Constr. Facilities & Temp. Controls	1	LS	$75,000	$ 75,000
2	Demolition & Site Preparation	1	LS	$50,000	$ 50,000
3	Foundation Stabilization	250	CY	$30	$ 7,500
4	8-inch Waterline	300	LF	$50	$ 15,000
5	8-inch Gate Valve	6	Each	$800	$ 4,800
6	8-inch Tees	2	Each	$600	$ 1,200
7	Misc. fittings, elbows	1	LS	$5,000	$ 5,000
8	12-inch Gate Valve	1	Each	$1,200	$ 1,200
9	12-inch Drain Line	80	LF	$60	$ 4,800
10	Flap Valves	2	Each	$800	$ 1,600
11	Altitude Valve & Appurtenances	1	Each	$10,000	$ 10,000
12	0.34 MG Reservoir	1	LS	$300,000	$ 300,000
13	Concrete Base	2200	SF	$15	$ 33,000
14	AC Pavement & Repair	200	Ton	$110	$ 22,000
15	Aggregate Base	500	Ton	$40	$ 20,000
16	Import Granular Fill	1500	CY	$20	$ 30,000
17	Precast Modular Retaining Wall	500	SF	$30	$ 15,000
18	8' Security Fencing	420	LF	$25	$ 10,500
19	24' Double Swing Gate	1	Each	$1,500	$ 1,500
20	Site & Roadway Excavation & Drainage	2000	CY	$15	$ 30,000
21	Solar Powered Mixer	1	LS	$15,000	$ 15,000
22	Solar / Battery Alarm SCADA	1	LS	$20,000	$ 20,000
23	Landscaping	1	LS	$6,000	$ 6,000
			Construction		**$ 679,100**
			Contingency		**$ 68,000**
			Engineering		**$ 122,300**
			Geotechnical		**$ 20,000**
			Admin / Legal		**$ 20,000**
		TOTAL ALTERNATIVE COST			**$ 909,400**

PRELIMINARY COST ESTIMATE					
Back Acres Water Reservoir - Preliminary Engineering Design Report					
Tri City Water & Sanitary Authority					
Alternative 2 - East of Westridge					
Project No. 100.01					January 6, 2013
No.	Item	Quantity	Unit	Unit Price ($)	Total Price ($)
1	Constr. Facilities & Temp. Controls	1	LS	$75,000	$ 75,000
2	Demolition & Site Preparation	1	LS	$50,000	$ 50,000
3	Foundation Stabilization	600	CY	$30	$ 18,000
4	8-inch Waterline	900	LF	$50	$ 45,000
5	8-inch Gate Valve	7	Each	$800	$ 5,600
6	8-inch Tees	2	Each	$600	$ 1,200
7	Misc. fittings, elbows	1	LS	$10,000	$ 10,000
8	12-inch Gate Valve	1	Each	$1,200	$ 1,200
9	12-inch Drain Line	80	LF	$60	$ 4,800
10	Flap Valves	2	Each	$800	$ 1,600
11	Altitude Valve & Appurtenances	1	Each	$10,000	$ 10,000
12	0.34 MG Reservoir	1	LS	$300,000	$ 300,000
13	Concrete Base	2200	SF	$15	$ 33,000
14	AC Pavement	200	Ton	$110	$ 22,000
15	Aggregate Base	1400	Ton	$40	$ 56,000
16	Import Granular Fill	1500	CY	$20	$ 30,000
17	Precast Modular Retaining Wall	500	SF	$30	$ 15,000
18	8' Security Fencing	420	LF	$25	$ 10,500
19	24' Double Swing Gate	1	Each	$1,500	$ 1,500
20	Site & Roadway Excavation & Drainage	2500	CY	$15	$ 37,500
21	Solar Powered Mixer	1	LS	$15,000	$ 15,000
22	Solar / Battery Alarm SCADA	1	LS	$20,000	$ 20,000
23	Landscaping	1	LS	$6,000	$ 6,000
			Construction		**$ 768,900**
			Contingency		**$ 76,900**
			Engineering		**$ 138,500**
			Geotechnical		**$ 20,000**
			Admin / Legal		**$ 20,000**
			TOTAL ALTERNATIVE COST		**$ 1,024,300**

PRELIMINARY COST ESTIMATE

Back Acres Water Reservoir - Preliminary Engineering Design Report

Tri City Water & Sanitary Authority

Alternative 3 - NE of Westridge

Project No. 100.01 January 6, 2013

No.	Item	Quantity	Unit	Unit Price ($)	Total Price ($)
1	Constr. Facilities & Temp. Controls	1	LS	$75,000	$ 75,000
2	Demolition & Site Preparation	1	LS	$50,000	$ 50,000
3	Foundation Stabilization	750	CY	$30	$ 22,500
4	8-inch Waterline	1500	LF	$50	$ 75,000
5	8-inch Gate Valve	8	Each	$800	$ 6,400
6	8-inch Tees	2	Each	$600	$ 1,200
7	Misc. fittings, elbows	1	LS	$15,000	$ 15,000
8	12-inch Gate Valve	1	Each	$1,200	$ 1,200
9	12-inch Drain Line	80	LF	$60	$ 4,800
10	Flap Valves	2	Each	$800	$ 1,600
11	Altitude Valve & Appurtenances	1	Each	$10,000	$ 10,000
12	0.34 MG Reservoir	1	LS	$300,000	$ 300,000
13	Concrete Base	2200	SF	$15	$ 33,000
14	AC Pavement	200	Ton	$110	$ 22,000
15	Aggregate Base	1700	Ton	$40	$ 68,000
16	Import Granular Fill	1500	CY	$20	$ 30,000
17	Precast Modular Retaining Wall	500	SF	$30	$ 15,000
18	8' Security Fencing	420	LF	$25	$ 10,500
19	24' Double Swing Gate	1	Each	$1,500	$ 1,500
20	Site & Roadway Excavation & Drainage	3500	CY	$15	$ 52,500
21	Solar Powered Mixer	1	LS	$15,000	$ 15,000
22	Solar / Battery Alarm SCADA	1	LS	$20,000	$ 20,000
23	Landscaping	1	LS	$6,000	$ 6,000
			Construction		**$ 836,200**
			Contingency		**$ 83,700**
			Engineering		**$ 150,600**
			Geotechnical		**$ 20,000**
			Admin / Legal		**$ 20,000**
			TOTAL ALTERNATIVE COST		**$ 1,110,500**

Equipment Supplier & Supporting Materials

Appendix

D

Engineering America

TO:	**ALL BIDDING CONTRACTORS**
BID:	**BUDGETARY TANK QUOTE MYRTLE CREEK, OR**
DATE:	**FEBRUARY 25, 2013**

BUDGETARY PROPOSAL

TANK:

ONE (1) GLASS FUSED TO STEEL "COBALT BLUE" *AQUASTORE®* BOLTED STORAGE TANK MANUFACTURED BY CST STORAGE OF DEKALB, ILLINOIS AND DESIGNED PER AWWA D-103 09 DESIGN STANDARDS

TANK ACCESSORIES:

1. EAI TO FURNISH COMPLETE TANK AND CONCRETE FOUNDATION DESIGN SUBMITTAL STAMPED BY THE TANK MANUFACTURER'S REGISTERED STRUCTURAL ENGINEER IN THE STATE OF OREGON.

2. EAI TO PROVIDE ALL NECESSARY MATERIAL, LABOR, AND EQUIPMENT FOR THE PURPOSE OF INSTALLING THE TANK.

3. TANK EXTERIOR COLOR SHALL BE COBALT BLUE, INTERIOR COLOR SHALL BE WHITE.

4. TANK FLOOR SHALL BE CONCRETE. *CONCRETE RINGWALL FOUNDATION AND FLOOR SHALL BE PROVIDED BY OTHERS.*

5. ONE (1) CLEAR SPAN ALUMINUM GEODESIC DOME, ONE (1) 24" GRAVITY VENTILATOR, ONE (1) ACCESS HATCH, ONE (1) STAINLESS STEEL SAFETY CABLE FROM ROOF HATCH TO VENT, 35 PSF SNOW LOAD, AND A 100 MPH WIND RATING.

6. ONE (1) PASSSIVE SACRIFICIAL ANODE CATHODIC PROTECTION SYSTEM TO PROTECT INTERIOR WETTED SURFACES OF TANK.

7. ONE (1) 24" SIDEWALL MANWAY DOOR WITH DAVIT ARM IN OUTER TANK.

8. ONE (1) ALUMINUM LADDER AND GALVANIZED SAFETY CAGE WITH OSHA APPROVED STEP-OFF PLATFORM.

9. ONE (1) 6" CARBON STEEL OVERFLOW PIPE WITH INTERIOR WEIR BOX AND PIPE TERMINATING AT TWENTY-FOUR (24") INCHES ABOVE GRADE.

10. ONE (1) 8" NOZZLE FOR INLET AND ONE (1) 8" NOZZLE FOR OUTLET. NOZZLES SHALL BE LOCATED ON THE EXTERIOR OF THE TANK ONLY.

11. TANK INSTALLED PER ENGINEERING AMERICA, INC FACTORY CERTIFIED AQUASTORE BUILDERS.

647 Hale Ave N
Oakdale, MN 55128
Ph: 651-777-4041
Fax: 651-777-5312
www.engamerica.com

18900 W 158th Street, Suite E
Olathe, KS 66062
Ph: 913-782-7774
Fax: 913-782-7776
www.eai-central.com

6706 N Franklin
Loveland, CO 80538
Ph: 970-962-9400
Fax: 970-962-9412
www.eaiwest.com

300 S Hamilton Place
Gilbert, AZ 85233
Ph: 480-895-1106
Fax: 480-802-7704
www.engamerica.com

12. FREIGHT TO JOBSITE INCLUDED. *OWNER TO PROVIDE FORK LIFT FOR UNLOADING TANK MATERIALS.*

13. OWNER TO STAKE OUT CETNER OF TANK LOCATION.

14. SALES TAX INCLUDED.

EXCLUDED ITEMS:

1. ALL SITE WORK, EXCAVATING, STRUCTURAL FILL BACKFILLING, ETC.
2. ANY CONCRETE FOUNDATION WORK.
3. ALL UNDERGROUND PIPING.
4. ANY SPECIAL INSPECTIONS REQUIRED BY THE OWNER/GENERAL CONTRACTOR.
5. ANY SOILS OR COMPACTION TESTING.
6. ANY UNION OR PREVAILING WAGE LABOR RATES.
7. GEOTECHNICAL REPORT.
8. ANY ELECTRICAL.
9. FILLING/DRAINNING THE TANK WITH WATER.
10. INSULATION.
11. HEATER.
12. PERMITS.
13. OVERTIME PAY.

TANK PRICING:

ONE (1) 39' DIA. BY 29' HIGH GLASS FUSED TO STEEL TANK …………………… $ 230,000.00

Tank Diameter: 39.16', Tank Height: 28.43', Tank Capacity: 256,148 Gallons (0" Freeboard)

ONE (1) 39' DIA. BY 29' HIGH EPOXY COATED TO STEEL TANK ………………… $ 200,800.00

Tank Diameter: 39.16', Tank Height: 28.43', Tank Capacity: 256,148 Gallons (0" Freeboard)

ONE (1) 50' DIA. BY 24' HIGH GLASS FUSED TO STEEL TANK ……….…………… $ 271,000.00

Tank Diameter: 50.35', Tank Height: 23.84', Tank Capacity: 355,168 Gallons (0" Freeboard)

ONE (1) 50' DIA. BY 24' HIGH EPOXY COATED TO STEEL TANK ……..…………… $ 233,500.00

Tank Diameter: 50.35', Tank Height: 23.84', Tank Capacity: 355,168 Gallons (0" Freeboard)

VALIDITY: Price is firm for a period of thirty (30) days. Price is subject to change due to fluctuating iron ore and aluminum price increases. Price can be held upon receipt of letter of intent from owner.

APPROVAL DRAWINGS: Shop drawings on the tank and foundation are completed and stamped by our registered structural engineer in the State of Oregon.

DELIVERY: Delivery of materials is approximately eight (8) weeks after approved shop drawings.

QUALITY: Installation service offered in the Proposal will be performed by trained personnel regularly engaged in the installation of Aquastore storage tanks as manufactured by CST Storage. All work will be performed in an excellent workmanship manner and in accordance with the tolerances and specifications called for by the manufacturer.

BONDING: Payment & Performance bond is not included, but can be provided at an additional charge. Please contact us for the additional charge.

INSURANCE: We will provide a certificate of insurance covering our portion of the work and will meet the requirements of the owner's contract documents.

After you have had an opportunity to review the above proposal and should you have questions, feel free to call me at 503-320-6891.

SINCERELY,

ENGINEERING AMERICA, INC.

TROY CAIRNS
EAI SALES

TO:	**ALL BIDDING CONTRACTORS**
BID:	**BUDGETARY TANK QUOTE** **TRI - CITY, OR**
DATE:	**APRIL 3, 2013**

BUDGETARY PROPOSAL

TANK:

ONE (1) GLASS FUSED TO STEEL "COBALT BLUE" *AQUASTORE®* BOLTED STORAGE TANK MANUFACTURED BY CST STORAGE OF DEKALB, ILLINOIS AND DESIGNED PER AWWA D-103 09 DESIGN STANDARDS

TANK ACCESSORIES:

1. EAI TO FURNISH COMPLETE TANK AND CONCRETE FOUNDATION DESIGN SUBMITTAL STAMPED BY THE TANK MANUFACTURER'S REGISTERED STRUCTURAL ENGINEER IN THE STATE OF OREGON.

2. EAI TO PROVIDE ALL NECESSARY MATERIAL, LABOR, AND EQUIPMENT FOR THE PURPOSE OF INSTALLING THE TANK.

3. TANK EXTERIOR COLOR SHALL BE COBALT BLUE, INTERIOR COLOR SHALL BE WHITE.

4. TANK FLOOR SHALL BE CONCRETE. *EAI SHALL PROVIDE THE CONCRETE, REBAR, AND FORMWORK. EAI ASSUMES SOIL BEARING PRESSURE OF 3,000 PSF (POUNDS PER SQUARE FOOT). FOUNDATION PRICE MAY CHANGE UPON COMPLETION OF SITE SPECIFIC GEOTECHNICAL REPORT. GEOTECHNICAL SOILS REPORT PROVIDED BY OTHERS.*

5. ONE (1) CLEAR SPAN ALUMINUM GEODESIC DOME, ONE (1) 24" GRAVITY VENTILATOR, ONE (1) ACCESS HATCH, ONE (1) STAINLESS STEEL SAFETY CABLE FROM ROOF HATCH TO VENT, 35 PSF SNOW LOAD, AND A 100 MPH WIND RATING.

6. ONE (1) PASSSIVE SACRIFICIAL ANODE CATHODIC PROTECTION SYSTEM TO PROTECT INTERIOR WETTED SURFACES OF TANK.

7. ONE (1) 24" SIDEWALL MANWAY DOOR WITH DAVIT ARM IN OUTER TANK.

8. ONE (1) ALUMINUM LADDER AND GALVANIZED SAFETY CAGE WITH OSHA APPROVED STEP-OFF PLATFORM.

9. ONE (1) 6" CARBON STEEL OVERFLOW PIPE WITH INTERIOR WEIR BOX AND PIPE TERMINATING AT TWENTY-FOUR (24") INCHES ABOVE GRADE.

10. ONE (1) 8" NOZZLE FOR INLET AND ONE (1) 8" NOZZLE FOR OUTLET. NOZZLES SHALL BE LOCATED ON THE EXTERIOR OF THE TANK ONLY.

11. TANK INSTALLED PER ENGINEERING AMERICA, INC FACTORY CERTIFIED AQUASTORE BUILDERS.

12. FREIGHT TO JOBSITE INCLUDED. *OWNER TO PROVIDE FORK LIFT FOR UNLOADING TANK MATERIALS.*

13. OWNER TO STAKE OUT CETNER OF TANK LOCATION.

EXCLUDED ITEMS:
1. ALL SITE WORK, EXCAVATING, STRUCTURAL FILL BACKFILLING, ETC.
2. ALL UNDERGROUND PIPING.
3. ANY SPECIAL INSPECTIONS REQUIRED BY THE OWNER/GENERAL CONTRACTOR.
4. ANY SOILS OR COMPACTION TESTING.
5. ANY UNION OR PREVAILING WAGE LABOR RATES.
6. GEOTECHNICAL SOILS REPORT.
7. ANY ELECTRICAL.
8. FILLING/DRAINNING THE TANK WITH WATER.
9. INSULATION.
10. HEATER.
11. PERMITS.
12. OVERTIME PAY.
13. SALES TAX.

TANK PRICING:

ONE (1) 59' DIA. BY 24' HIGH GLASS FUSED TO STEEL TANK ……….…………… $ 415,210.00

Tank Diameter: 58.75', Tank Height: 23.84', Tank Capacity: 483,422 Gallons (0" Freeboard)

ONE (1) 59' DIA. BY 24' HIGH EPOXY COATED TO STEEL TANK ……….………… $ 369,425.00

Tank Diameter: 58.75', Tank Height: 23.84', Tank Capacity: 483,422 Gallons (0" Freeboard)

VALIDITY: Price is firm for a period of thirty (30) days. Price is subject to change due to fluctuating iron ore and aluminum price increases. Price can be held upon receipt of letter of intent from owner.

APPROVAL DRAWINGS: Shop drawings on the tank and foundation are completed and stamped by our registered structural engineer in the State of Oregon.

DELIVERY: Delivery of materials is approximately eight (8) weeks after approved shop drawings.

QUALITY: Installation service offered in the Proposal will be performed by trained personnel regularly engaged in the installation of Aquastore storage tanks as manufactured by CST Storage. All work will be performed in an excellent workmanship manner and in accordance with the tolerances and specifications called for by the manufacturer.

BONDING: Payment & Performance bond is not included, but can be provided at an additional charge. Please contact us for the additional charge.

INSURANCE: We will provide a certificate of insurance covering our portion of the work and will meet the requirements of the owner's contract documents.

After you have had an opportunity to review the above proposal and should you have questions, feel free to call me at 503-320-6891.

SINCERELY,

ENGINEERING AMERICA, INC.

TROY CAIRNS
EAI SALES

647 Hale Ave N
Oakdale, MN 55128
Ph: 651-777-4041
Fax: 651-777-5312
www.engamerica.com

18900 W 158th Street, Suite E
Olathe, KS 66062
Ph: 913-782-7774
Fax: 913-782-7776
www.eai-central.com

6706 N Franklin
Loveland, CO 80538
Ph: 970-962-9400
Fax: 970-962-9412
www.eaiwest.com

300 S Hamilton Place
Gilbert, AZ 85233
Ph: 480-895-1106
Fax: 480-802-7704
www.engamerica.com

GridBee™
www.gridbee.com

Medora Corporation
3225 Highway 22 • Dickinson, ND 58601
Tel: (701) 225-4495 • www.MedoraCo.com

SolarBee®
www.solarbee.com

Potable Water Quotation

Date: 04/03/2013

Re: Existing Back Acres Water Storage Tank and New Back Acres Water Storage Tank

Project #: 5342

To: Sean Moran, PE
Midea Development, LLC
lordofkarma@hotmail.com • 541-404-3729

From: Jim Joyce, PumpTech Inc., Medora Corporation local representative, Bellevue WA
jjoyce@pumptechnw.com • 425-644-8501

Harvey Hibl, Medora Corporation West U.S. Manager, Westminster CO
harvey.hibl@medoraco.com • 303-469-4001

Amy Dinius, Medora Corporation Sales Engineering Dept., Dickinson ND
amy.dinius@medoraco.com • 866-437-8076

Dear Sean,

Thank you for requesting this quotation. We are very pleased to work with your City to provide high quality potable water circulation equipment at an economical price. This project fits our capabilities well, and we will do everything possible to ensure your project flows smoothly and meets your goals and expectations. Please contact me or any of the contacts mentioned above with any questions. Thank you, Amy

PROJECT DESCRIPTION

1. Tank Names & Location

Existing Back Acres Water Storage Tank and New Back Acres Water Storage Tank are located in Myrtle Creek, OR.

2. Tank Descriptions

Existing Back Acres Water Storage Tank description: Welded-steel, ground storage tank, volume given 500,000 gallons however volume rated at 107,000 gallons, height 16 feet, diameter 33.75 feet, flow rate 73,000 gallons per day, max fill rate 160 gallons per minute.

New Back Acres Water Storage Tank description: New tank in the design phase, glass fused to steel, ground storage tank, volume given 355,000 gallons however volume rated at 425,700 gallons, height 29 feet, diameter 50 feet, flow rate 73,000 gallons per day, max fill rate 160 gallons per minute.

3. Project Objectives

The objective is to provide thorough mixing of the tanks to reduce water age, stagnation, stratification, short circuiting, and cold-climate ice buildup. Thorough mixing not only improves water quality, it also allows for representative sampling of the tank water, and disinfectant boosting if ever needed.

4. Medora Co. Recommendation/System Design for this Installation

To meet the above objectives, we recommend the installation of two (2) SB500PWc v18 machines, placing one unit in each tank.

Note: The minimum hatch size for this installation is 18" diameter with unobstructed clearance.

Performance Guaranty: This mixer will completely mix the subject tank. In continuous operation, (1) at least once per 24 hours all water temperatures within the tank shall converge to within 0.8 degrees C, and (2) at least once per 72 hours all chlorine concentrations within the tank shall converge to within 0.18 mg/l.

5. Equipment Description

SB500PWc v18: 500 gpm (0.72 MGD) total flow, 316-stainless steel and non-corrosion polymer construction, 25-year life high-efficiency brushless electric motor designed to provide day and night operation with a solar-charged battery power system, digital control system for intelligent power management specific to this application, six parameter SCADA outputs, two (2) 80-watt solar panels and control box mounted on a 316SS pedestal, 6" diameter fluid intake hose, and fluid intake injection assembly (injection hose from the intake to the top of the tank). NOTE: (A) This machine is a special collapsible unit; (B) There is minimal impact from mounting PV panels and control box (typically only one penetration), and the integrity of the tank's coating is maintained; (C) This model can be installed through a hatch with 18" diameter minimum unobstructed clearance; (D) See General Provisions - Medora Corporation's Limited Replacement Warranty for information on the most extensive warranty in the industry. Operating footprint: 96 inches diameter. Shipping crate size: 72 inches length x 48 inches width x 59 inches height. Shipping weight: 435 lbs.

6. Equipment Cost

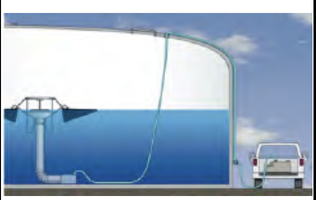
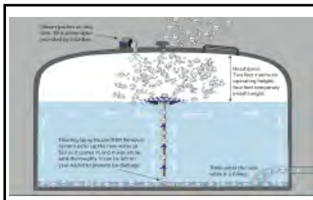
Solar Machine

Quantity	Equipment Description	Purchase Cost Each	Purchase Cost Total
2	SB500PWc v18 Machines. FOB Medora factory in Dickinson, ND	$20,440	$40,880
		Total for Equipment:	$40,880

This price expires in **90 days** per the terms below. For budgeting purposes, please add **10%** for inflation for orders after that time. Factory price adjustments due to inflation are usually required once or twice per year.

Installation of this equipment is not complicated, and well within the scope of work most cities or contractors can perform. An installation manual is provided with all machines. Placement right under a hatch is usually fine, however if this a chloramine system with nitrification problems, then the unit should be placed at or near the lowest part of the tank floor. If a "wet install" is desired, in- tank boats or divers can be used, or see the optional factory delivery and placement in the next section.

7. Machine Options

Options for the Solar-Powered Models		
 Factory Delivery and Placement	Includes delivery and placement of the above equipment into the tank, and supplying any crane or lifting assistance that may be needed. It also includes bringing the electric cord from the mixer to the outside of the tank via a through-wall fitting Medora will supply and install through a tank wall, roof, or vertical side of a raised hatch. **Note:** a multiple unit delivery discount has been applied, both units must be installed during the same Factory Crew site visit for the discount to apply.	Valid for this project only For one unit $10,355, for two units $9,220 ea = $18,440
SCADA	*All v18 models come standard with a SCADA brain-board with six outputs*. For on-site communication options, please contact our SCADA Engineering Department.	Please request option list and pricing.
LED RPM Indicator	*Recommended when SCADA is not available.* An electronic pulsing monitor is added to the digital controller and a flashing green LED beacon is located outside of the tank. The LED indicates the SolarBee impeller rotational speed, and the beacon can be directionally targeted for ground level viewing.	$950 per mixer.
Beekeeper Service Program	The Beekeeper is a program that utilizes Factory Crews to service and maintain proprietary designed equipment. The Beekeeper provides for more than just maintenance and service: • It extends the warranty during the term of the Beekeeper • It covers damage from Acts of God and vandalism • It provides for power system upgrades and updates • It provides hardware, firmware, and software for computer upgrades • It provides scientific and technical support • It provides for scheduled and unscheduled field service calls • and more, please request the Beekeeper brochure for more details	Call for Pricing.
 Portable Disinfectant Boost System	*Consider when occasional on-site boosting is desired.* Portable Disinfectant Boost System (designed to be installed in the back of a pickup), safe, durable chemical transfer system to boost disinfectant in potable water reservoirs. Boosting rate up to 4 gpm, one system can treat multiple tanks, approximate dimensions: 20" W x 52" L x 20" H. Air compressor (4 cfm @ 60 psi) is required to operate the air-powered diaphragm pump; air compressor not included. Brochure available upon request.	$6,300
 THM Removal System	Effective and economical spray nozzle system that works in conjunction with a GridBee / SolarBee mixer to strip TTHM from potable water storage tanks and clearwells. For more information on the THM removal system, please contact us or visit MedoraCo.com.	Call for pricing.

8. General Provisions

A. Equipment Purchase, Not a Construction Project: This equipment is portable, and can be easily relocated or removed entirely from the premises at any time. It does not become an integral part of any building or other structure, or part of "real estate." Therefore, to purchase it, the City should use the same procedure as for purchasing other portable equipment, such as a forklift, a drill press, or an office desk. Medora reserves the right not to accept an order if the purchase is incorrectly characterized as a "construction" project." Medora has not found any State or other jurisdiction where construction or contractor statutes apply to portable equipment that is sold by a factory, with on-site final assembly and placement performed by factory personnel.

B. Assumptions: This quotation may be based on worksheets, calculations or other information that has been provided by the City. The City should bring to Medora's attention any discrepancies, errors in data, or false assumption that Medora may have made while preparing this quotation.

C. Expiration: This quotation expires in 90 days, or on the date of any new quotation for this project, whichever is sooner.

D. Delivery Time: Delivery time must fit the Medora crew schedule, and usually is 60-150 days.

E. Payment Terms: For a federal, state, or local government purchaser with a good credit rating, full payment is due in US dollars 30 days after invoice date, which is generally the date when the goods leave the Medora factory. For a non-government purchaser, full payment must be made by credit card or cashier's check before the goods leave the Medora factory though, in some cases, based on availability of a payment bonding or a bank Letter of Credit, 30 day credit terms may be extended upon special request by the purchaser. If there are any issues with these payment terms, please do not rely on this quotation until the issues have been resolved with Medora.

F. Add for Taxes and Any Governmental Fees: Except as indicated above, no taxes, tariffs or other governmental fees are included in the quote shown above, nor are there any costs added for special insurance coverage the customer may require. It is the customer's responsibility to pay all local, state, and federal taxes, including, sales and use taxes, business privilege taxes, and fees of all types relating to this sale, whether they are imposed on either Medora or the customer, or whether these taxes and fees are learned about after the customer orders the equipment. The customer's purchase order should indicate any taxes or fees due on equipment and/or services, and whether the customer will pay them directly to the governing body or include the tax payment with the purchase for Medora to submit them to the governing body.

G. Add for Special Insurance Requirements: Medora Corporation maintains adequate liability and workman's compensation insurance to generally comply with its requirements for doing business in all fifty U.S. states, and will provide at no charge certificates of insurance when requested. However, if additional insurance or endorsements beyond the company's standard policy are required by the customer, then the costs of those additional provisions and/or endorsements will be invoiced to the customer after the costs become known.

H. Add for Special Training, Safety, Signage, or Other Requirements: Medora has a very strong safety training program for its employees. If any special training classes for Medora personnel are required by the customer, please notify Medora well in advance. The cost of this training will be added to this quotation or invoiced to the customer separately. The same applies to any other special requirements the customer may have, including providing of project signage or any other requirement.

I. Safe and Accessible Tank Condition Required. This quotation is based on the best information made available to us by the above date. If this equipment is ordered, Medora's engineering and installation team will need detail information and photographs to plan the installation. If the detail information changes the installation scope significantly, Medora reserves the right to withdraw or alter this quotation, even if the equipment has already been ordered. To avoid surprises, the City should supply detailed tank information and photos as soon as possible. To ensure the safety of Medora's installation crews, it is the City's responsibility to make sure that all antennas (radio, cell phone, other) located at or near the tank site are inactivated during the installation and/or service of this equipment.

J. Customer to Follow Medora's Maintenance and Safety Guidelines: The customer agrees to follow proper maintenance, operating, and safety instructions regarding the equipment as contained in the safety manual that accompanies the equipment or is sent to the customer's address.

K. Regulatory Compliance. The customer must comply with all applicable Federal and State governmental regulations. It is the customer's sole responsibility to inquire about governmental regulations and ensure that GridBee and SolarBee equipment is deployed and maintained so as to remain in compliance with these regulations and guidelines, and to hold Medora harmless from any liability caused by non-compliance with these regulations and guidelines.

L. Medora Corporation's Limited Replacement Warranty:

> **All new and factory-refurbished SolarBee™ equipment** is warranted to be free of defective parts, materials, and workmanship for a period of 2 years from the date of installation. In addition, the SolarBee brushless motor is warranted for a period of 10 years from the date of installation. Photovoltaic modules (solar panels) carry manufacturer warranties, some ranging up to 25 years (see manufacturer's warranty for details). This warranty is valid only for SolarBee equipment used in accordance with the owner's manual, and consistent with any initial and ongoing factory recommendations. This warranty is limited to the repair or replacement of defective components, at Medora's discretion. The first 2 years of warranty include parts and onsite labor if SolarBee delivery and installation was purchased. Parts and in-factory service are included if the equipment was self-installed. In lieu of sending a factory service crew to the site for minor repairs, Medora may choose to send the replacement parts to the owner postage-paid and, in some cases, may pay the owner a reasonable labor allowance to install the parts.
>
> **Except as stated above, Medora and its affiliates expressly disclaim any and all express or implied conditions, representations and warranties,** on products furnished hereunder, including without limitation all implied warranties of merchantability or fitness for a particular purpose. Please consult your state law regarding this warranty as certain states may have legal provisions affecting the scope of this warranty.

M. Other Limitation of Liability. Many of the employees at Medora Corporation have extensive scientific and practical knowledge relating to solving water quality problems. From time to time, they may offer solicited or unsolicited advice, ideas, judgment or opinions on how to deal with certain situations, none of which offers a guarantee of future events. Due to the many factors, complexity and uncertainty involved in solving water problems, the City agrees to release Medora Corporation and its affiliates, employees and agents from and against any and all claims, liabilities, costs and expenses which the City may incur or become subject to related to or arising out of any services or products furnished by Medora Corporation to the City, except to the extent that any claim, liability or expense results from the gross negligence or intentional misconduct of Medora as determined in a final judgment by a court of competent jurisdiction. In no event will Medora Corporation or its affiliates be liable for any damages caused by failure of buyer to perform buyer's responsibilities or for failure to follow Medora Corporation's advice. In no event will Medora Corporation or its affiliates be liable for any lost profits or use or other punitive, special, exemplary, consequential, incidental or indirect damages, however caused, on any theory of liability, whether or not Medora Corporation has been advised of such damages, or reasonably could have foreseen the possibility of such damages, or for any claim against buyer by another party.

N. Medora Corporation's Lease Provisions:

Standard Agreement: Pricing in the above quotation is based on 5 years, 60 monthly payments, and a $0 down payment. For a quotation based on other terms, please call Medora Corporation, at 1-866-437-8076.

Non-Appropriation Provision: Lessee's (borrower's) payment obligation will terminate if the lessee fails to appropriate in future budgets the funds needed to make the lease payments. Because of this non-appropriation provision, neither the lease nor the lease payments are considered debt, and payments can be made from the savings in your operating budget.

Maintenance of the Equipment: Lessee is to provide minor routine care and maintenance of the Equipment as described in the owners manual. The Beekeeper Service Program is required, and is included in the cost shown above for the term of the lease. See above for description of the Beekeeper program.

Additional Lease Provisions: If the lease option is selected, a master equipment lease/purchase agreement will be sent to lessee, that shall cover all terms and conditions of the lease.

9. To Accept This Quotation

To order the equipment, please issue a purchase order to Medora Corporation, 3225 Hwy. 22, Dickinson, ND 58601. The purchase order can be mailed to the address above, faxed to 866-662-5052, or emailed to the home office at orderprocessing@medoraco.com. The purchase order should refer to the date of this quotation, and will be assumed to include this entire quotation by reference.

If purchase orders are not utilized, please sign and date below, provide billing information, and fax to 866-662-5052 or email to orderprocessing@medoraco.com.

Signing below acknowledges acceptance of this quotation. Please indicate which one of the following options you have chosen.

☐ Solar Machine Purchase

☐ Machine Option___________________

Signature	Date
Printed Name	Title

GridBee™ www.gridbee.com

Medora Corporation
3225 Highway 22 • Dickinson, ND 58601
Tel: (701) 225-4495 • www.MedoraCo.com

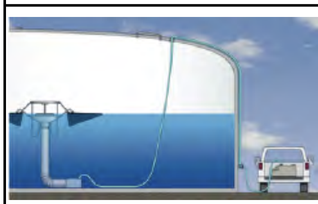
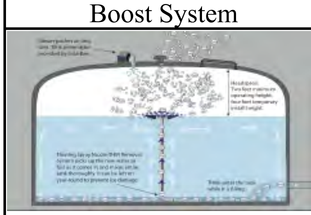
SolarBee® www.solarbee.com

Budget Estimate for GS-12-120v Potable Tank Mixer - Last updated: Feb 1, 2013

Equipment Description	Purchase Cost Each
GS-12-120v mixer with 75 feet of submersible cable. FOB Factory (estimated UPS $200). Note: Please verify price before ordering.	**$7,800**

The GS-12 mixer performance is guaranteed. It is the most effective and competitive mixer on the market. Full specifications are available at *www.MedoraCo.com*. Upon request, independent Computational Fluid Dynamics (CFD) modeling can be provided supporting an upward flow rate of 38,000 gpm in a 5 MG tank of dimensions 145 ft diameter x 40.5 ft tall, with water of 1.0000 specific gravity.

Installing this equipment is well within the scope of work that most cities and contractors can perform. An installation manual is provided with every machine. Placement under the hatch is usually fine; however, if this a chloramine system with nitrification problems, then the unit should be placed at or near the lowest part of the tank floor.

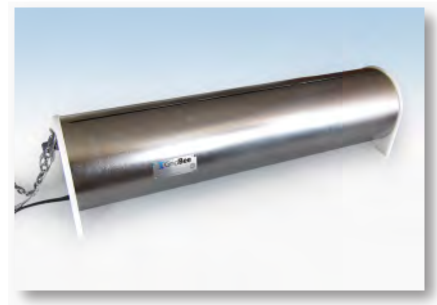
Optional Items		
 GS-12 Control Box with SCADA Monitoring	Control Box: NEMA 4X enclosure, UL listed, HOA switch, contactor for mixer control, 15-Amp GFCI, run indicator light, grounding lug, 120 VAC male molded plug, and locking latch for security. SCADA: 4-20 mAmp current transducer provides analog output for motor current which allows for monitoring proper operation, and a 24 VDC relay for remote on / off control of the mixer.	$950 Shipped with mixer for electrical contractor installation
Chemical Injection Line Kit	75 ft long x 1/2" ID injection hose kit connects to fitting on the intake of mixer and to top of tank; kit is shipped loose with mixer for customer / contractor installation.	$300
GS-12 Extended Warranty	Optional 7-year factory warranty, in lieu of standard 2-year warranty. This is a full-replacement warranty on the mixer, control box, and wiring.	$1,200
 Portable Disinfectant Boost System	*Consider when occasional on-site boosting is desired.* Portable Disinfectant Boost System (designed to be installed in the back of a pickup) is a safe, durable chemical transfer system to boost disinfectant in potable water reservoirs. Approximate dimensions: 20" W x 52" L x 20" H. With boosting rates up to 4 gpm, one system can treat multiple tanks. An air compressor (4 cfm @ 60 psi) is required to operate the air-powered diaphragm pump; electric or engine drven air compressor not included.	$6,300 FOB Factory
 THM Removal System	Proven and economical Spray Nozzle THM Removal Systems that work in conjunction with a GridBee / SolarBee mixer to strip TTHM from potable water storage tanks and clearwells. For more information on the THM removal systems, please contact us or visit *www.MedoraCo.com.*	Call for pricing.
 Factory Delivery and Placement	Includes delivery and placement of the mixer into the tank by a factory crew that is trained to work at heights, over water and in confined space with special equipment designed for this type of installation. Includes bringing the submersible electric cable from the mixer to the outside of the tank via a through fitting, which typically is installed through the roof or the vertical side of a raised hatch, cable is terminated in a junction box at the top of the tank.	$3,000 to $15,000 Varies with tank height and tank construction

GridBee GS-12 Submersible Mixer

GridBee GS-12 submersible mixer thoroughly mixes potable water storage tanks of a wide size range - especially the bottom three feet, the most critical part of the tank. Economical to purchase and operate, the grid-powered GS-12 is easily installed by lowering through any 12" or larger tank hatch. There is no need to enter or drain the tank.

Features & benefits of the GS-12 submersible mixer:

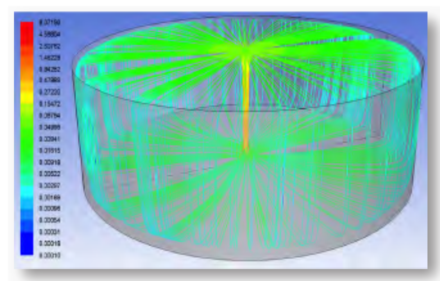
- Eliminate ice damage to tanks in cold climates
- Provide uniform water age and disinfectant distribution
- Prevent stagnation, thermal stratification, and short-circuiting
- Reduce nitrification in chloraminated systems
- Use less disinfectant and produce fewer disinfection byproducts
- Impact the tank boundry layers where bacteria builds up



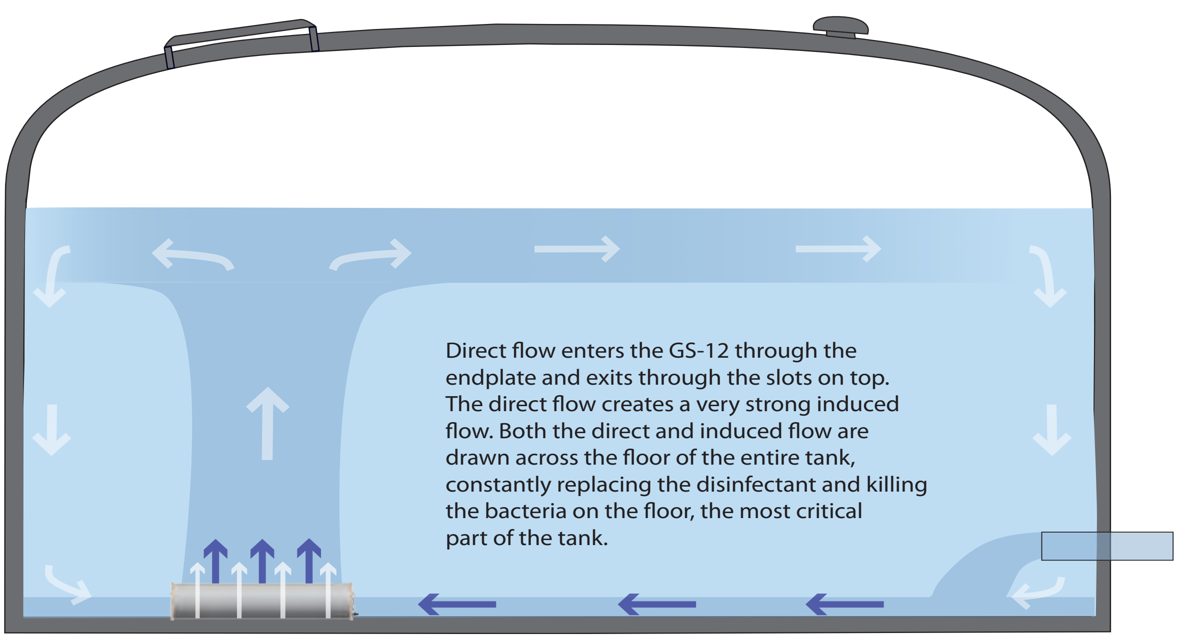
- Low power consumption
- Low-impact feet and endcaps will not damage interior tank coatings
- Certified to NSF / ANSI Std 61-G
- Optional NEMA 4X control box with SCADA monitoring
- Compatible with disinfectant boost and THM removal systems

Specifications:

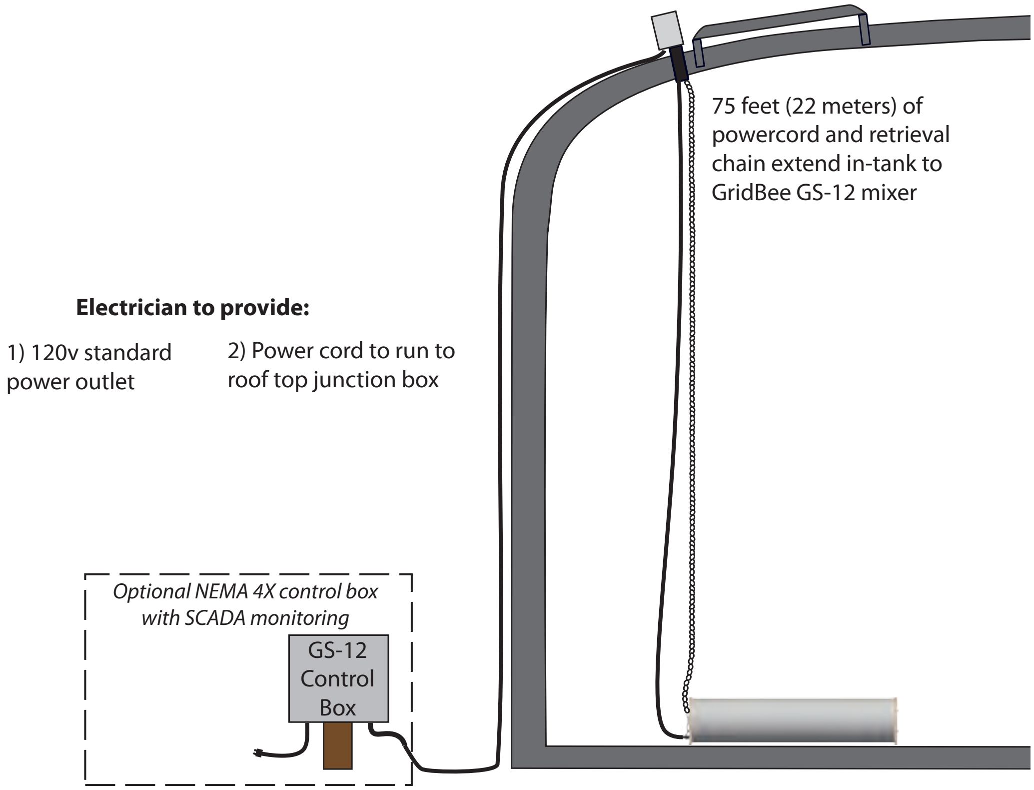
- Dimensions: 36 inches (92 cm) long, 11.5 inches (30 cm) diameter
- Weight: 70 pounds (32 kg); shipping weight 110 pounds (50 kg)
- 120v option draws 10 amps, requires 15 amp delay fuse or 20 amp standard fuse (48v and 240v options available)
- 2-year parts and labor warranty



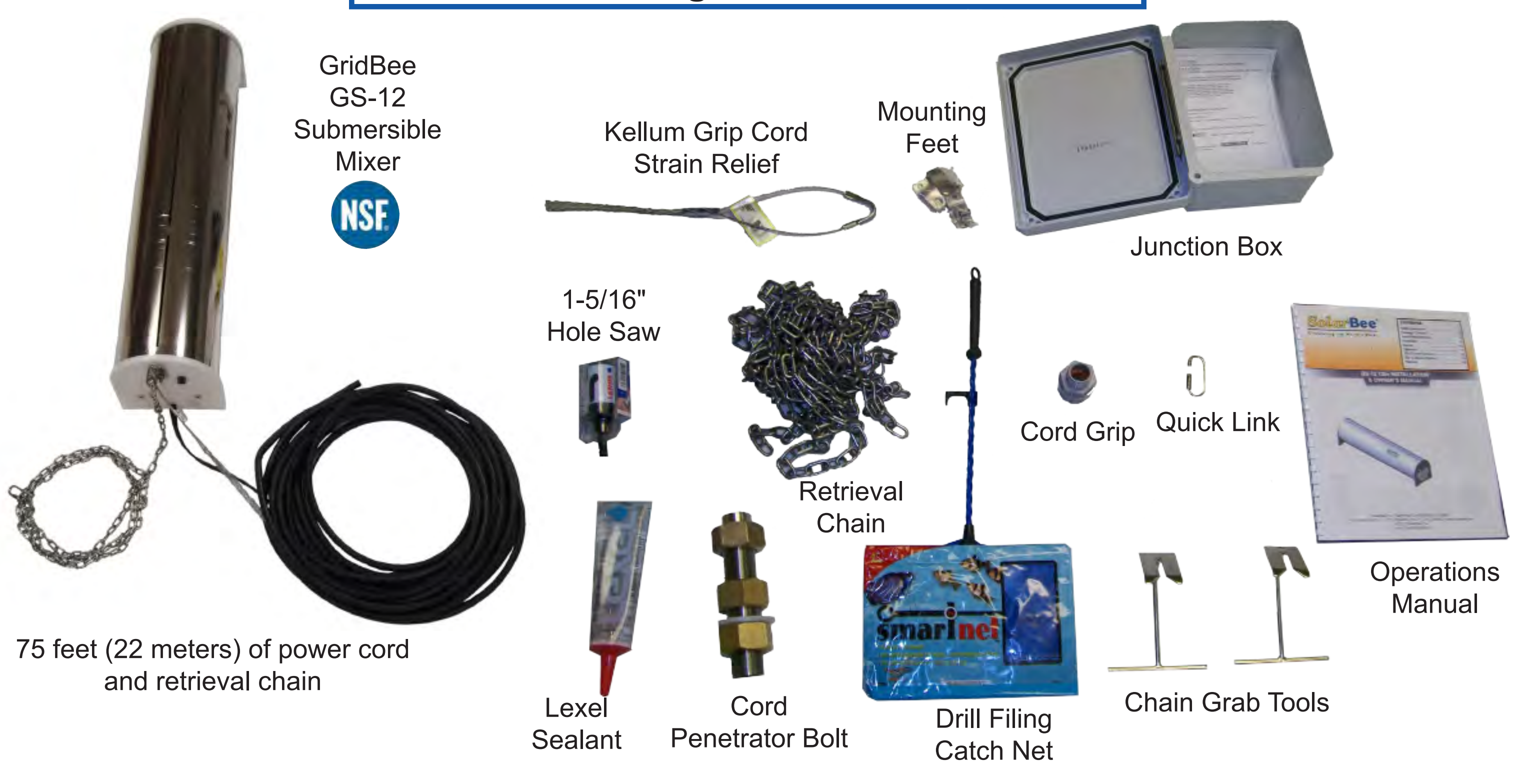
GS-12 in 5MG Reservoir



Installation Overview



Package Contents



SolarBee® and GridBee® are brands of Medora Corporation

Medora Corporation
3225 Hwy 22 • Dickinson, ND 58601
Ph +1 866 437 8076 • www.medoraco.com

GS12_20130207 • © 2013 Medora Corporation

Locally Represented By:

SolarBee®

Circulating the World's Water

POTABLE WATER MIXERS

Why Mix Your Potable Tank?

Active mixing in water storage tanks ensures uniform distribution of disinfectants and representative sampling. Well-mixed tanks consume less disinfectant chemical, produce fewer disinfection by-products, and eliminate the need for energy-intensive and costly deep-cycling or flushing.

Stagnation in Potable Water Storage Reservoirs Can Cause:

- Loss of residual disinfectant (chlorine or chloramine)
- Inconsistent water age, taste and odors
- Thermal stratification - even 0.1º C differential can inhibit mixing effects of normal inflow and outflow
- Nitrification and high heterotrophic plate counts
- Excessive ice buildup and tank damage in cold climates

SolarBee Offers a Complete Line of Potable Mixers

Mix Any Size and Shape of Tank, Corner to Corner

SB500PWc
SB1250PWc

SB1250PW

SB2500PW
SB5000PW

SB10000PW

Benefits of SolarBee's Potable Water Mixing:

- Uniform distribution of disinfectants, consistent residual readings, representative sampling
- Impacts the tank boundry layers where the bacteria build up, provides uniform water age
- Prevents stagnation, thermal stratification, and short-circuiting
- Reduces nitrification and high heterotrophic plate counts
- Reduces ice buildup and tank damage in cold climates





SolarBee, Inc. • 3225 Hwy 22 • Dickinson, ND 58601
Ph +1 701 225 4495 • Fax +1 701 225 0002 • +1 866 437 8076 • www.solarbee.com
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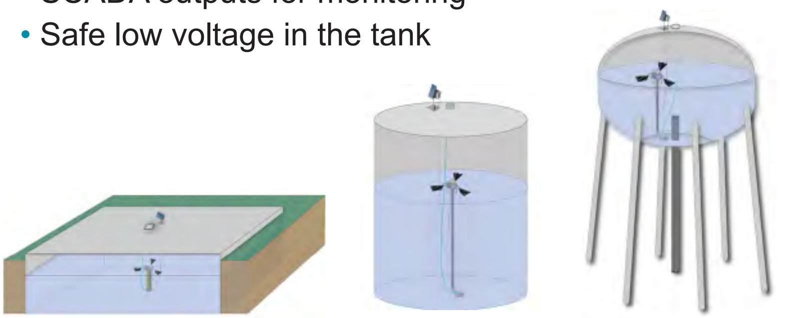
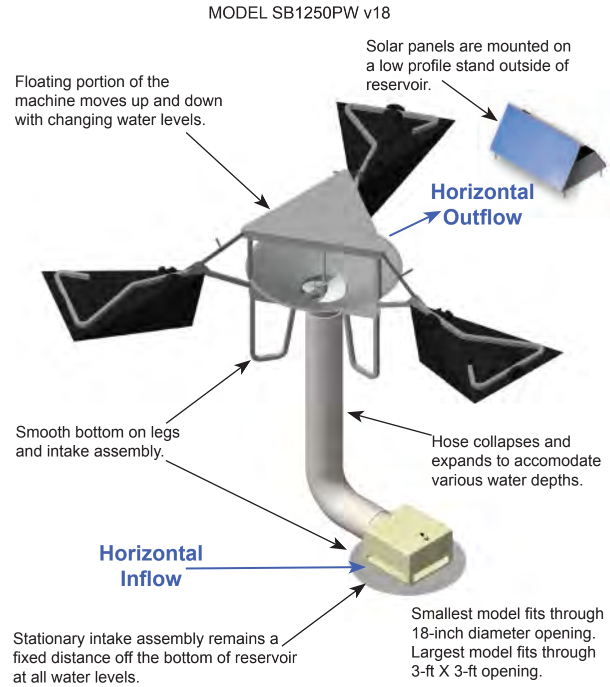
PW20110224

Features of SolarBee Mixing Equipment:

- Operates day and night on solar power or low energy grid power
- Collapsible design allows for customer or factory installation
- Fits through hatches as small as 18" diameter
- Certified to NSF / ANSI Std 61-G
- Injection system provided for boosting
- One moving part / 25 year expected life
- Self-adjusts for varying water levels
- SCADA outputs for monitoring
- Safe low voltage in the tank

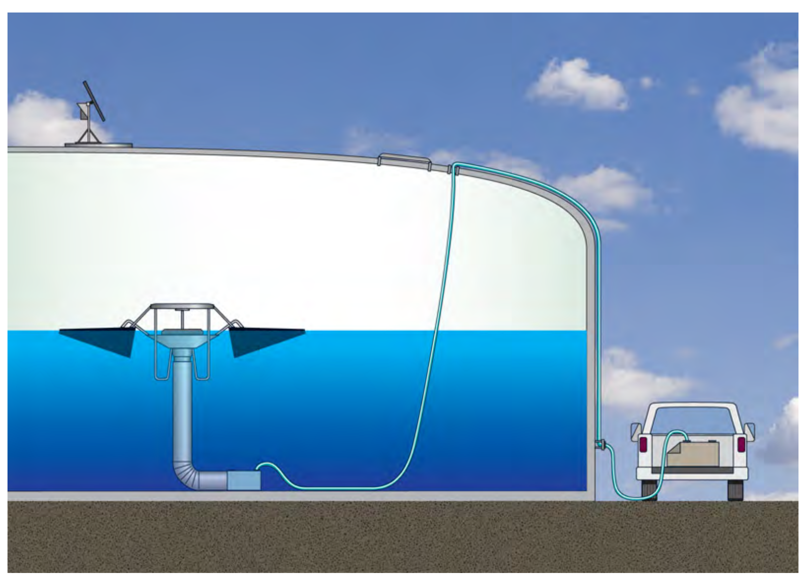


Underground Storage ***Ground Storage*** ***Elevated Storage***



Mix First, Then Boost!

Frequent boosting with small doses of chlorine is far less costly than having a major problem occur in a tank. All SolarBee potable water mixers are equipped with chemical injection capability, and we offer an optional Chlorine Boost System accessory product to allow you to dose small amounts of chlorine as required to maintain your desired chlorine residual level. The Chlorine Boost System is a portable, air-operated chlorine injection system. It is designed to be mounted in the back of a pickup truck, allowing a single operator to safely and reliably boost multiple tanks in one day from ground level. SolarBee mixing, frequent sampling, and the Chlorine Boost System, helps to ensure water quality compliance.



See what SolarBee's potable tank mixing can do for you.
Watch the video at www.potable.solarbee.com/video

SolarBee, Inc. • 3225 Hwy 22 • Dickinson, ND 58601
Ph +1 701 225 4495 • Fax +1 701 225 0002 • +1 866 437 8076 • www.solarbee.com
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OREGON WATER RESOURCES DEPARTMENT

2020 SOLICITATION

WATER PROJECT GRANTS AND LOANS

GRANT APPLICATION

ATTACHMENT # 5

LETTERS OF SUPPORT

May 20, 2020

Paul Wilborn, General Manager
Tri City Water & Sanitary Authority

Re: Tri City Water & Sanitary Authority
Letter of Support
New Water Tank for Improved Service to Community

Dear Paul:

Thank you for informing me concerning Tri City's efforts to improve water service to our community. I understanding you are seeking grant funding from the Oregon Water Resources Department to help address your community deficiency in water storage capacity and fire suppression capability.

Our underserved community will benefit from this project in a number of ways. I am the Public Works Director of the City of Myrtle Creek. Myrtle Creek and Tri City share not only residents and business, but also our water resources. Our communities have collaborated in a number of ways to help provide excellent water service to our residents and businesses. Tri City must develop this new water storage capability if it hopes to continue its mission to provide excellent service to local residents and businesses. Economic development opportunities are the lifeblood that keeps communities thriving. This water storage tank project is critical to ensure that local residents and business can live and work in our communities, and that our economies continue to grow. This improvement will help to benefit the public and local business by ensuring that residential and business assets are protected from fire risk. Investors are attracted to this area partially due to the ability of Tri City to provide a safe and well served community. Tri City is seeking to not only to improve service and reduce risks to the community, but also to protect our precious water resources through actively reducing water system leaks and waste. These efforts help to set the stage with developers and investors, that Tri City is open for business.

On behalf of the City of Myrtle Creek, I fully support Tri City's efforts to better serve our community, and to help enable it to grow long into the future.

Please feel free to contact me if I can provide additional information that will help to improve Tri City's probability of constructing this important community project.

Sincerely,

Quinn Pickering
Public Works Director
City of Myrtle Creek
qpickering@ci.myrtle-creek.or.us
(541) 863-3171

- City of Riddle -

(541) 874-2571 P.O. Box 143 * Riddle, Oregon 97469 Fax (541) 874-2625 E-mail:coriddle@frontiernet.net

City Government
Public Works
Water Quality
Wastewater Treatment
Water Treatment
Parks and Recreation
Public Education

May 08, 2020

Paul Wilborn, General Manager
Tri City Water & Sanitary Authority

Re: Tri City Water & Sanitary Authority
Letter of Support
New Water Tank for Improved Service to Community

Dear Paul:

Thank you for informing me concerning Tri City's efforts to improve water service to our community. I understanding you are seeking grant funding from the Oregon Water Resources Department to help address your community deficiency in water storage capacity and fire suppression capability.

Our underserved community will benefit from this project in a number of ways. I am the City Manager of the City of Riddle. Riddle and Tri City share not only residents and business, but also our water resources. Our communities have collaborated in a number of ways to help provide excellent water service to our residents and businesses. Tri City must develop this new water storage capability if it hopes to continue its mission to provide excellent service to local residents and businesses. Economic development opportunities are the lifeblood that keeps communities thriving. This water storage tank project is critical to ensure that local residents and business can live and work in our communities, and that our economies continue to grow. This improvement will help to benefit the public and local business by ensuring that residential and business assets are protected from fire risk. Investors are attracted to this area partially due to the ability of Tri City to provide a safe and well served community. Tri City is seeking to not only to improve service and reduce risks to the community, but also to protect our precious water resources through actively reducing water system leaks and waste. These efforts help to set the stage with developers and investors, that Tri City is open for business.

On behalf of the City of Riddle, I fully support Tri City's efforts to better serve our community, and to help enable it to grow long into the future.

Please feel free to contact me if I can provide additional information that will help to improve Tri City's probability of constructing this important community project.

Sincerely,

Kathleen M Wilson
Manager/Recorder
City of Riddle
coriddle@frontiernet.net
541-874-2571

"The City of Riddle is an Equal Opportunity Provider and Employer"

May 07, 2020

Paul Wilborn, General Manager
Tri City Water & Sanitary Authority

Re: Tri City Water & Sanitary Authority
Letter of Support
New Water Tank for Improved Service to Community

Dear Paul:

Thank you for informing me concerning Tri City's efforts to improve water service to our community. It is my understanding you are seeking grant funding from the Oregon Water Resources Department to help address our community deficiency in water storage capacity and fire suppression capability.

Our underserved community will benefit from this project in a number of ways. I am a business leader in the community and have been working to create economic opportunities in Tri City's Industrial Park on I-5 at the exit 103 interchange. Tri City provides service not only to the South Umpqua Industrial Park and to community businesses, but also to its residents, who live and work here. This improvement will help to benefit the public and local businesses by ensuring that residential and business assets are protected from fire risk. Investors are attracted to this area partially due to the ability of Tri City to provide a safe and well served community. Tri City is seeking to not only improve service and reduce risks to the community, but also to protect our precious water resources through actively reducing water system leaks and waste. These efforts help to set the stage with developers and investors, that Tri City is open for business.

As we continue in the effort to bring more businesses to our area and provide more services and retail selection to our residents, it is vital to continue to improve our infrastructure, giving potential new businesses the confidence they need to make the next big step and invest in our community. Our residents deserve to have more services and selection closer to home. The more we continue to improve basic infrastructure, the more it will be a win/win for businesses and residents alike.

On behalf of MSK Building Supply, I fully support Tri City's efforts to better serve our community, and to help enable it to grow long into the future.

Please feel free to contact me if I can provide additional information that will help to improve Tri City's probability of constructing this important community project.

Sincerely,

Jeffrey B. Johnson
President / Owner
MSK Building Supply
(541) 863-3127

UMPQUA ECONOMIC DEVELOPMENT
PARTNERSHIP

May 8, 2020

Paul Wilborn, General Manager
Tri City Water & Sanitary Authority

Re: Tri City Water & Sanitary Authority
Letter of Support
New Water Tank for Improved Service to Community

Dear Paul:

Thank you for informing me concerning Tri City's efforts to improve water service to our community. I understanding you are seeking grant funding from the Oregon Water Resources Department to help address our community deficiency in water storage capacity and fire suppression capability.

Our underserved community will benefit from this project in a number of ways. I am a business leader in the community and have been working to create economic opportunities in Tri City's Industrial Park. Tri City provides service not only to the Industrial Park and to community businesses, but also to its residents, who live and work here. This improvement will help to benefit the public and local business by ensuring that residential and business assets are protected from fire risk. Investors are attracted to this area partially due to the ability of Tri City to provide a safe and well-served community. Tri City is seeking not only to improve service and reduce risks to the community, but also to protect our precious water resources through actively reducing water system leaks and waste. These efforts help to set the stage with developers and investors, that Tri City is open for business.

On behalf of Umpqua Economic Development Partnership, I fully support Tri City's efforts to better serve our community, and to help enable it to grow long into the future.

Please feel free to contact me if I can provide additional information that will help to improve Tri City's probability of constructing this important community project.

Sincerely,

Wayne Patterson
Executive Director

COW CREEK BAND OF UMPQUA TRIBE OF INDIANS
GOVERNMENT OFFICES
2371 NE STEPHENS STREET, SUITE 100
ROSEBURG, OR 97470-1399
Phone: 541-672-9405
Fax: 541-673-0432

May 28, 2020

Paul Wilborn, General Manager
Tri City Water & Sanitary Authority
Via – Email

Re: Tri City Water & Sanitary Authority
Letter of Support
New Water Tank for Improved Service to Community

Dear Mr. Wilborn:

Thank you for informing the Cow Creek Band of Umpqua Tribe of Indians (Tribe) concerning Tri City's efforts to improve water service to the Tri City community. The Tribe understands that you are seeking grant funding from the Oregon Water Resources Department to help address your community deficiency in water storage capacity and fire suppression capability. Tri City is home to many Cow Creek Tribal Members who would benefit from this project.

On behalf of the Tribe, I would like to offer full support for Tri City's efforts to better serve the Tri City community and the needs of our tribal membership who live in the community.

Please feel free to contact me if I can provide additional information that will help to improve Tri City's probability of constructing this important community project.

Sincerely,

Jason Robison
Natural Resource Director
Cow Creek Band of Umpqua Tribe of Indians
jrobison@cowcreek.com
541-677-5516

TRI-CITY RURAL PROTECTION DISTRICT NO.4

May 11, 2020

Paul Wilborn, General Manager
Tri-City Water & Sanitary Authority

Re: Tri-City Water & Sanitary Authority
Letter of Support
New Water Tank for Improved Service to Community

Dear Paul:

Thank you for informing me concerning The Tri-City Water and Sanitary Authorities efforts to improve fire protection in our community. I understanding you are seeking grant funding from the Oregon Water Resources Department to help address our deficiency in water storage capacity and fire suppression capability.

Our underserved community will benefit from this project in a number of ways. I am the Chairman of the Board for the Tri-City Fire Department. I have been closely involved with the Tri-City area for the past 40 years and have seen the district suffer from substandard fire water flows in many areas. The addition of this new storage tank will vastly improve our fire water flows in an area that desperately needs it. This water storage tank project will also ensure that our community can continue to grow both in terms of residential properties and in new business development. New development can only occur in an area that has the sufficient resources to provide necessary services to the area. This new water tank will assure that Tri-City can provide a safe and well served community. I know that the Tri-City Water and Sanitary Authority has worked diligently to improve service and reduce risks to the community as well as to protect our precious water resources through actively reducing water system leaks and waste.

On behalf of Tri-City Fire Department, I fully support the efforts of the Tri-City Water and Sanitary Authority's efforts to increase our fire suppression efforts to better serve our community, and to help enable it to grow long into the future.

Please feel free to contact me if I can provide additional information that will help to improve Tri-City's probability of constructing this important community project.

Sincerely,

Joe Pospisil
Fire Board Chairman
Tri-City Rural Fire Protection District #4
tcfdboard@gmail.com
541-643-7756

2020 SOLICITATION

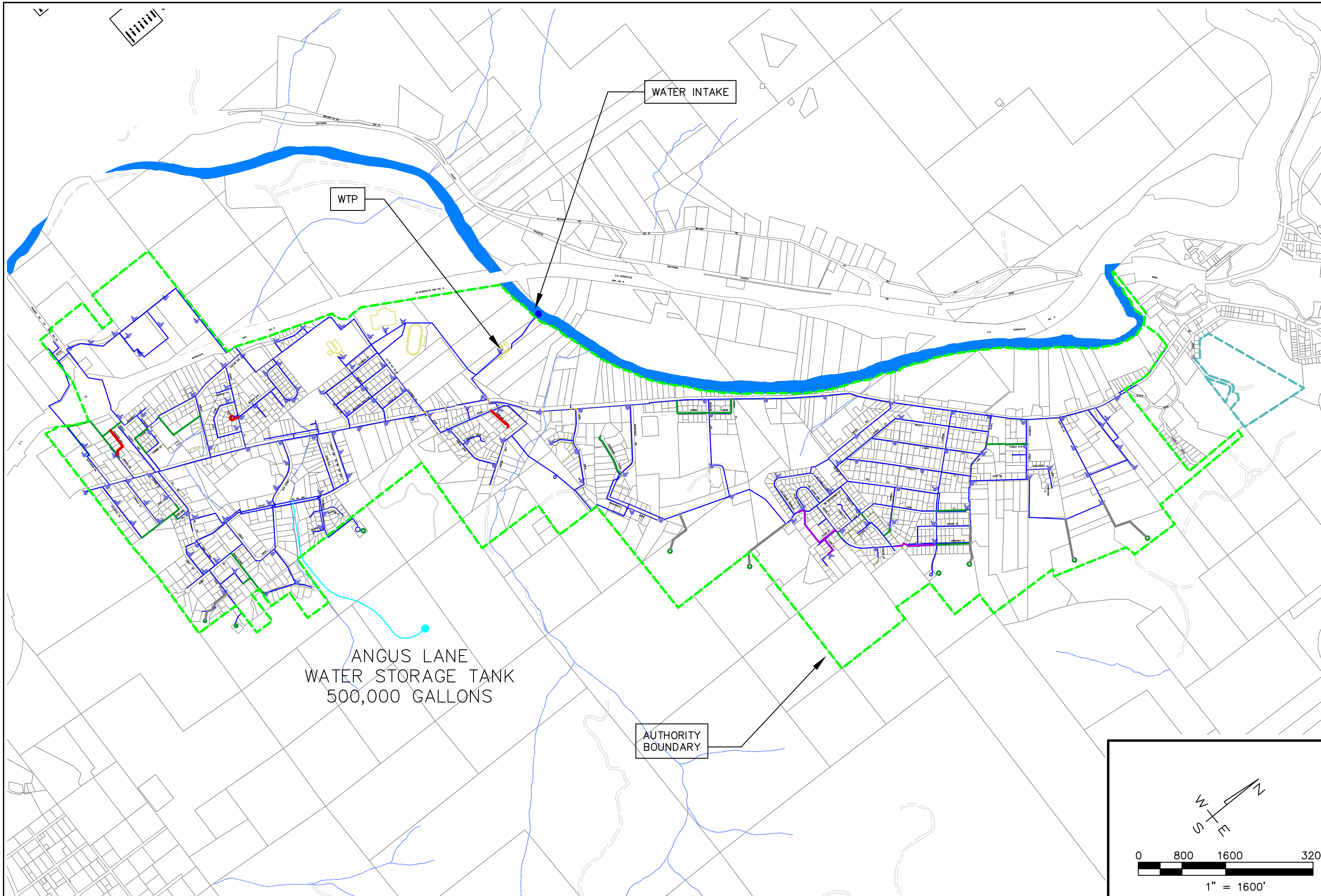
WATER PROJECT GRANTS AND LOANS

GRANT APPLICATION

ATTACHMENT # 6

TRI CITY WATER & SANITARY AUTHORITY

AREA MAP



TRI-CITY JOINT WATER AND SANITARY AUTHORITY

AREA MAP
T29S-30S, R5W

DRAWN BY: REB

DATE: Dec. 21, 2005

FIG. 1-2

OREGON
WATER RESOURCES DEPARTMENT

2020 SOLICITATION

WATER PROJECT GRANTS AND LOANS

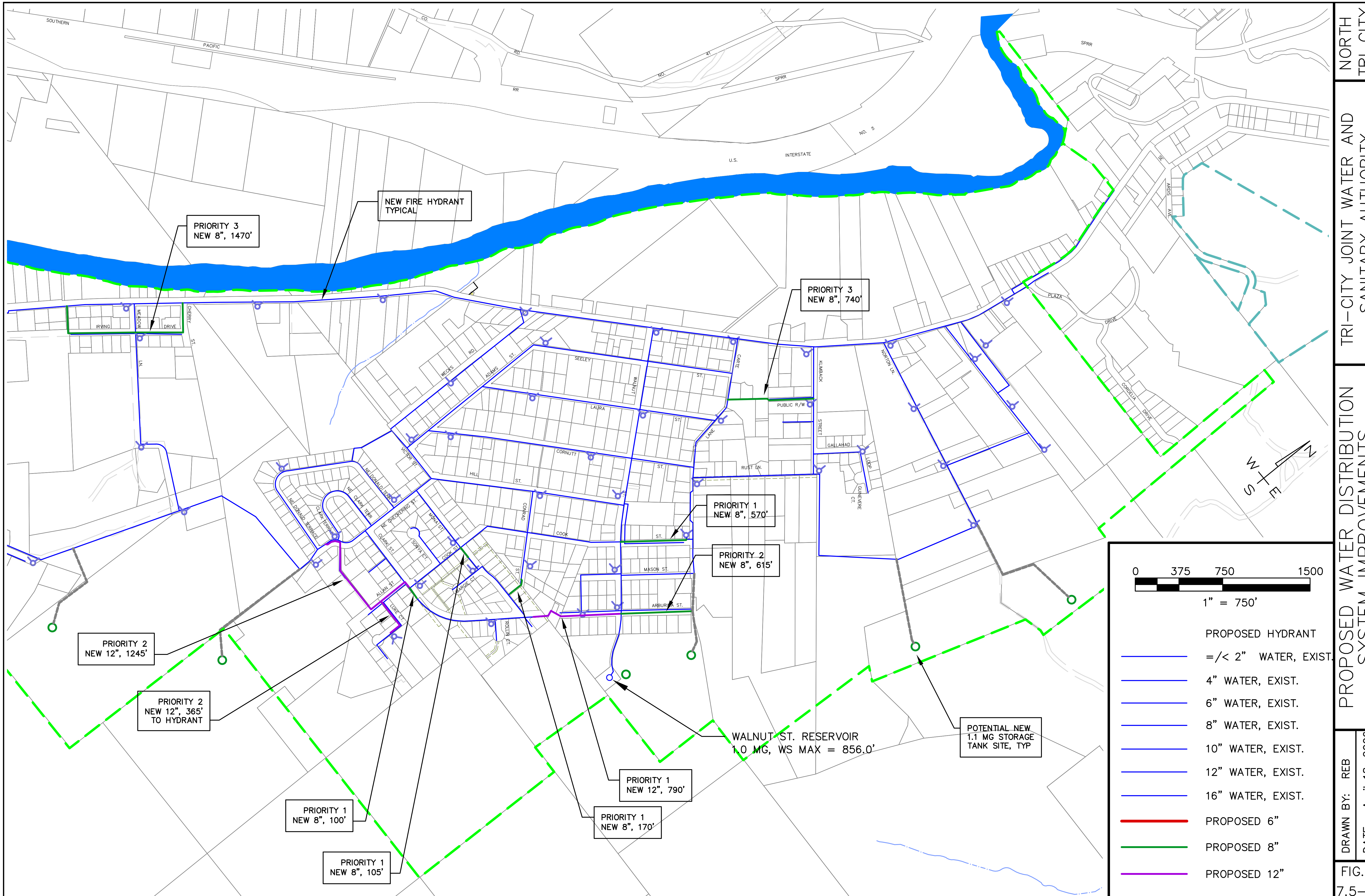
GRANT APPLICATION

ATTACHMENT # 7

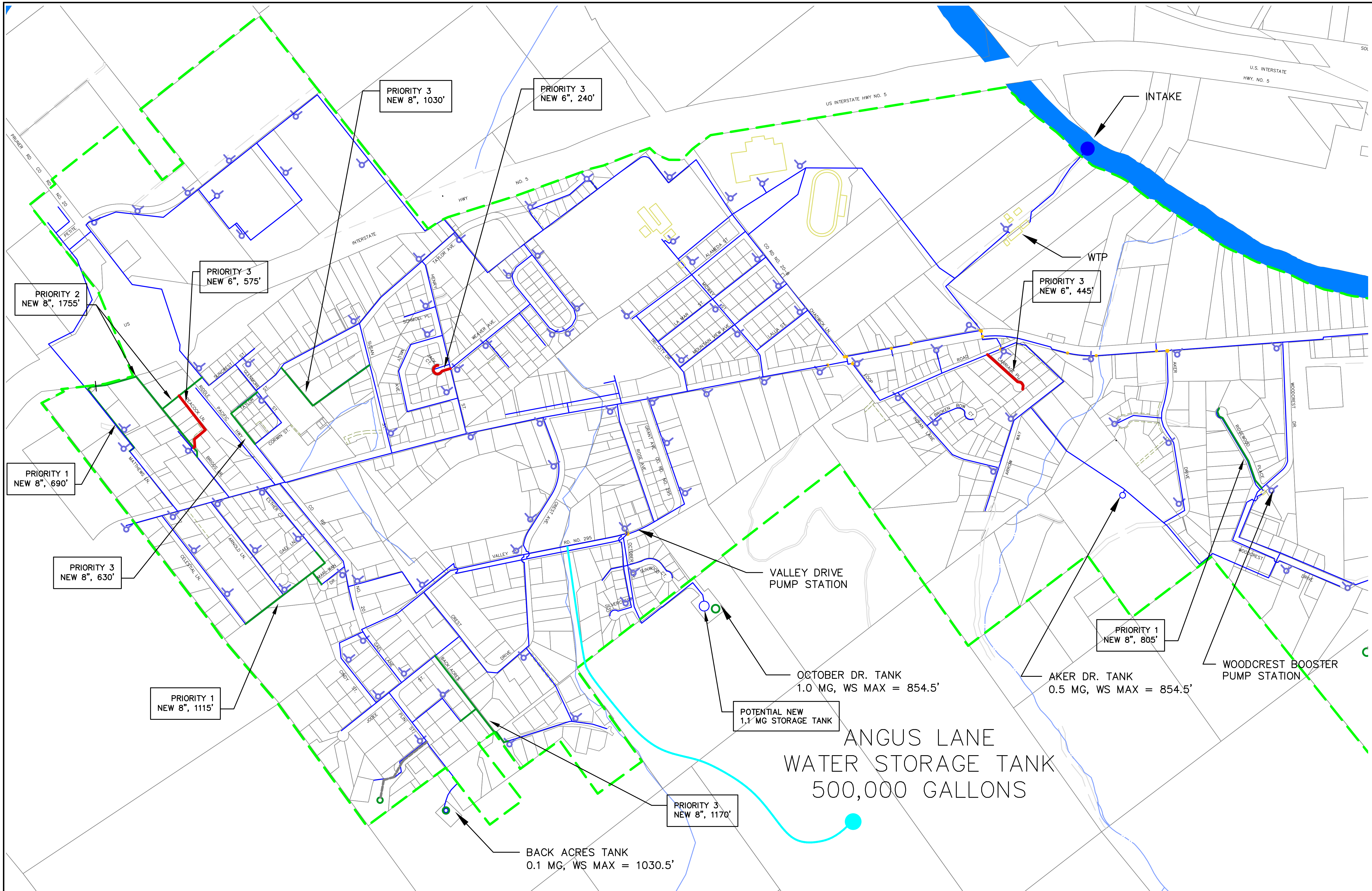
TRI CITY WATER & SANITARY AUTHORITY

RELEVANT MAPS

- **WATER DISTRIBUTION SYSTEM – NORTH AREA**
- **WATER DISTRIBUTION SYSTEM – SOUTH AREA**
- **ZONING MAP – NORTH AREA**
- **ZONING MAP – SOUTH AREA**



PRIORITY 3
NEW 8", 1470'
NEW FIRE HYDRANT
TYPICAL
PRIORITY 3
NEW 8", 740'
PRIORITY 1
NEW 8", 570'
PRIORITY 2
NEW 8", 615'
PRIORITY 2
NEW 12", 1245'
PRIORITY 2
NEW 12", 365'
TO HYDRANT
WALNUT ST. RESERVOIR
1.0 MG, WS MAX = 856.0'
POTENTIAL NEW
1.1 MG STORAGE
TANK SITE, TYP
PRIORITY 1
NEW 12", 790'
PRIORITY 1
NEW 8", 170'
PRIORITY 1
NEW 8", 100'
PRIORITY 1
NEW 8", 105'
0 375 750 1500
1" = 750'
PROPOSED HYDRANT
=/< 2" WATER, EXIST.
4" WATER, EXIST.
6" WATER, EXIST.
8" WATER, EXIST.
10" WATER, EXIST.
12" WATER, EXIST.
16" WATER, EXIST.
PROPOSED 6"
PROPOSED 8"
PROPOSED 12"
NORTH
TRI CITY
TRI-CITY JOINT WATER AND
SANITARY AUTHORITY
PROPOSED WATER DISTRIBUTION
SYSTEM IMPROVEMENTS
DRAWN BY: REB
DATE: April 18, 2006
FIG.
7.5-2



SOUTH TRI CITY

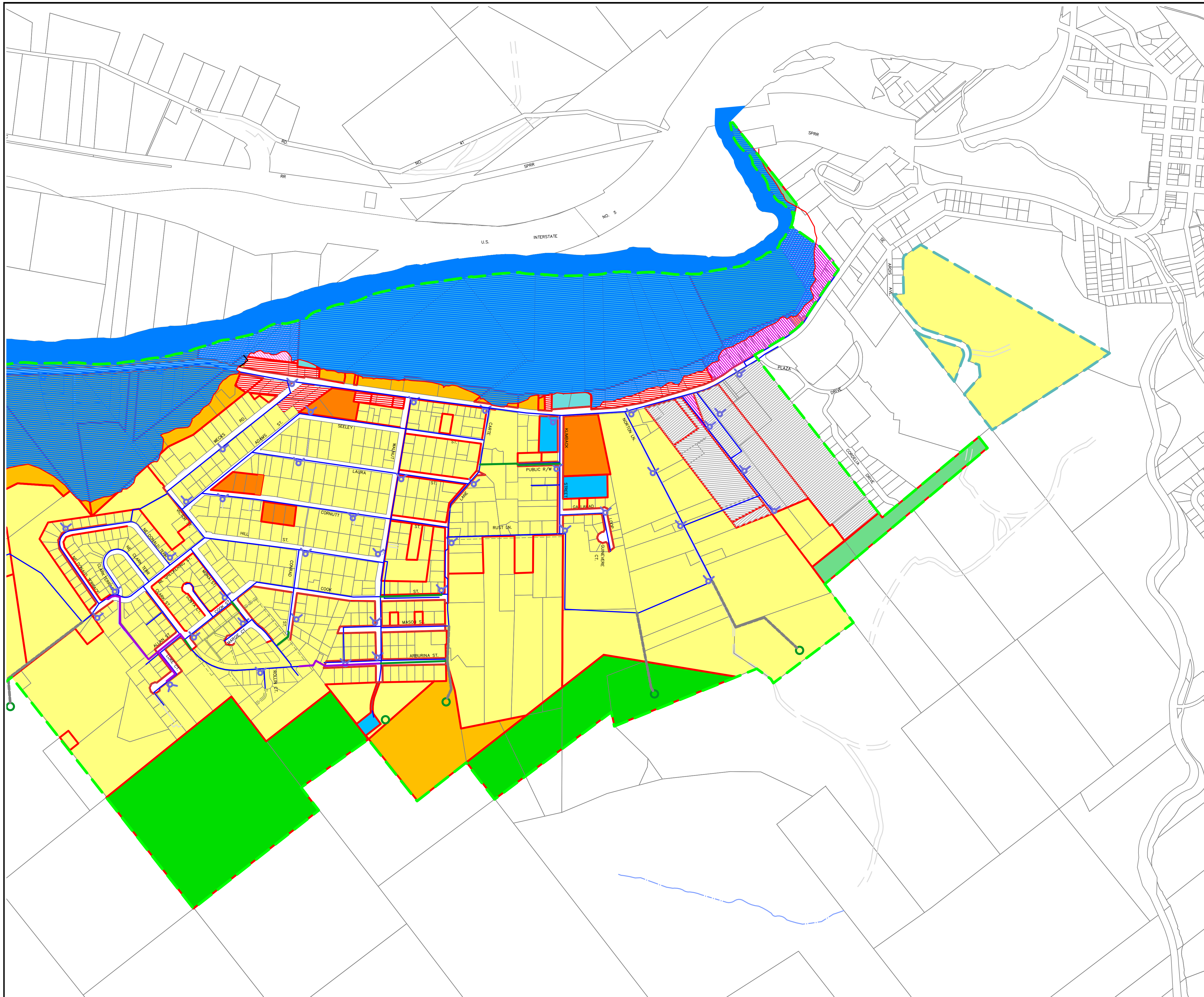
TRI-CITY JOINT WATER AND SANITARY AUTHORITY

PROPOSED WATER DISTRIBUTION SYSTEM IMPROVEMENTS

DRAWN BY: REB

DATE: April 18, 2006

FIG. 7.5-2



LEGEND

UGB	(1954 Acres) 3.05 SQ. MI.
R1	(832 Acres)
R2	(47 Acres)
RS	(263 Acres)
CT	(37 Acres)
C2	(33 Acres)
C3	(35 Acres)
M1	(17 Acres)
M2	(40 Acres)
M3	(87 Acres)
F1	(74 Acres)
F2	(79 Acres)
FF	(12 Acres)
FG	(50 Acres)
PR	(89 Acres)
R/W	(259 Acres)

N W E S

0 450 900 1800

1" = 900'

HBH Consulting Engineers

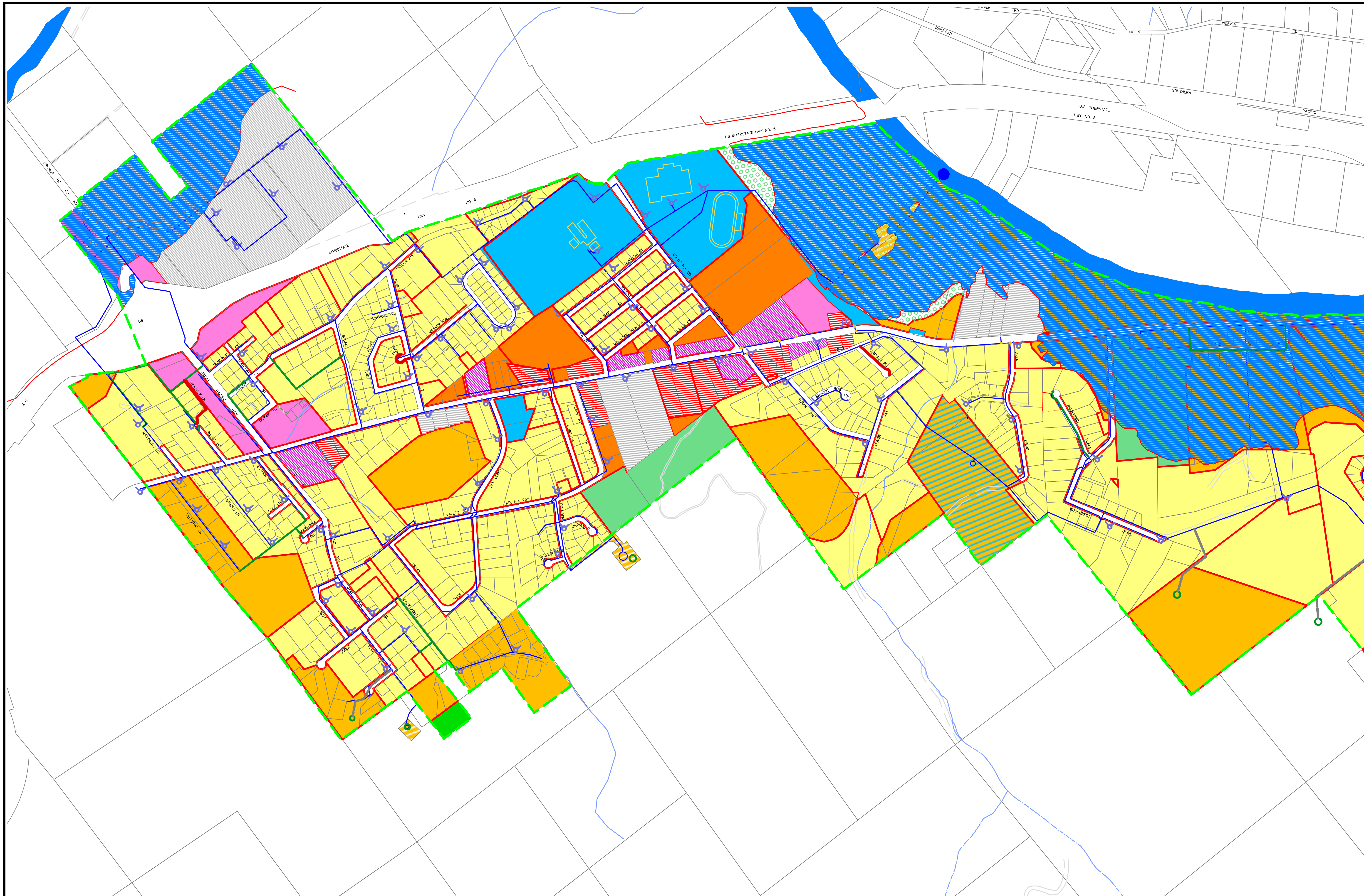
TRI-CITY JOINT WATER AND SANITARY AUTHORITY

ZONING MAP

DRAWN BY: REB

DATE: Nov. 3, 2004

FIG. 2-9


US INTERSTATE HWY NO. 5
U.S. INTERSTATE
HWY. NO. 5
SOUTHERN
PACIFIC
RAILROAD
WEAVER
RD.
NO. 41
VALLEY
RD. NO. 295