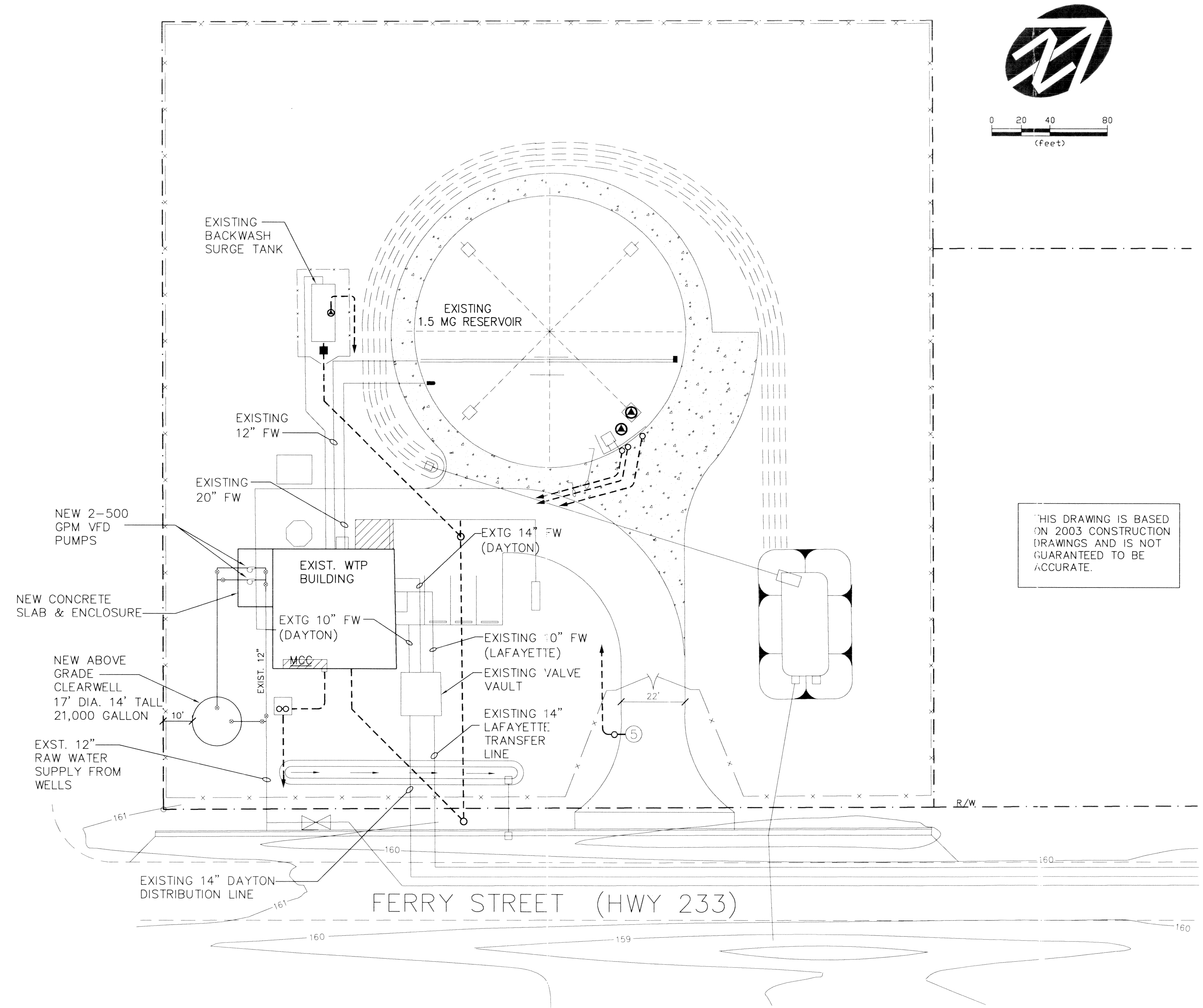

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CITY OF DAYTON, OREGON

WATER TREATMENT PLANT & BOOSTER PUMP STA.

BACKWASH CLEARWELL AND BOOSTER PUMP LAYOUT

FIGURE 7-1

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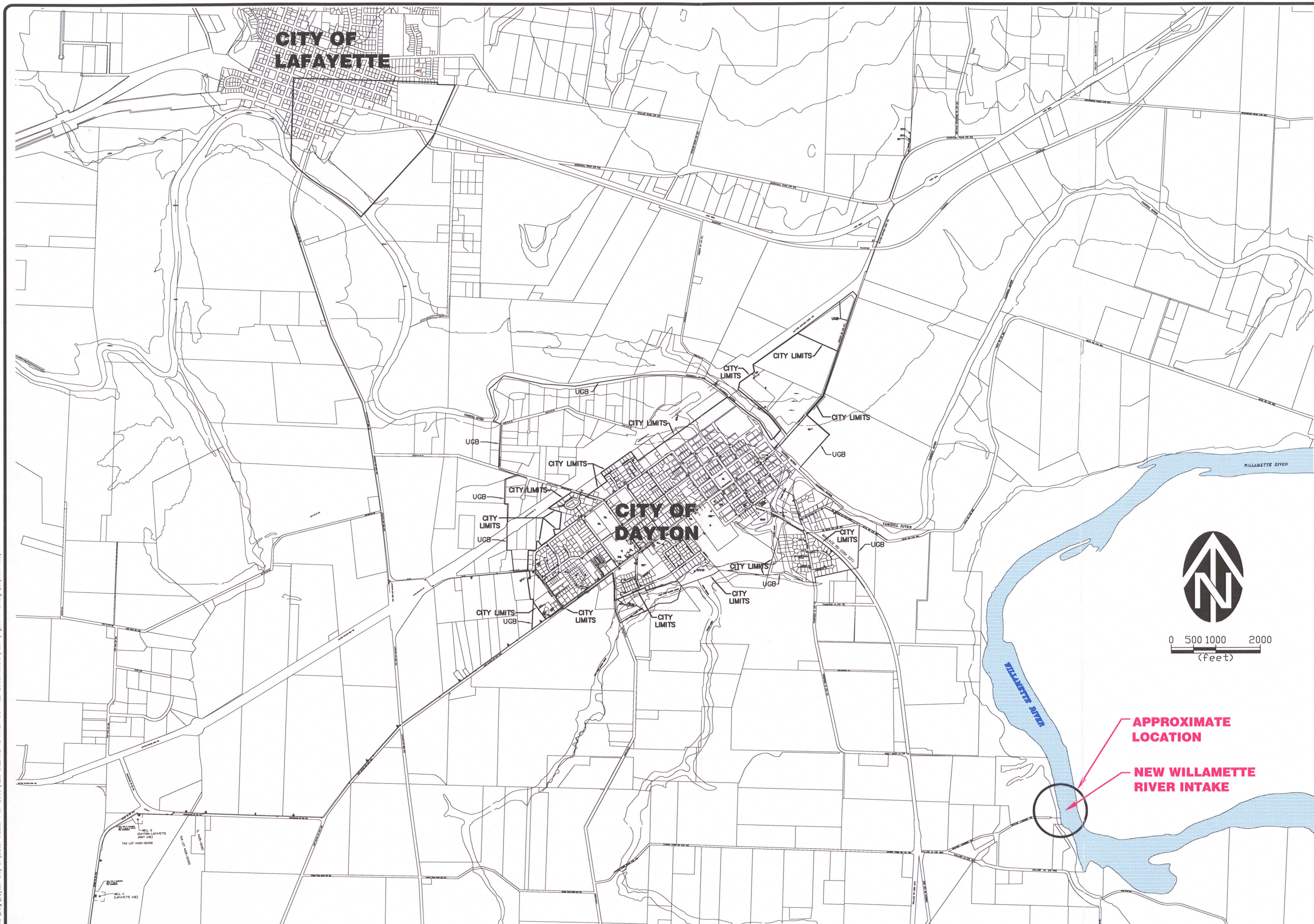

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CITY OF DAYTON, OREGON
DAYTON WATER MASTER PLAN

PROPOSED
WILLAMETTE RIVER INTAKE
LOCATION MAP

FIGURE
7-2

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7.3.2 Existing WTP, Evaluation of Finish Water Pumps

As discussed in Section 4.4.4, the distribution pumps at the WTP pump water into the distribution system, with the distribution system pressure being maintained by these pumps. The WTP was constructed with two electric distribution pumps and one propane powered high flow (fire) pump, with piping and valving for a third distribution pump. As noted on **Table 4-7**, the nominal design capacity of the distribution pumps was 250 and 500 gpm. The assumed capacity of the future distribution pump was 500 gpm.

As shown on **Table 5-4**, the projected peak demands will exceed the nominal capacity of the WTP existing distribution pumps early in the planning period. The City needs to plan to add the third distribution pump at the WTP within several years. It should be noted that if the leakage rate is reduced as discussed in this master plan, the distribution system demands will be decreased substantially, and the timeframe for adding the third pump may be able to be delayed for a number of years.

7.4 WATERSHED SPRINGS TREATMENT EVALUATION

As discussed in Section 4.3.2.3 and Section 4.8, the ODWP has expresses concerns that the watershed springs have the potential of being surface water influenced. Correspondence between the ODWP and the City suggests two potential alternatives to fix the potential GWUDI concern as listed below.

- *Alternative 1* – Reconstruction the spring collection system to bring it up to the construction standards found in OAR 333-061-0050(2)(b), such that the spring water can be considered groundwater and not subject to the surface water treatment rules. The ODWP did not dictate the exact improvements required to accomplish this, other than those items specifically required by state rules.
- *Alternative 2* - The other alternative is for the City to accept the watershed springs as GWUDI and subject to the surface water treatment rules. This would require construction of a surface water treatment facility and disinfection in order for the springs to remain in service.

As discussed in detail under Section 4.4.3, the existing slow sand filter is not considered adequate to provide the required level of treatment under Alternative 2, and it does not appear that it can be upgraded to provide the required level of treatment if the springs are determined in be a GWUDI source.

7.4.1 Watershed Springs, Improvement Alternatives

Four alternatives were developed and evaluated to address the issues facing the City relative to the springs. The first three alternatives are a stepwise progression of the options to avoid having the spring water classified as GWUDI, as well as avoiding the resultant surface water treatment requirements. The final alternative addresses the second option presented by the ODWP. The alternatives are summarized in **Table 7-2** below, generally listed from least to greatest cost.

Table 7-2 Watershed Springs, Water Quality Compliance Alternatives

Alternative	General Description of Improvements
1A	Reconstruction of Spring Boxes & Collection Piping
1B	New Spring Collection System w/HDPE Liners & Earth Covers
1C	Drill Groundwater Wells and Transfer Spring Water Rights
2	Accept Springs as GWUDI & Construct Surface WTP (Packaged Membrane Filtration Plant)

The following is a general description of each alternative, including the specific improvements anticipated, as well as the potential risks and long term costs associated with each.

7.4.1.1 Alternative 1A: Reconstruction of Spring Boxes & Collection Piping

This alternative consists of reconstruction of the existing springs boxes and collection piping to meet the requirements of 333-061-0050 (2)(b). In general, this will include the following.

(1) Interceptor ditch uphill from all spring collection boxes, to divert surface runoff away from the collection box locations. This may require some clearing and excavation along the toe of the slope to address those boxes located close to the hill.

(2) Provide a compacted bentonite/clay seal as required around the outside of the existing spring boxes to reduce the risk of surface water migrated down the outside of the spring boxes and entering the collection system.

(3) Replace spring box structures that do not have adequate structural integrity to allow for installation of the compacted bentonite seal noted above.

(4) Install each spring box with a lockable, watertight shoebox style lid to prevent leakage and to exclude surface water.

(5) Provide screened overflows for the existing spring boxes. We assume that where grades do not permit individual overflows, the City can construct a combined overflow to serve more than one collection box (collection boxes which are in close proximity to each other).

(6) Replace all collection piping between the spring boxes and the sand filter structure. Provide junction structures as required to allow inspection and monitoring of collection piping junction points.

(7) Provide a flow meter to separately monitor the flows from the upper and the lower spring areas.

(8) Add isolation valves to each of the spring systems with a bypass system (overflow or otherwise), to allow Public Works the flexibility to remove individual or groups of collection boxes off-line for maintenance without shutting of the entire spring system.

(9) If required by ODWP, install fencing and gates around the spring areas to keep deer and other large animals away from the area around the spring collection boxes.

(10) Repair roof on existing junction structure just uphill from sand filter structures.

While the proposed improvements are expected to adequately protect the spring water from contamination by fecal coliforms, *E. coli* or other listed pathogenic organisms, there is still a likelihood that (even with these improvements) that the spring water may still be subject to viral contamination. In order to address the disinfection inactivation requirements of the new Groundwater Rule, addition of chlorination equipment at the spring area is recommended. This will also address any existing inadequacies in the CT values prior to the first watershed services (the transmission mains should provide adequate contact time in all cases).

(11) Provide chlorination equipment at the spring locations. This will require the installation of a power service into the spring area from Breyman Orchards Road. For purposes of this evaluation, we are assuming that the power lines will be overhead from Breyman Orchards Road to the watershed

gate, and then underground to the spring area (unless an easement can be obtained across the property between Breyman Orchards Road and the spring area)., and that a single phase service will be adequate.

(12) Installation of a small auxiliary power generator to ensure that the chlorination system remains in service during power outages.

(13) Evaluate the feasibility of providing telemetry improvements (including a radio link) to allow for remote monitoring of the spring area flows and chlorination system.

(14) Provide the following improvements to the existing sand filter structures, *if they remain in service*. Until the efficacy of the proposed spring improvements is determined, the City may want to defer the construction of these improvements, since the subsequent alternatives (if required) may eliminate the need for the sand filter structures entirely. If the sand filter structures are removed from service, the spring water should be piped around structures.

- Repair the existing roof structures as required, and install new gutters between the two roofs.
- Consider reconstructing the roofs to eliminate runoff to the gutter between the two roofs.
- Raise the sidewalls or install sideboards to minimize the risk of splashing into the reservoirs.

Long term maintenance costs for these improvements is expected to be minimal, although the City will need to keep brush and trees cut back around the spring box areas to avoid intrusion or damage due to tree roots, etc.

As noted by GSI Water Solutions, the risk to proceeding with this alternative is that the existing spring boxes may not be able to adequately sealed to exclude surface water influence. Based on evaluations of the spring boxes by GSI Water Solutions in 2007 (see Appendix J), most of the spring boxes do not appear to be adequately sealed to the underlying bedrock, and the condition of the existing horizontal collection pipes from the spring boxes back into the hillside are unknown.

There is a considerable risk that even after completion of these improvements, that the City may still need to proceed with improvements under Alternative 1B (in order to avoid the GWUDI designation).

7.4.1.2 Alternative 1B: Construction of New Spring Collection System w/HDPE Liners & Earth Covers

If the improvements under Alternative 1A above are not pursued, or if pursued are not successful in avoiding the designation of the spring waters as GWUDI (for one or both spring areas), the City should proceed to Alternative 1B.

In general, anticipated improvements under Alternative 1B include the following.

(1) Excavate the area around the existing spring boxes, and install a drainrock collection area with a 60 mil HPDE cover that is keyed back into the hillsides above the spring areas. This will require the construction of the following system components at each spring collection area.

- Concrete cutoff wall at the lower end of the spring collection area excavation (to anchor the liner to, and to provide the lower containment seal for the drainrock collection area).
- Excavation of spring collection area.

- Installation of a geotextile fabric bottom liner in the excavated spring collection area, following by washed drainrock to provide adequate collection area and volume, following by a geotextile fabric top liner and a 60 mil HDPE cover keyed into the slopes behind the spring collection areas, and sloped to direct all near surface infiltration over the concrete cutoff wall.
- Installation of a bentonite/clay seal around the perimeter of the HDPE cover, followed by installation of a ±2 foot thick earth cover to protect the HPDE cover and collection area.

(2) Provide the screened overflows, collection piping, isolation valves and flow meters similar to those recommended under Alternative 1A. It is likely that many of the Alternative 1A improvements would not be able to be utilized under Alternative 1B.

(3) Provide the chlorination facilities as recommended under Alternative 1A.

(4) Repair or upgrade the existing sand filter structures as noted under Alternative 1A, if they remain in service. Otherwise, pipe around the structures and abandon them.

Long term maintenance costs for these improvements is also expected to be minimal, although the City will still need to keep brush and trees cut back around the spring collection areas to avoid intrusion or damage due to tree roots, etc.

Contrary to Alternative 1A, the risk of this alternative being unsuccessful in addressing the GWUDI concerns is considered to be very small. However, it will likely be more expensive, but has the potential of allowing the City to increase the volume of spring water captured, particularly during the summer months. Even if the spring water is susceptible to viral contamination from near surface groundwater, the proposed disinfection (chlorination) improvements should address these concerns as required under the Groundwater Rule.

7.4.1.3 Alternative 1C: Drill Groundwater Wells and Transfer Spring Water Rights

Another option would be to drill a number of wells (either horizontal or vertical) in the watershed area (assuming that the water rights could be transferred from the springs to the new wells). Since this could potentially require the transfer of water rights between two different aquifers, it is not clear that the WRD would approve this approach.

Due to the uncertainties related to the water rights transfers, pursuit of this alternative would require additional investigations and evaluations by a hydrogeologist and water rights examiners to determine if this approach is even feasible. It would also require that the power to the site be installed as a 3 phase service (for the new well pump motors), which will be considerable more expensive than the single phase service for the chlorination system proposed under Alternative 1A or 1B. The new wells would also require the installation of a larger 3 phase auxiliary power generator to run the well pumps.

The long term operation costs associated with this alternative would be substantially higher, due to the requirement for pumping the water from the new wells (as opposed to the current gravity flow from the springs).

7.4.1.4 Alternative 2: Accept Springs as GWUDI & Construct Surface WTP

This alternative assumes that the springs are determined to be GWUDI, and that a surface water treatment plant will be required. This alternative should be considered only as a last option, and the City will need to make the decision as to whether the relatively small flows justify the expense of the treatment plant.

For purposes of this evaluation, we assume that the new WTP will be a packaged membrane filtration or similar plant. It is assumed that this approach will require road improvements for improved all-weather access, 3 phase power, SCADA and telemetry for remote operation & monitoring, a discharge permit for the filter backwash water, etc. The existing road to the spring area would need to reconstructed to allow access for the equipment delivering the skid mounted membrane filtration plant. A larger auxiliary power generator will be required for operation of the new WTP during power outages.

While the surface water treatment alternative would entail the least risk, it would also entail the greatest initial and long term cost. The long term operations and maintenance costs for this approach will be substantial. A complete predesign report would be required prior to obtaining financing for this alternative, in order to prepare detailed cost estimates and to develop preliminary budget estimates. Due to the cost and complexity of this alternative, and the high likelihood that the City can address the spring issues by one of the options under Alternative 1, this approach will not be considered further at this time.

7.4.1.5 Spring Improvement Recommendations

As noted above, the alternatives are listed in order of increasing cost. However, they are also listed in order of decreasing risk of impacts due to surface water contamination potential. Based on discussions with GSI Water Solutions and review of the previous 2007 spring box evaluations, it is recommended that the City proceed with Alternative 1B once funding can be obtained. This approach was successfully applied to two different springs (with similar configurations) in the Lafayette watershed area. In both of those Lafayette cases, the yield from the springs was actually higher after completion of the improvements that it was before. Since the City's water-rights to the existing Dayton Springs are significantly greater than the current dry season flows (ie. during the time when the City most needs the water), we recommend that the City pursue the alternative with the potential for increasing the summer spring flow rates (ie. Alternative 1B).

Depending on the ability of the City to obtain funding sources to complete the permanent upgrades, they may wish to proceed with some of the improvements summarized under Alternative 1A as interim measures (ie. such as installation of new water-tight lids, backfilling around the existing spring boxes with new compacted bentonite/clay seals, trenching diversion channels around the upper side of the spring boxes etc).

Due to the lack of topographic surveys of the entire spring areas, as well as lack of geotechnical data (ie. depth of soil to bedrock, etc.), the current construction cost estimates for the proposed improvements are based on a number of general assumptions relating to the size of the required new spring collection areas. We have tried to be conservative with the assumptions, and as such it is hoped that the City will be able to complete spring upgrades for less than the proposed budget amounts. The City should authorize the preparation of a detailed predesign report to verify all assumptions and estimates herein (including topographic survey and geotechnical investigations of both the upper and lower spring areas), before deciding whether the cost justifies the water which will be obtained.

As noted in the discussion above, all of the alternatives entail the addition of a chlorination facility at the watershed spring site. We assume that the City will utilize liquid hypochlorite injected just downstream of the spring collection areas (at or by the existing sand filter structures).

7.5 REGIONAL SURFACE WTP EVALUATION

As discussed in Chapter 6, options for new water supply sources in or near Dayton are limited. As described in the 2008 "Yamhill County Water Supply Analysis" completed by HDR, Inc. for the Yamhill County Water Task Force, one potential option for additional water supply would be new Willamette River intake and surface water treatment plant (the options available to procure the water rights on the Willamette were discussed in Chapter 6).

A new Willamette River source would require a new intake screen and intake pump station, raw water transmission lines and a surface water treatment facility. Since the City of Dayton and Lafayette already have an intertie between the two cities, a new long term water source would benefit both towns. However, a project of this size and complexity would be very difficult (or impossible) for the Dayton to finance by itself, or even to finance in conjunction with Lafayette.

As noted in Section 6.4.3, it is anticipated that the development of a regional surface water source and treatment plant would require the participation (and leadership) of the City of McMinnville, as well as potential participation by other Yamhill County cities. Interconnections between Dayton/Lafayette and McMinnville could bring three cities together to help finance the project and meet the long-term water supply needs for all three communities.

As part of this Water Master Plan, we have conducted a field investigation of by land and boat (in March 2009) to evaluate potential raw water intake locations along the Willamette River. Due to extensive flood plains and unstable river banks, options in the Dayton area are limited. We located one area which appears to have a stable bank on the west side of the river, without the extensive erosion and slide zones apparent in many other locations.

At the location shown on **Figure 7-2**, the river bank extends well above the river and the flood levels, and appears to be composed of consolidated mudstone over consolidated mudstone/sandstone deposits. The river depth near this location ranged from ±40 to ±50 feet (as measured from the boat sonar). Further geologic and geotechnical work will be required to verify feasibility of this location for a raw water intake and screen. However, it appears feasible and will be carried forward for purposes of this study.

Since the raw water intake and WTP will likely be a regional facility that could serve a multiple jurisdictions (and the number and identify of the jurisdictions is unknown at this time), sizing of intake facilities, pipe lines, etc, are outside of the scope of this master plan. The following is a conceptual summary that may be utilized for an intake and WTP in this location.

The Willamette River intake could be constructed by drilling a vertical shaft on the stable area on the top of the river bluff. The vertical shaft could begin well above the 100-year flood plain, and extend below the Willamette River bottom and serve as a wetwell. A horizontal shaft could then be drilled (from the bottom of the vertical shaft) to the Willamette River, and terminated with an intake screen.

There are two conceptual alternatives developed relative to the location of the new surface WTP and associated finish water pipelines, summarized as follows. Both options include a raw water intake and pump station as summarized above.

- *Alternative 1.* The first option developed (**Figure 7-3**) consists of a raw water pump station on the river bluff near the Willamette River, which would pump raw water to a new surface WTP located just within the southeast corner of the Dayton UGB (ie. between SE Neck Road and Hwy 221/Wallace Road). A finish water pipeline would tie directly to the Dayton distribution system, with a separate finish water pipeline to McMinnville (alignment to be determined). This option would minimize impacts to agricultural lands, and would likely be the simplest in terms of land use approvals. The proposed WTP site is undeveloped, outside of the City Limits but within the UGB.

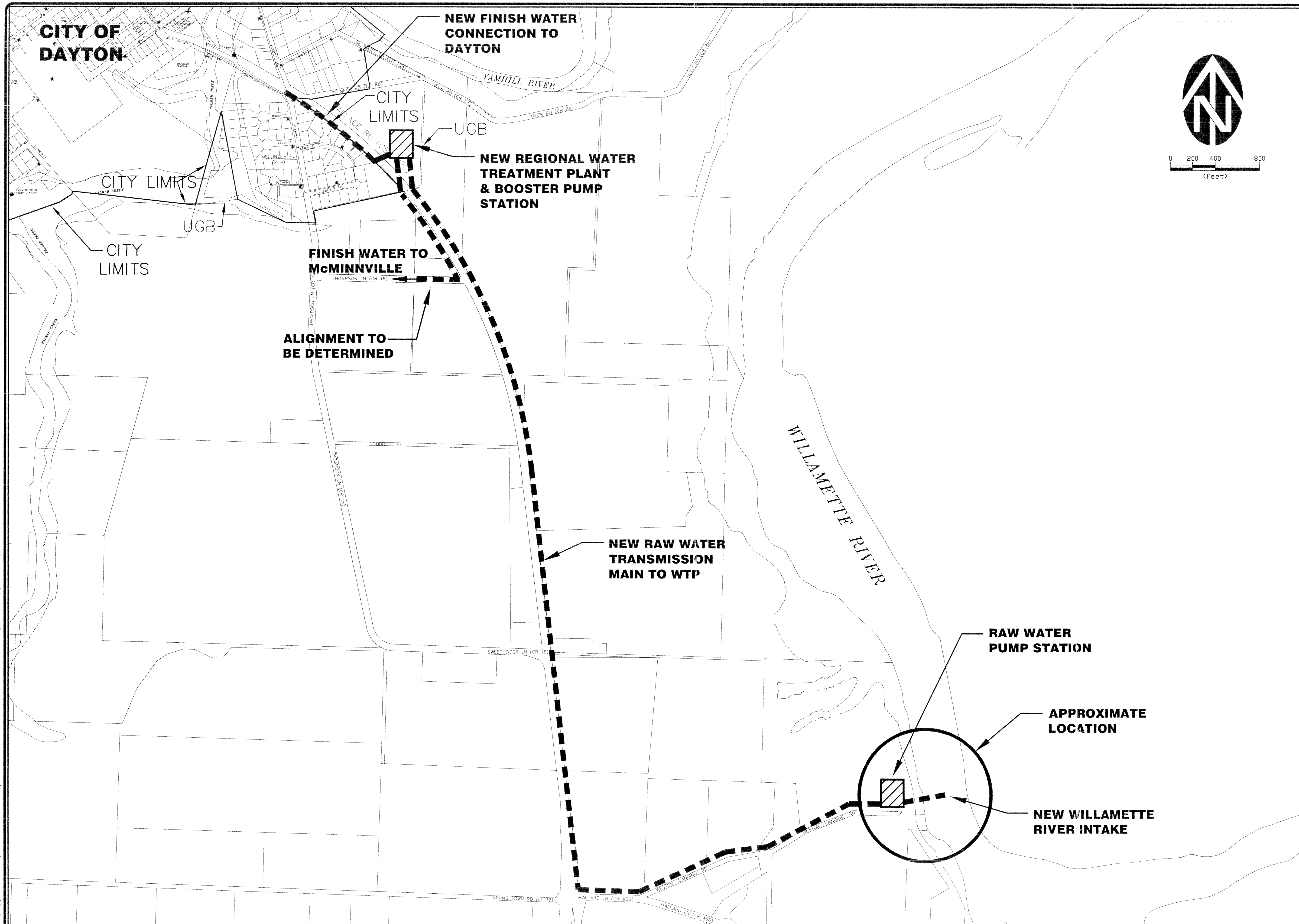
- *Alternative 2*. The second option developed (**Figure 7-4**) consists of siting both the raw water pump station and the new WTP on the river bluff near the Willamette River. A finish water pipeline could be constructed to tie to the Dayton distribution system, with a separate finish water pipeline to McMinnville (alignment to be determined). While this option would consolidate the raw water pump station and the new WTP on a single site, it would involve significant impacts to agricultural lands located outside of the Dayton UGB.

Both new WTP siting alternatives may have other significant advantages and disadvantages that are beyond the scope of this water study, which will need to be analyzed further to determine feasibility.

For either of the alternatives to be feasible, a feasibility study will need to be initiated to determine process for obtaining required Willamette River water rights, and how these water rights would be allocated to the involved municipalities (Dayton, McMinnville, Lafayette and/or others).

Once it is determined the manner in which OWRD will allow water to be withdrawn from the Willamette (ie. new water rights or federal storage water), and the interested municipalities are determined, the size of the pumping, treatment and transmission mains can be determined and preliminary construction cost estimates can could be determined. Since it is unlikely that Dayton will be the lead agency in developing feasibility or pre-design studies to pursue this option, we recommend that the City adopt a resolution formalizing the City's interest in participating in a feasibility study that includes investigation of the various aspects described above, and a copy of this resolution provided to Yamhill County and McMinnville for future reference.

Development of estimates for this regional surface WTP is beyond the scope of this water master plan.



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CITY OF DAYTON, OREGON
DAYTON WATER MASTER PLAN
REGIONAL WTP OPTION
INTAKE PS, TRANSMISSION & WTP LOCATION (ALTERNATIVE 1)

FIGURE
7-3

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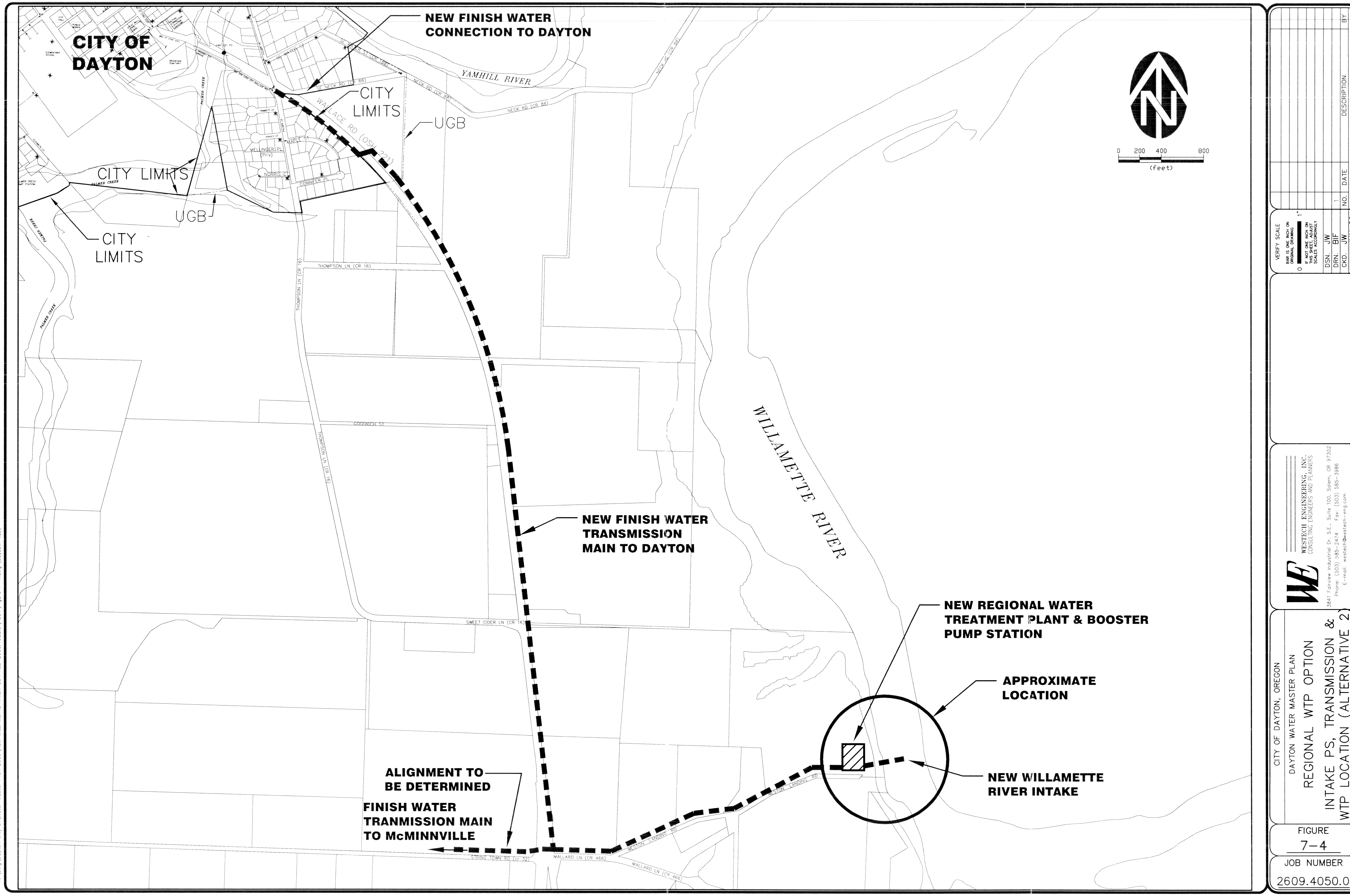

CITY OF DAYTON
NEW FINISH WATER CONNECTION TO DAYTON
CITY LIMITS
UGB
CITY LIMITS
UGB
CITY LIMITS
YAMHILL RIVER
WILLAMETTE RIVER
0 200 400 800
(feet)
NEW FINISH WATER TRANSMISSION MAIN TO DAYTON
NEW REGIONAL WATER TREATMENT PLANT & BOOSTER PUMP STATION
APPROXIMATE LOCATION
NEW WILLAMETTE RIVER INTAKE
ALIGNMENT TO BE DETERMINED
FINISH WATER TRANMISSION MAIN TO McMINNVILLE
CITY OF DAYTON, OREGON
DAYTON WATER MASTER PLAN
REGIONAL WTP OPTION
INTAKE PS, TRANSMISSION & WTP LOCATION (ALTERNATIVE 2)
WESTECH ENGINEERING, INC.
CONSULTING ENGINEERS AND PLANNERS
FIGURE
7-4
JOB NUMBER
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7.6 Recommended Approaches & Improvements

Based on the above discussions, it is clear that the City must proceed with various improvements to the water treatment system in order to provide for long term growth.

Since the various improvements address issues related to different sources, the City may need to proceed with more than one project concurrently. The City should begin implementing these recommendations in the very near future, since the current available firm water supply is very close to the present MDD.

The recommended improvements and studies are summarized in **Table 7-3** (at the end of this chapter).

7.6.1 Water Loss Reduction (Transmission & Distribution Improvements)

As discussed in Section 5.4.7 and Section 6.7.1, the water loss experienced by the Dayton distribution system and the watershed transmission system is higher than desired.

When prioritizing water system improvements, the City should bear in mind that reduction in distribution system leakage also reduces the City's water treatment requirements. Further reduction in water losses from the in-town distribution system and replacement of the watershed transmission main will have the same effect as increasing the source production and treatment capacity available for use by consumers.

Recommendations for distribution system improvements that will reduce water loss (and increase the effective volume available from the City's existing sources) are included in Chapter 8.

7.6.2 WTP Pressure Filter Backwash Improvements

As discussed under Section 7.3.1, the existing WTP needs to be upgraded to ensure that adequate water is available for backwashing the pressure filters (350 gpm minimum). The recommended improvements involve the construction of a 21,000 gallon above grade clearwell structure on the west side of the existing WTP building.

Recommended budget numbers to cover the capital costs for these recommended improvements appear in Chapter 12.

7.6.3 WTP Finish Water Pump Improvements

As discussed under Section 7.3.2, the City will need to add the third distribution pump at the WTP within the planning period. The timing for when the third pump will need to be added is dependent on the extent of the water loss reduction that the City is able to achieve. The City should continue to monitor the peak demands, and schedule the installation of the third pump by no later than 2020.

Recommended budget numbers to cover the capital costs for these recommended improvements appear in Chapter 12.

7.6.4 WTP Fire Pump Improvements

As discussed under Section 4.4.4.3, the fire flows measured in the distribution system are less than would be expected based on the rated flow-head curve of the fire pump provided. While the addition of the secondary transmission main from the WTP (see Chapter 7) is anticipated to help this situation, the City should troubleshoot the cause of this discrepancy what can be done to resolve it in the interim. Since the cause of this situation is not readily apparent, detailed improvement recommendations will be deferred to a separate study. Recommended budget numbers to cover the costs for the recommended study and anticipated improvements appear in Chapter 12.

7.6.5 Watershed Springs Improvements

As discussed under Section 7.4.1.5, the recommended improvements at the watershed springs consist of reconstruction of the spring collection area and replacement of the existing spring boxes with a below grade collection system protected by a 60 mil HDPE cover and soil cap (Alternative 1B). This work is anticipated to be done as a two stage program (beginning with the West (Lower) Spring area, and then proceeding to the East (Upper) Spring area.

While the City is acquiring funding to proceed with Alternative 1B, City Public Works should consider interim improvements to the existing spring boxes (some of the items summarized under Alternative 1A, such as installation of new water-tight lids, backfilling around the existing spring boxes with new compacted bentonite/clay seals, trenching diversion channels around the upper side of the spring boxes etc). This interim work can likely be completed by Public Works crews.

Recommended budget numbers to cover the capital costs for these recommended improvements appear in Chapter 12.

7.6.6 Regional Water Treatment Plant (Willamette River Source)

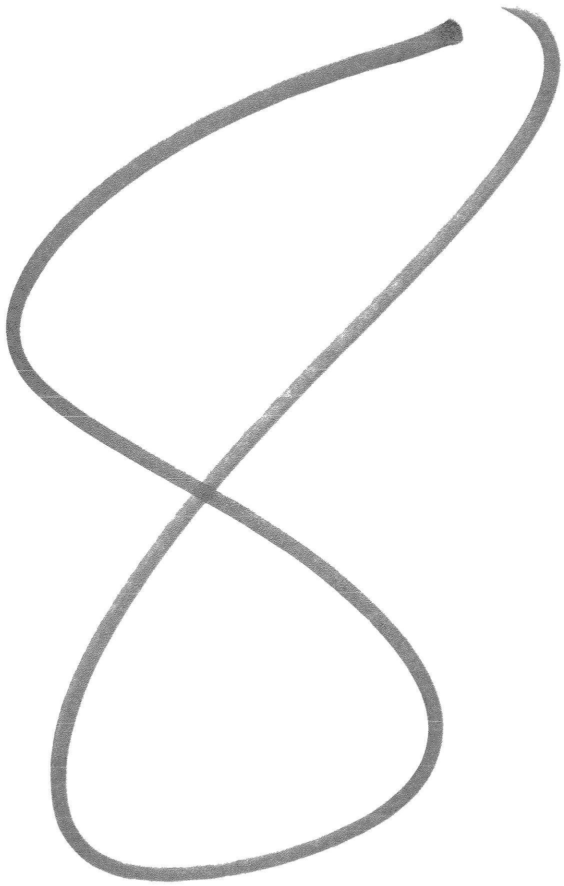
As discussed under Section 7.5, there is potential for a regional surface water treatment facility (drawing water from the Willamette River) to be located at or near the southeast corner of Dayton. Since it is unlikely that Dayton will be the lead agency in developing feasibility or pre-design studies to pursue this option, we recommend that the City adopt a resolution formalizing the City's interest in participating in a feasibility study for such a regional water facility, and a copy of this resolution be provided to Yamhill County and McMinnville for future reference.

7.6.7 Summary of Recommended Treatment Improvements

The following table is a brief summary of the various water treatment improvement recommendations developed by this master plan. For more details on particular projects, refer to the discussions in the body of the study.

Table 7-3 Recommended Water Treatment Improvements & Projects

Project Code	Project
WT-1	Replace steel transmission & distribution lines to increase volume of source water available for consumption (see recommended improvements in Chapter 8)
WT-2	Install clearwell and influent pump station at WTP to ensure adequate backwash flowrates
WT-3A	Add third Dayton distribution pump at WTP
WT-3B	WTP Finish Water Pump Improvements
WT-4	Watershed Spring improvements & chlorination to address potential GWUDI issues (Alternative 1B, Section 7.4.1)
WT-5	Alternative 1C Watershed Spring improvements (only if Alternative 1B is not successful)
WT-6	Adopt resolution of support for regional WTP option adjacent to Dayton.



CHAPTER 8
DISTRIBUTION SYSTEM EVALUATION

Chapter Outline

DISTRIBUTION SYSTEM EVALUATION

CHAPTER 8

8.1 INTRODUCTION

The combination of piping, storage, pump stations, and supporting infrastructure is conventionally defined as a water distribution system. The discussions of this chapter narrow this definition by excluding water storage. Evaluations and recommended improvements to the City's water storage facilities are presented in Chapter 9. Since they are part of the WTP improvements, evaluation of the finished water pumps and the fire pump are included in Chapter 7.

The evaluations of this chapter were derived from the creation and study of a computerized hydraulic model designed to replicate the City's pumps, reservoirs, and distribution network. This model was used to simulate various operational modes, fire flow scenarios, and failure states in order to verify improvement recommendations. These recommendations are presented at the end of this chapter. Capital costs and a prioritized ranking of the recommendations appear in Chapter 12.

8.2 EVALUATION CRITERIA

8.2.1 Sizing and Capacity

The primary purpose of a water distribution system is to deliver the full range of consumer demands and fire flows at pressures suited for the particular use. To accomplish this, the distribution system utilizes a combination of larger transmission mains and networks of smaller distribution mains.

For purposes of this evaluation, transmission mains are defined as larger diameter pipes (12-inches or larger) designed to convey larger flows over longer distances from the point of storage to point of use, or waterlines which are dedicated to convey flows between source and treatment or storage facilities. Distribution mains are comprised of pipes smaller than 12-inches in diameter and provide connectivity throughout the service area. Distribution mains must satisfy both provide both normal consumer domestic demands <u>and</u> fire flows, and thus experience a wide range of operating velocities. Distribution mains are evaluated on their ability to provide fire flow during MDD periods. The City's PWDS require new waterlines to be a minimum of 8-inches diameter for single-family residential areas, and 10-inches or larger for industrial, commercial, and multi-family areas with fire flows above 1500 gpm.

The American Water Works Association (AWWA) recommends a limit of 5 feet per second (fps) for transmission mains and a maximum of 10 fps for distribution mains. The City's PWDS, by comparison, permit a line velocity of 6 fps for ADD conditions and allow a maximum of 10 fps for MDD plus fire flows. Maximum headloss recommendations for transmission and distribution mains are limited to 3 and 10 feet per 1,000 feet respectively. Exceeding these headloss criteria may result in loss of hydraulic conductivity and increased energy costs.

The following standards are recommended to determine water distribution system adequacy.

- Peak hour demands for the entire system must be met with system pressures remaining above 20 psi.
- The system must be capable of delivering the required fire flows to all portions of the distribution system (in combination with the maximum day demand) while maintaining a minimum residual pressure of 20 psi at all service connections.

8.2.2 System Pressure

Pressure is the primary metric for evaluating the ability of a distribution system to deliver water. There are several concepts relating to water system pressure that must be defined for purposes of this discussion.

- *Pressure and Head.* Water pressure (sometimes called head pressure) is directly related to the height to which water will rise in a standpipe at that location. Each psi of water pressure equates to 2.31 feet of water column height in a standpipe (the standpoint can be real or hypothetical). Under conditions of no flow through the pipelines, the water level elevation (in real or imaginary standpipes) will be the same at all points in a pressurized distribution system (to visualize this concept, imagine a lake, where under no-flow conditions the water level elevation is the same at all points). Therefore as the elevation of the ground surface changes, the height of water column above that same point will change proportionately, and the pressure will change (conceptually, as the lake bottom elevation goes up or down, the water depth (and water pressure on the bottom) at that point also changes).
- *Pressure Change with Elevation.* Based on the pressure/head concept noted above, water pressure (ie. head pressure) will increase with decreasing ground elevation, and will decrease as the ground elevation increases.
- *Static Pressure.* As noted above, pressure in a pipeline is constant at all points in that pipeline ONLY when there is no flow through the pipeline, AND when the elevation remains the same at all points. As noted above, in a real distribution system, the static pressure increases or decreases with changing ground elevation.
- *Head Loss.* As water flows through a pipe, pressure decreases along the length of the pipe due to friction losses between the water and the pipe walls. Similar to dry friction, water friction and turbulence along a pipeline walls results in energy losses from the moving object (ie. flowing water), with the energy loss being manifested as reduced pressure. When the flow stops, the friction losses also stop, and so the system returns to static pressure levels.
- *Dynamic Pressure.* The dynamic pressure (sometimes called residual pressure) is the pressure measured at a point in the distribution system under some defined flow condition. While the *static* pressure in the distribution system remains relatively constant at a given point, the *dynamic* pressure (ie. the actual observed pressure) can change dramatically. Therefore, pressure at any given point in the distribution system generally decreases as demand for water (and flow velocity) increases.

As was noted in Section 4.4.4.2, both the distribution pumps at the WTP and the watershed PRV station are set to maintain a target static pressure of approximately 60 psi (as measured at the WTP), with static pressure experienced by individual users varying based on the ground elevation throughout town (typical static pressures range from 60 psi at the WTP to ±85 psi on the east side of the pedestrian bridge). The actual (dynamic) pressure experienced by individual users also varies based on the flowrate through the pipelines between the point of supply (ie. WTP pumps or the PRV station) and the user.

Periods of heavy fire flow demand depress system pressures significantly. ODWP standards (OAR 333-061-0025) stipulates that water suppliers must maintain a minimum pressure of 20 psi to all service connections at all times, including times of peak fire flow demand. Fire flows are typically modeled concurrent with the maximum day demand.

The Oregon Plumbing Specialty Code (OPSC) defines 80 psi as the maximum unregulated pressure for domestic water services (OPSC 608.2). System pressures above this range are to be reduced with a pressure regulating valves on the individual water service. This plan recommends maintaining normal operating pressures at their current levels.

8.2.3 Fire Protection

Table 5-7 in Section 5.6 details the fire flow standards adopted by the City. These standards will be utilized in the fire flow calculations of this chapter to ensure that the distribution system is suitably sized and configured to reliably deliver the required fire flows to all areas within the city limits.

8.2.4 Deficiency Categories

In general, distribution system deficiencies fall into several general categories. Many elements of the water system may be experiencing more than one of these problems at the same time. These categories will be used to identify the deficiencies associated with particular elements of the system in the discussions of this chapter.

- *Lack of Capacity*. Undersized pipes cannot deliver peak water demands or fire flows. Although the water system may have capacity to deliver domestic flows, it is often unable to convey larger flows that may be required in an emergency. Pipes in this category have excessive headloss and create flow restrictions. This problem should be addressed either by increasing the size of the existing waterline or constructing new waterlines.
- *Lack of Facility*. Problems in this category are caused by the absence of a waterline, valve or hydrant, or inadequate looping to provide redundancy or reliability. In such cases new components should be constructed in order to increase system reliability or to simplify system operations.
- *End of Useful Life*. This category of problem is the result of old, damaged, or worn out pipes. The most common examples of this problem are leaky pipes and broken valves or hydrants. The correction of these problems requires the replacement or reconstruction of the failing component.

8.3 HYDRAULIC MODEL DEVELOPMENT

8.3.1 Model Methodology

Computerized modeling of water distribution systems is a proven and effective method for simulating and analyzing the performance of a distribution system under a wide range of operational and hydraulic conditions. A properly constructed and calibrated model permits a robust evaluation of the distribution system and often allows the designer to replicate and evaluate hydraulic scenarios that are too difficult or costly to perform in the real world. Such scenarios are useful to determine the overall strength of a distribution system and to identify weaknesses that require remediation. The evaluation of future pipeline sizes and routing can also be economically performed to assure that the expansion of the distribution system occurs in an optimized fashion.

The modeling software used for this project was WaterCAD , a commercial modeling software package developed by Bentley Systems Incorporated. This software was utilized to calculate the distribution of flow throughout the distribution network and to quantify flow rates, pressures, headlosses, reservoir levels, and well pump operating points under various consumer demand patterns and fire flow scenarios.

The general methodology used in the modeling process was to examine the existing distribution grid during various demand and fire flow scenarios. Pressure, flow, or connectivity deficiencies were used to formulate improvement scenarios to remedy the problem. These scenarios were evaluated to determine their efficacy.

8.3.2 Model Development

At the most basic level the hydraulic model consists of nodes and links. Nodes represent the various elements of the system including water sources, pumps, storage tanks and pipe intersections. Links predominantly represent pipes and define the relationship between each node. The creation of the model utilized information from a variety of sources. The City's existing distribution system maps were used as a base in the early building stage and this information was supplemented with information from record drawings, previous engineering studies, field reconnaissance, and discussions with City staff.

Model pipe elements were constructed based on the diameter, length and material type of each pipe. Hazen-Williams roughness factors were assigned to the pipes based on the pipe material type and age. These initial roughness factors were later modified in the calibration process as described in Section 8.3.3. Model nodes were placed at pipeline intersections, near fire hydrant locations, and in locations to simulate clustered water service connections. The model nodes were populated with topographic information to ensure that elevation differences across the planning area were properly accounted for.

Existing pumps were replicated to perform at the currently utilized levels and set points. Reservoirs were constructed with tanks to match the physical geometry and assigned elevations to match the existing facilities.

Fire hydrants were modeled using a hydrant element that accounted for a typical 6-inch diameter hydrant lateral with an average length of 15 feet.

As in industry standard, the pipe network was simplified or 'skeletonized' to a certain degree. This process eliminated or combined short pipe segments, consolidated pipe junctions and eliminated small diameter pipes with insignificant connectivity. These simplifications were carefully conducted to ensure integrity and hydraulic equivalency with the physical distribution system.

Once the distribution network was created, the water demands established in Chapter 5 were allocated to specific nodes across the system. Demands from the larger water users were selectively modeled as discrete demands at the locations designated on the billing records.

8.3.3 Model Calibration

Model calibration is the process of adjusting model input data and structure so that the simulated hydraulic output sufficiently mirrors observed field data. Model calibration is typically an iterative process whereby the model is executed to calculate flows and pressures for all or a series of nodes in the distribution system. These results are then compared to physical measurements taken at those same nodes. Pipe roughness factors are then adjusted to increase or decrease pressures and flows and the model is re-run. This process continues until the model results converge with the measured data to an acceptable level of accuracy.

The calibration process for this model utilized flow and pressure data extracted from a set of hydrant flow tests performed by Westech and City personnel. Fire flows, as well as static and residual pressures were measured at approximately 10 hydrants throughout town. After calibration, the error between the model and the field fire flow measurements averaged of less than 10%. This level of calibration falls within the conventional standards for calibration.

8.3.4 Model Scenarios

The calibrated model was used to investigate a number of hydraulic scenarios in the distribution system. These scenarios were evaluated using a combination of steady state and dynamic simulations. The simulations produced a snap-shot of hydraulic conditions at a fixed period in time.

In particular, the hydraulic scenarios investigated include the following under existing conditions.

- Existing peak hour demands.
- Existing maximum day demands.
- Fire flows to each model node in combination with the existing maximum day demand (without WTP in operation).

The model was also used to simulate various improvements to the distribution system to identify the most cost-effective solutions to the system deficiencies. Simulations with several combinations of the improvements were analyzed.

The results from the computer simulations were used to develop a list of long-range improvements required to address system deficiencies and to serve the City through the planning period. Since transmission pipelines are not well suited for incremental expansion, it is most cost effective to size the pipes for fully built-out conditions. Steady state simulations of the future system at buildout were performed to determine the required size of the transmission mains. The following simulations were performed.

- Peak hour demands at build-out.
- Peak day demands at build-out.
- Fire flows to each model node in combination with the existing maximum day demand at build-out.

8.4 DISTRIBUTION SYSTEM ANALYSIS

The evaluation of the existing distribution system was performed to identify system deficiencies and possible remedies for the portion of town currently served by the existing distribution grid, as well as improvements to serve future growth-related needs. This section presents improvements for the distribution system broken down into three categories comprised of transmission, distribution and fire flow improvements. **Table 8-1** at the end of the chapter summarizes these improvements.

8.4.1 Transmission Analysis

The City has two existing transmission mains (excluding the dedicated lines between the wellfield wells and the WTP), as follows.

- *Ferry Street Transmission Main.* This line consists of the 14-inch and 12-inch waterlines that begin at the WTP and extend along Ferry Street to 4th Street. There is no existing transmission main to serve the area south of Palmer Creek.

- *Watershed Transmission Main.* This line consists of an 8-inch steel waterline that begins at the watershed springs and extends to the intersection of Ferry Street and 1st Street.

The Ferry Street transmission mains were installed as part of the 2004 water project, and consist of ductile iron pipes installed along the north side of Ferry Street (see Chapter 4). Ferry Street in this area is also Hwy 223, and is an ODOT right-of-way. Although the original design drawings for the 2004 water project show the Ferry Street transmission main extending as a 12-inch line to 2nd Street, as-built noted from the contractor appear to show that it was reduced to 8-inch at the 4th & Ferry intersection. Improvements to extend the transmission network to serve the area south of Palmer Creek are discussed later in this chapter.

Although the Ferry Street transmission main appears to be adequately sized to convey the required flows within the specified velocity or pressure design parameters to most of the downtown core, it appears that the existing fire pump may not be properly configured to be able to send design fire flows through this transmission main to the various distribution mains (ie. it does not appear to be a transmission main capacity issue).

Deficiencies identified in the existing transmission system include the following general categories.

8.4.1.1 Transmission Deficiencies.

Deficiencies identified in the existing transmission system are summarized below, and along with recommended improvements to address these deficiencies.

8.4.1.1.1 Secondary Transmission Line from WTP

The existing flows from the WTP (including from the fire pump) are entirely dependent on the transmission main along Ferry Street. The only significant distribution line connecting to the transmission main between the WTP and 9th Street is the 8-inch main along Flower Lane. However, this main connects to the 6-inch loop along Church Street, and provides little (if any) fire flow redundancy if the Ferry Street transmission line is damaged or out of service for any reason. Any problems with the transmission main along Ferry Street will result in the loss of fire flows to essentially the entire city.

Therefore, it is recommended that a new 12-inch transmission main be constructed from the discharge side of the WTP vault and connected to the distribution grid at the start of the proposed new distribution/transmission line along Ash Street (east of Flower Lane). This will serve a number of purposes, including the following.

(1) Provide a redundant flow path with capacity to serve peak domestic flows for the entire City

(2) Provide a redundant path for fire flows to the High School/Middle School campus,

(3) Provide a WTP transmission pipeline that is not located within the ODOT right-of-way), and

(4) Provide a path for the future fire flows to the transmission main serving the UGB area north of Hwy 18. Without this transmission line, the construction of the 10-inch distribution loop along Ash Street (Flower to 9th) as discussed below would still be limited by the capacity of the existing 8-inch pipe along Flower Lane. It appears that this redundant transmission main from the WTP is necessary to convey adequate fire flows to the future 10-inch transmission line north of Hwy 18 (discussed below).

Figure 8-1 shows one conceptual alignment for the new transmission main, which could be routed to the northeast corner of the WTP/reservoir property, and then through an easement to and along the west side of Flower Lane to Ash Street. Construction in an easement along Ash Street will avoid conflicts with the several existing pipelines already located in Flower Lane (14" Lafayette transmission line, 8-inch Dayton waterline, the 2 & 3-inch raw water lines from Flower Lane & 11th Street wells, the gravity sewer line and existing storm drain lines). Although this will require easements across private property, the first property to be crossed is currently undeveloped, and the second property is the First Baptist Church (where there appears to be adequate space to route the waterline along the Flower Lane frontage between the building and the street).

8.4.1.1.2 Watershed Transmission Line

As previously noted, the transmission main from the watershed is well beyond the end of its useful life (ie. 1930s vintage steel line), as well as being significantly undersized, and should be replaced with a new 12-inch minimum size waterline as soon in the planning period as feasible. The new PRV station constructed in 2009 was constructed and valved to allow for connection of this future 12-inch transmission main. There are several segments of this transmission main, summarized as follows. This entire waterline (with the exception of a segment near McDougal Corner) is a 1930s vintage steel line that is well beyond its design life, and has experienced multiple leaks and repairs in recent years.

Watershed Springs to Watershed Reservoirs (within City watershed)

A new 8-inch PVC waterline from the springs to the reservoirs is anticipated. An easement (or long-term lease) for the portion within the lease lands (encompassing the spring and filter area) will be required prior to construction of this waterline, unless the City purchases or otherwise acquires this property.

Watershed Reservoirs to McDougal Wells

Although a short segment of this transmission line below the reservoir is located on City property, the majority of this steel waterline is located on private property, over which the City has not been able to locate a recorded easement. The alignment of this waterline between the City watershed and the McDougal Well sites is not entirely certain (although it appears to be along the westerly boundary of Tax Lot 4309-1600, 17185 NE McDougal Rd). The City needs to verify the existing waterline alignment, clear brush & trees along its length to allow for inspection and maintenance, and obtain easements to allow for installation of a replacement waterline. As noted above, this segment and all of the following segments (to 1st & Ferry) are to be replaced with a new 12-inch waterline.

McDougal Wells to Hwy 99W

This portion of the waterline appears to be located within the McDougal Road right-of-way (county road), although the exact alignment is unknown. It is anticipated that the new waterline will be installed within the County right-of-way.

Hwy 99W Crossing & McDougal Corner

The portion of the waterline crossing Hwy 99W is apparently a 1930's vintage steel line, while the portion from the south side of the highway to the south side of the railroad tracks (to Kreder Road) is 8-inch ductile iron installed in 1992. If the new waterline is installed prior to construction of the Newberg-Dundee Bypass, it is recommended that it be installed in an easement outside of the ODOT right-of-way, even if the City has to pay for such an easement. This will minimize the risk of the City being required to pay to relocate the waterline when the bypass is constructed (as would happen if it is installed within the ODOT right-of-way).

Kreder Road to PRV Station

This portion of the waterline is located within the Kreder Road right-of-way (county road). It is anticipated that the new waterline will be installed within the County right-of-way.

PRV Station to 1st & Ferry (Yamhill River Crossing)

The existing waterline across the Yamhill River is suspended on the existing wooden pedestrian bridge. As noted in Section 4.6.4.1, the existing Yamhill River pedestrian bridge (at the easterly end of Ferry Street) is near or at the end of its design life, and will require increasingly expensive repairs in the coming years. This bridge is not expected to last through the planning period of this water study.

ODOT's Tier 2 Draft Environmental Impact Statement for the Newberg-Dundee Bypass (released in June 2010) includes a plan to replace the existing pedestrian bridge with a new vehicular bridge at the same location (ie. to connect Kreder Road and Ferry Street across the Yamhill River). However, the timeframe for this new bridge is highly uncertain, as it is dependent on federal funding. Since the City expects to have to replace the existing waterline early in the planning period, it does not appear feasible to wait until the new ODOT Ferry Street bridge is complete in order to replace the river crossing.

As discussed in Section 4.4.1.1, OBEC Consulting Engineers estimated the ongoing maintenance and upgrade costs to potentially exceed $700,000 from approximately 2011 to 2030. OBEC also recommended that the City place a high priority on maintenance or replacement of the bridge due to the critical function it serves the City (this recommendation predates ODOT's selection of the option for a new Ferry Street bridge). OBEC's bridge replacement cost estimates range from $3.8 million to $6.9 million depending on the bridge type (this assumes a replacement pedestrian bridge).

Given the cost associated with ongoing bridge maintenance and/or replacement of the pedestrian bridge, and the anticipated timeframe for the new ODOT bridge, the following alternatives were developed.

(1) One long-term option would be to eliminate reliance on a pedestrian bridge entirely, and install a new water transmission line under the Yamhill River using directional drilling. Preliminary estimates for directionally drilling under the Yamhill River (for both the water and sanitary sewer) are approximately $1,107,000. Assuming that the pedestrian bridge would be removed during the 20 year planning period, this option would require both waterline and the sanitary sewer force main to be bored under the river. This option would eliminate the ongoing maintenance associated with exposed pipelines suspended from bridges, and is substantially less expensive than a new pedestrian bridge.

(2) Another option evaluated was to route the water line to the Hwy 18 bridge north of town and hang the transmission main from this bridge. In discussions with ODOT about this option, ODOT indicated that this existing bridge was not designed to accommodate additional pipelines of this type, and that this option may require upgrades to the existing bridge to ensure that its

integrity is not compromised. A detailed structural evaluation of the existing bridge to determine the feasibility of installing the waterline on the bridge is beyond the scope of this water study.

The use of the Hwy 18 bridge crossing alternative will require the installation of approximately 3,400 feet of additional 12-inch transmission main (ie. from Kreder Road to Hwy 18, along Hwy 18 to Dayton interchange, and then south along 3rd Street to Oak Street). Based on estimated pipeline costs from Chapter 12, this additional transmission main would be about $500,000, plus the cost of the bridge crossing work itself. Given that this approach would require that the existing sewer force main (which also hangs on the pedestrian bridge) also be relocated to the Hwy 18 bridge, and the fact that the additional sewer force main length would be even greater than the water main (±4800 feet), this option appears to be significantly more expensive than the directional drill alternative.

If the existing Hwy 18 bridge is replaced as part of the ODOT work on the Newberg-Dundee Bypass, the new bridge could conceivably be designed to accommodate the water & sewer pipelines. However, since the timeframe for the transmission main replacement is expected to precede the ODOT bypass work, and given the cost for the additional pipelines, installation on the new bridge is not considered to be a viable option, and is not considered further.

It is not considered feasible to install the new larger diameter watershed transmission main on the existing pedestrian bridge, and the new transmission line replacement will most likely precede the construction of the new ODOT Ferry Street bridge. Therefore, we recommend that the City plan for installing the transmission main under the river via directional drilling.

8.4.1.1.3 Palmer Creek Bridge Crossing (Hwy 221 Bridge)

The existing waterline is inadequately size to convey required fire flows to the south portion of Dayton. As discussed below, the waterline crossing the highway bridge will be fed by two waterlines connecting to the Ferry Street transmission main (existing 8" & new 10" mains). A 12-inch transmission main is recommended for the Palmer Creek bridge crossing (from Mill Street to Neck Road). In order to minimize the work within the ODOT right-of-way, it is recommended that the new transmission main be routed along the old 3rd Street right-of-way to the north edge of the cemetery, and then west to the ODOT right-of-way, then north across the bridge and along the west side of Hwy 221 to Neck Road.

As with the Hwy 18 bridge alternative, a preliminary structural evaluation of the existing Hwy 221 Palmer Creek bridge should be performed to verify that the new transmission main can be suspended from the existing bridge without major (ie. expensive) structural bridge upgrades. Such a structural evaluation is beyond the scope of this water study, but merits further consideration by the City. We recommend that the City retain an engineering firm specializing in bridge construction and modification to conduct this preliminary structural evaluation (similar to the firm that did the pedestrian bridge structural evaluation & design work). If the preliminary structural evaluation indicates problems with utilizing the existing bridge, the only other feasible alternatives would be to either install the pipeline down the ravine and bore a casing under the creek, or install the pipeline via directional drilling (ie. likely drill beginning near Neck Road and exiting near Mill Street).

For purposes of this master plan, it is assumed that the new pipeline can be suspended under the existing bridge.

8.4.1.1.4 Transmission Line to area south of Palmer Creek bridge

Although the 2004 water project included a new transmission along Ferry Street from the WTP to 4th Street, it did not include a transmission line to provide higher volume flows to the area south of Palmer Creek. A 12-inch transmission main (or multiple pipes with equivalent capacity) is recommended to provide these fire flows.

City records indicate that the 12-inch Ferry Street transmission main (along the north side of Ferry Street to 4th) was stubbed south across Ferry Street on the east side of 4th Street, but was not continued south along 4th. The waterline east along Ferry was reduced to 8-inch at 4th Street. An 8-inch line also runs south along 3rd to from Ferry to Mill Street (these lines were installed in 2004 to replace the old 2" & 6" steel lines in this commercial core). The old 4" steel waterline along 4th Street (south of Ferry) was not replaced.

There are two feasible routes available to extend a larger diameter transmission main from the 4th and Ferry intersection to the intersection of 3rd & Mill (where it can connect to the proposed transmission main across the highway bridge over Palmer Creek). These routes are along Ferry and 3rd Street, or along 4th and Mill Streets.

There is already a new 8-inch PVC waterline along Ferry Street & 3rd Street, and replacement along this alignment would involve utility work within the ODOT rights-of-way (ie. both Ferry & 3rd). Therefore, it is recommended that the new transmission main be installed along 4th Street and Mill Street. In terms of hydraulic capacity, the combination of a <u>new</u> 10-inch waterline along 4th and Mill, in conjunction with the <u>existing</u> 8-inch waterline along Ferry & 3rd, is roughly equivalent to a single 12-inch transmission main.

Since this will minimize the work required in the ODOT right-of-way and minimize replacement of existing PVC lines, this approach is recommended.

8.4.1.1.5 Transmission Lines to UGB Area North of Hwy 18

When the area north of Hwy 18 is annexed into the City and developed, water mains adequate to provide fire flows to this area will be required. Since the Dayton Comprehensive Plan Map designates all of the land north of Hwy 18 as residential, it is assumed that fire flows for this area will be 1500 gpm or less for single family development. If any of these areas are rezoned for commercial, industrial or multi-family development, larger diameter transmission mains across Hwy 18 will likely be required. As shown on **Figure 8-1**, this area is to be fed by a looped system in the future, with mains cross the highway at 3rd Street and at Fletcher Road.

8.4.1.1.6 Regional Water System Transmission Lines

An evaluation of transmission main alignments and sizes associated with potential future regional water system is beyond the scope of this study. It is assumed that a separate analysis will be performed when the County or a group of cities decides to move forward with this option.

8.4.2 Distribution System & Fire Flow Analysis

This section evaluates the adequacy of the distribution system to deliver domestic water and fire flows to all service areas, as well as an evaluation of the adequacy of system looping, etc. Looped distribution systems are more desirable than branched systems because, coupled with sufficient valving, it allows flows to be routed around the failure of any single distribution pipe. This provides service redundancy and facilitates repair work while keeping outage areas as small as possible. A looped configuration also provides multiple water paths to any specific point in the system, which reduces velocities along any given flow path and increases the system's ability to provide high volume fire flows (assuming the looped lines are adequately sized).

As noted in Section 4.5.2, the Dayton Service Level encompasses essentially all of Dayton within the City Limits, with typical pressures ranging from about 60 to 80 psi. The distribution system is fed from two directions (ie. from the WTP pumps and from the watershed PRV). Deficiencies identified in the existing distribution system are summarized below (as noted under Section 8.2.4, many elements of the water system may be experiencing more than one of these deficiencies.

8.4.2.1 Capacity Deficiencies (Fire Flows)

The City's distribution grid is generally adequate to provide an adequate level of service for domestic flows, there are a number of pipelines need to be upsized to accommodate fire flow requirements (see **Figure 8-1**), as summarized below. As noted above, ODWP rules require public water suppliers maintain a minimum pressure of 20 psi at all service connections at all times, including during fire flow events. The current distribution system is incapable of delivering maximum design fire flows while maintaining 20 psi at all service connections.

8.4.2.1.1 North of Church Street & West of 9th Street (Palmer Addition area)

The area north of Church Street and west of 9th Street is served by undersized (and often antiquated) waterlines. A new 10-inch waterline along Ash Street would not only provide fire flows to this portion of town, but would act as the transmission line for the future 10-inch transmission loop north of Hwy 18. This will also replace the old 4" steel waterline along 9th Street, and the old 6" waterlines along Church Street and Ash Street, as well as the small galvanized line connecting from just north of Flower Lane to Ash Street.

8.4.2.1.2 4th Street (Ferry to Mill) & Mill Street (4th to 3rd)

While most of the old waterlines in the core area of town were replaced in 2004, it did not include replacement of the old 4-inch steel line along 4th Street south of Ferry Street. As noted under the transmission main discussions above, the new waterline along 4th Street will need to be a minimum of 8-inch diameter. However, upsizing this line from 8 to 10-inch will result in significant overall savings to the City, since it will avoid the need to upsize relatively new 8-inch lines along Ferry and 3rd Street to accommodate transmission requirements to the south portion of town.

8.4.2.1.3 3rd Street (Ferry to Church)

A new 10-inch line along 3rd Street in this area will provide fire flows for the structures along this street, as well as providing for transmission flows to the line proposed for the area north of Hwy 18.

8.4.2.1.4 5th Street (Oak to Church

Although the waterlines north and south of this block were replaced in 2004, this existing 6-inch line also need to be replaced in order to strengthen the grid in this area and assure adequate fire flows.

8.4.2.1.5 Church Street (east of 2nd)

This dead end stretch of street is currently served by a 1¼-inch line, with no hydrant coverage. A new 8-inch waterline and hydrant is needed to serve this area.

8.4.2.2 Lack of Facility and Looping Deficiencies.

The City's distribution grid generally provides an adequate level of looping in the main core area of town, but not in some of the outlying areas. There are several general areas in town where waterline do no existing or looping is not considered adequate to meet either water service requirements or to meet reliability standards as summarized below (see **Figure 8-1**).

8.4.2.2.1 Palmer Creek crossing @ 1st Street

In order to provide looping and redundancy for the water system south of Palmer Creek, the existing 4" steel waterline between the south end of 1st Street and the north end of Palmer Lane needs to be replaced. This line should be replaced with a new 8-inch diameter waterline, either along the existing alignment (1st Street to the end of Palmer Lane), or from the end of Palmer Lane to Water Street (at the County boat landing park). It is assumed that this pipeline can be installed either by trenching down the ravine and installing a bore casing under the creek, or install the pipeline via directional drilling (ie. likely with drill beginning at the end of Palmer Land and exiting on Water Street north of the boat ramp). For purposes of this study, trenching down the ravine and boring under the creek is assumed. Easements will be required for either alignment.

8.4.2.2.2 Main Street (2nd to 3rd Street)

A new 8-inch line along Main Street from 4th to 3rd Street will provide looping for this area of town, and provide a line to service the lots on the north side of Main (these lots are currently served by water services that run along Main Street).

8.4.2.2.3 Warmscombe Drive (Bell to Willert)

When development occurs on the parcel between the two street sections, this 8-inch loop will need to be completed.

8.4.2.2.4 Dayton RV Park Area

When additional development occurs to the east of the existing RV park, the existing 6" waterline serving the RV park and hydrants is not adequate to serve as a fire service for this area. A larger diameter line will likely need to be extended from Kreder Road and tied back to the existing RV park line. Based on the current industrial zoning for this area, it is assumed that this will be a 10-inch or 12-inch line to provide fire flows (depending on type of development).

Since the majority of the fire flow storage is located at the WTP (in the main service level), the distribution & fire line to serve the industrial land will need to be connected to the Kreder Road transmission main downstream of the existing PRV station, and connected to the existing loop through the RV park.

8.4.2.3 End of Useful Life

As existing pipes and valves near the end of their useful life, they should be replaced before failure occurs. Depending on several factors, it can be reasonably assumed that even new waterlines (PVC or ductile iron) will have a 75 year service life. Most of the City's distribution grid was replaced as part of the 2004 water project, or has been installed in conjunction with development within the past 15 years.

The City still have significant quantities of galvanized steel or wrapped steel pipe. These pipes were typically installed from the 1930 to the early 1960s. They have exceeded their service life and must be replaced as soon as feasible (see **Figure 8-1** & **Figure 8-2**), as summarized below. This plan recommends replacing all these pipes as early in the planning period as possible.

8.4.2.3.1 Main Street (7th to 8th Street)

While most of the old waterlines in the core area of town were replaced in 2004, it did not include replacements of the old 2" galvanized steel waterline along Main Street between 7th Street and 8th Street. A new 8-inch waterline should be installed, along with a new fire hydrant at 7th & Main to meet recommended hydrant spacing.

8.4.2.3.2 Main Street (3rd to 2nd Street)

The old 6" steel line along this street needs replacement by a new 8-inch waterline (old steel line west of this point was replaced in 2009).

8.4.2.3.3 4th Street (Ferry to Mill) & Alder Street (4th to 3rd)

The 4th Street line replacement was discussed above. The old 2" galvanized steel line along Alder Street needs replacement with a new 8-inch waterline.

8.4.2.3.4 Fletcher Road & Foster Road Waterline

Fletcher Road is currently served by a small diameter galvanized steel line (1930s vintage) that crosses Hwy 18. This line serves areas within the UGB but outside the current City Limits. Replacement of this line was discussed under the transmission main evaluation above.

8.4.2.3.5 Thompson Road Waterline

Thompson Road is currently served by a small diameter galvanized steel line (1930s vintage) that crosses the ravine at the south end of Palmer Lane. This line serves areas outside of the current UGB. A new 8-inch waterline along Hwy 18 to Thompson Road is anticipated, with a smaller diameter rural waterline along Thompson Road to maintain existing services. The existing steel line across the ravine will be abandoned in place.

8.4.2.3.6 Palmer Lane (Norris Court to Conifer Place)

City records indicate that the old steel waterline may still exist between Norris Court and Conifer Place. This old line needs to be replaced with a new 8-inc PVC waterline when the old line crossing the ravine is abandoned.

8.4.2.3.7 McDougal Road Waterline (west of McDougal Corner)

Although the size and exact alignment of this small diameter waterline is unknown, local knowledge of this line indicates that it is an old galvanized line along most of its length. We are not aware of any records showing that this is a private waterline, although there was reports of a meter vault where this line connects to the waterline transmission line (although the City has not been able to locate such a vault in this area). It is expected that this line will need to be replaced during the planning period.

8.4.2.3.8 Leaks and Faulty Connections

There were numerous locations where the 2004 water distribution improvements were reconnected to existing old steel waterlines. One result of the falling out between the City and the engineering firm responsible for that project was that as-built drawings for the constructed improvements were never prepared, particularly at reconnections between the old and new systems. In some cases, there were waterline connections to the old system that were unknown to the City, which resulted in several cases in old lines remaining in service in parallel with new waterlines (the City has located and disconnected a number of these old lines since 2004, mainly when leaks surfaces in locations where waterlines were not supposed to be). There have also been issues with leaks at connections between the old steel pipes and the new PVC or ductile iron pipes installed in 2004. We recommend that the City budget money each year to deal with these faulty connections as they are discovered in the future.

8.4.2.4 Upgrades to Water Services, Private Waterlines and Backflow Devices

Due to the age of the Dayton water system, many of the water service connections date back many decades. As mainlines are reconstructed or replaced, current City policy requires that the water service lines between the mainline and the water meter be replaced as part of the project. The portion of the water service line beyond the water meter is the responsibility of the property, with a major incentive to repair leaks because the property owner is responsible to pay for any water that passes through the meter.

Many of the water services were also installed prior to the implementation of the State Cross Connection Control Program (ie. backflow prevention). As previously noted, the City currently has a State certified cross connection control specialist on staff.

8.4.2.4.1 Overlength Service Lines prior to Water Meter

There are several locations where there are old service lines that run for extended distances prior to the water meter. Most of these over-length services (of which we are aware) are classified as private services, with the property owner responsible for repair of leaks or maintenance. However, since there are extended length of service line (often crossing private property or running parallel along streets) without a meter to flag the higher flows due to leaks, corrosion and defects in these service lines can exist undetected for months or years before they are discovered and fixed.

Current City standards and policy require water meters at the edge of the right-of-way or easements in order to ensure that leaks from private services (and the water wasted due to such leaks) is the responsibility of the property owner (ie. the property owner is responsible for all water that passes through the meter, including water leaks from private service lines beyond the meter).

Locations of concern (that we are aware of) include the followings. This list is not all inclusive, and the same recommendations apply to other long water service lines which are not specifically listed.

Foster Road Services

There are two properties on Foster Road served by very long service lines (13957 Foster Road & 14700 Foster Road). The first has a water service along Foster Road that is in excess of 1000 feet in length before the meter (connection point is at Fletcher & Foster). The second has a water service that crosses Hwy 18 that is in excess of 1100 feet in length before the meter (connection point is on Ash Street between Willert & Bell). We recommend that the City relocate the meters for these services to a point adjacent to the mainline.

Watershed Services

There are a number of properties between the City watershed and Hwy 99W that have City water services. The City should carefully evaluate these water services and ensure that the meters are located as close to the City mainline as possible, and at a location that ensures that adequate disinfection CT values are provided in conformance with the Groundwater Rule. In particular, the meter for 4195 NE Breyman Orchards Road is located adjacent to the house, while the water service is connected to the watershed transmission line to the west of this location. Due to the vintage of this water service, the location of the mainline tap is unknown. We recommend that the City relocate the meter for this service to a location adjacent to the watershed transmission main, accessible for meter reading, and verify that the tap is downstream of the chlorination point (just downstream of the McDougal Wells). The length of the water service from the chlorination building to the existing house is approximately 1600 feet.

McDougal Corner Services

There are no known record drawings or information on the alignment of waterlines and service lines serving the commercial properties at McDougal Corner (ie. the Dayton Bypass addresses). However, some of the meter are set at locations that appear to be relatively distant from the mainlines serving them. We recommend that the City relocate the meters for these services to a point adjacent to the mainline(s).

8.4.2.4.2 Private Waterlines & Rural Waterlines

In some cases, there are private waterlines serving multiple water uses or properties. Current City policy requires a master water meter on these lines near the mainline connection, and that payment for maintenance, repairs and any water leakage downstream of the meter is the responsibility of the property owners.

McDougal Road Waterline

As previously noted, the status of the small diameter waterline along McDougal Road (west from McDougal Corner) is unclear. At one point this was reported to be a private waterline with a master meter, but the City has not been able to find the location of the supposed master meter. We recommend that the City research City records and consult with the City Attorney in order to make a definitive determination on the ownership and status of this waterline. If it is a private rural waterline, the maintenance and repair responsibilities of the property owners served should be formalized, as well as the responsibility for the cost for any leaked or unaccounted for water.

Regardless of whether or not this is a private or a public waterline, we recommend that the City install a master meter on this waterline at the connection point to the transmission main (assuming that the existing master meter cannot be located). This will serve as a valuable tool for monitoring the condition of this line and detecting leaks early (ie. by comparison of master meter readings with the individual meter readings).

Thompson Road Waterline

As previously noted, Thompson Road is served by a small diameter waterline that is scheduled for replacement in the near future. As with the McDougal Road line, we recommend that the City install a master meter on this waterline at the intersection of Hwy 221 & Thompson Road, with the primary purpose of this meter being to allow monitoring of the line and early leak detection in the future. If it is decided not to install the master meter at this time (due to the line being new), the meter box and meter setter assembly should be installed to allow for installation of a water meter in the future without having to excavate and cut the line or interrupt service.

Fletcher Road Waterline

As previously noted, Fletcher Road is currently served by a small diameter waterline that will not be replaced until the area north of Hwy 18 is annexed into the City and redeveloped. We recommend that the City install a master meter on this waterline (at Fletcher & Howard Johnson Loop), with the primary purpose of this meter being to allow monitoring of the line and early leak detection in the existing mainline (particularly since this line is well beyond the end of its normal service life).

8.4.2.4.3 Cross Connection Control

As existing water services without adequate backflow preventers are identified, we recommend that the City continue their policy and program of requiring the property owners to install backflow devices meeting state standards, and to test these devices annually as required. The following is not an inclusive list.

Schools, Commercial & Industrial Services

We recommend that the City allocate adequate budget and staff resources to continue the ongoing evaluation of existing water services serving schools, commercial and industrial facilities to identify services that predate current backflow standards, and which do not have backflow devices meeting current standards.

McDougal Corner Services

As the in-town services, we recommend that the City continue to evaluate the commercial water services in this area, to ensure that water commercial uses requiring backflow prevention have such backflow devices installed on their respective services.

Recommended improvements to address these distribution system deficiencies are summarized in the following section.

8.4.3 Water Loss Evaluation

A detailed evaluation of the water losses (leakage & unaccounted-for water) from the distribution system is contained in Section 5.4.7. As noted under Section 6.7.1, reduction of distribution system losses will be equivalent to developing new sources that increase the available water supply. As noted under Section 9.4.1, reduction of distribution system losses will decrease the storage volume required for equalization and standby storage. As noted in Chapter 7, reduction of distribution system losses will reduce the requirements on the water treatment system. As such, the City should consider the reduction of water losses as a major priority, as it will result in significant benefits to all four areas of the water system (source, treatment, distribution and storage).

8.4.4 Water Age Evaluation

Water quality is emerging as a major concern for many utilities. An important indicator of water quality is the age of the water in the pipes, also known as the hydraulic residence time. Based upon a survey of

800 utilities, an AWWA publication reported an average distribution system retention time of 1.3 days, with a maximum retention time of 3.0 days. Examples of much longer retention times in portions of water supply systems have been reported. Water retention time is primarily a function of water demand, system operation, and system design. Water quality can change as it moves between sources of supply and treatment to the consumer. While there is no set requirement for minimum or maximum water age, utilities should be cognizant of their system's water age because elevated water age can lead to taste and odor complaints, increases in temperature, increases in disinfection byproducts, decreases in disinfection residual, and other water quality issues. The appropriate water age for any particular system is a function of the age and material of the pipes, the type of disinfection utilized (chloramines versus chlorine), and the amount of organic matter in the system. The configuration of the City's existing distribution system does not raise concerns about excessive water age.

8.4.5 Distribution Improvements for Developments

Outside of large diameter transmission lines, the expansion of the distribution grid to serve new developments is anticipated to occur in areas selected by developers. In such cases the City's PWDS provide a sound basis for ensuring that a properly sized and looped grid is constructed around the larger diameter transmission mains. Beyond this, localized distribution improvements will be evaluated on a case by case basis. For the above reasons, these projects are not included in the water systems project list.

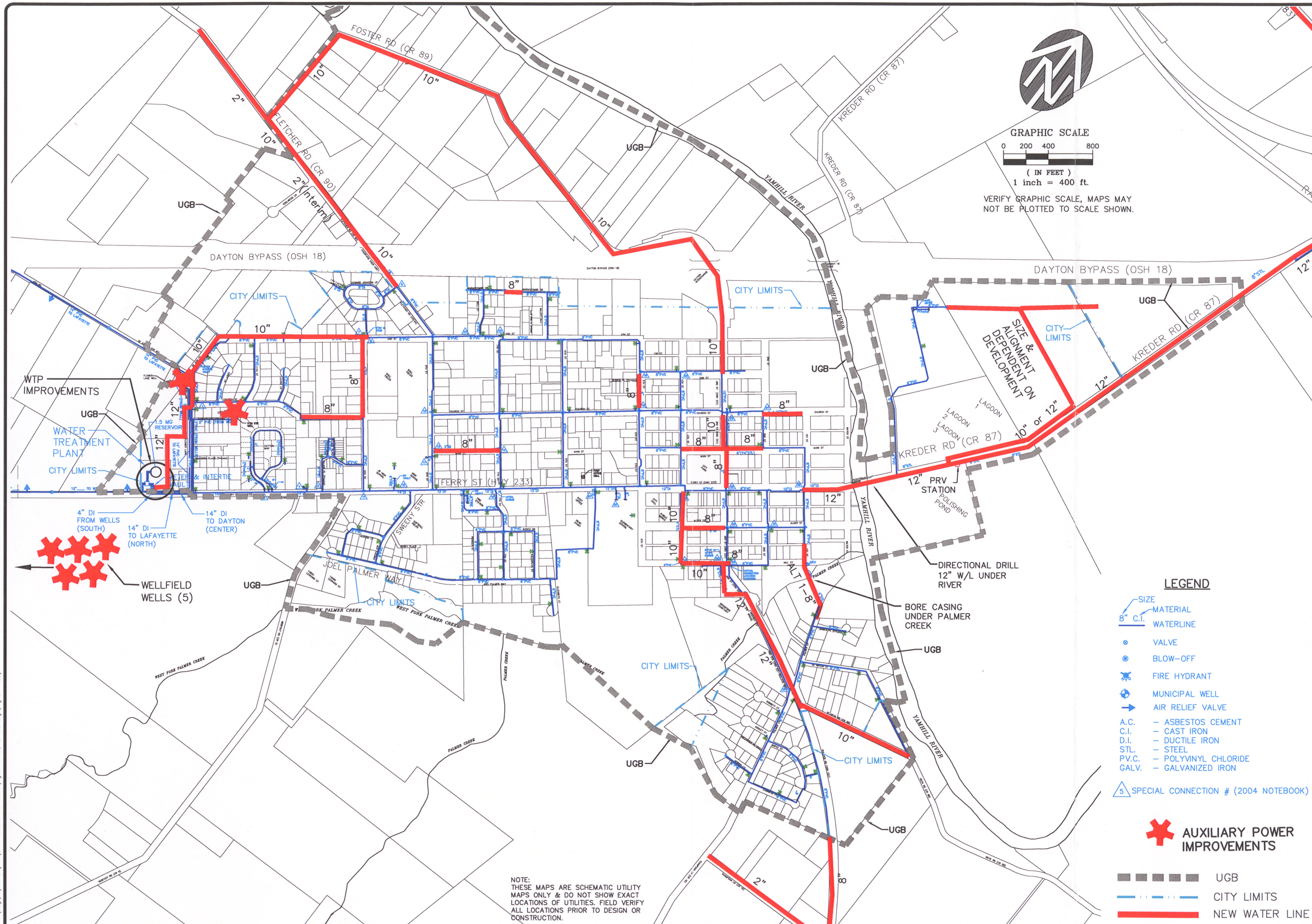
8.5 SUMMARY OF RECOMMENDED DISTRIBUTION IMPROVEMENTS

Several improvement projects have been identified based on the hydraulic analyses presented in this chapter. Transmission line improvements make up a significant portion of the work. Distribution projects have been identified to improve a combination of capacity and age deficiencies. Other improvement projects have been identified to strengthen fire flows, system redundancy and provide additional connectivity between the WTP and distribution grid. These improvement recommendations are summarized in **Table 8-1** and graphically depicted on **Figure 8-2** and **Figure 8-3**. This table does not include replacement of all of the small diameter lines or all waterline lines that may be required as part of individual developments.

Recommended budget numbers and prioritization of these projects is presented in Chapter 12.

Table 8–1 Recommended Transmission/Distribution Improvements & Projects

Project Code	Location	Extg φ (inch)	New φ (inch)	Length (feet)
	Transmission System(*generally listed east to west*)			
T-1	Watershed springs transmission main (springs to watershed reservoirs)	8 Stl	8	800
T-2	Watershed transmission main (watershed reservoirs to McDougal Rd)	8 Stl	12	4,200
T-3	Watershed transmission main (McDougal Rd @ wells to PRV station)	8 Stl/DI	12	6,800
T-4	Watershed transmission main (PRV Station to 1st/Ferry). Install new main under Yamhill River by directional drilling	8 Stl	12	1,500
T-5	4th Street transmission main (4th/Ferry to 4th/Mill)	4 Stl	10	700
T-6	Mill Street transmission main (4th/Mill to 3rd/Mill)	8 PVC	10	450
T-7	Hwy 221 Palmer Creek bridge transmission main (Mill Str to Neck Rd)	6/8 PVC	12	1,650
T-8	Fletcher Road/Foster Road transmission main	1½ GI, 0	10	8,200
T-9	Ash Street transmission main (Flower/Church to Ash/9th)	6 PVC	10	2,100
T-10	WTP secondary transmission main (WTP to Church & Flower)	0	12	1,100
	Distribution System (*generally listed west to east*)			
D-1	9th Street (Ash to Church)	4 Stl	8	750
D-2	Church Street (9th toward Laurel)	6 Stl	8	600
D-3	Main Street (8th to 7th)	2 GI	8	600
D-4	Warmscombe Drive	0	8	200
D-5	5th Street (Oak to Church)	6 PVC	8	350
D-6	Main Street Replacement (2nd to 3rd)	6 Stl	8	390
D-7	Main Street Replacement (3rd to 4th)	0	8	360
D-8	3rd Street (Church to Main)	4 Stl	10	350
D-9	3rd Street (Main to Ferry)	2 GI	8	350
D-10	Church Street (west of 2nd)	1¼ PVC	8	350
D-11	Alder Street (4th to 3rd)	2 GI	8	400
D-12	Palmer Creek crossing (Option 1: Palmer Ln to 1st)	4 Stl	8	800
D-13	Palmer Creek crossing (Option 2: Palmer Ln to Water Str, directional drill)	0	8	1150
D-14	Neck Road (Hwy 221 to Water Street)	0	10	1100
D-15	McDougal Road rural waterline	2 PVC/GI	2	3,800
D-16	Thompson Road rural waterline	1½, 1 GI	8, 2	2,500
D-17	Fletcher Road rural waterline (interim repair till annexation)	1¼ GI	2	3,000
D-18	East Dayton Industrial Area waterline	0	12	4,200
D-19	Over-length service modifications (Foster Rd, Watershed, McDougal Rd)	-	-	-
D-20	Master meters on rural waterlines (Fletcher Rd, McDougal Rd, Thompson)	-	-	-
D-21	Commercial services at McDougal Corner	-	-	-



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REVISIONS

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DSN. JW
DRN. LS
CKD. JW
DATE: OCT 10

WESTECH ENGINEERING, INC.
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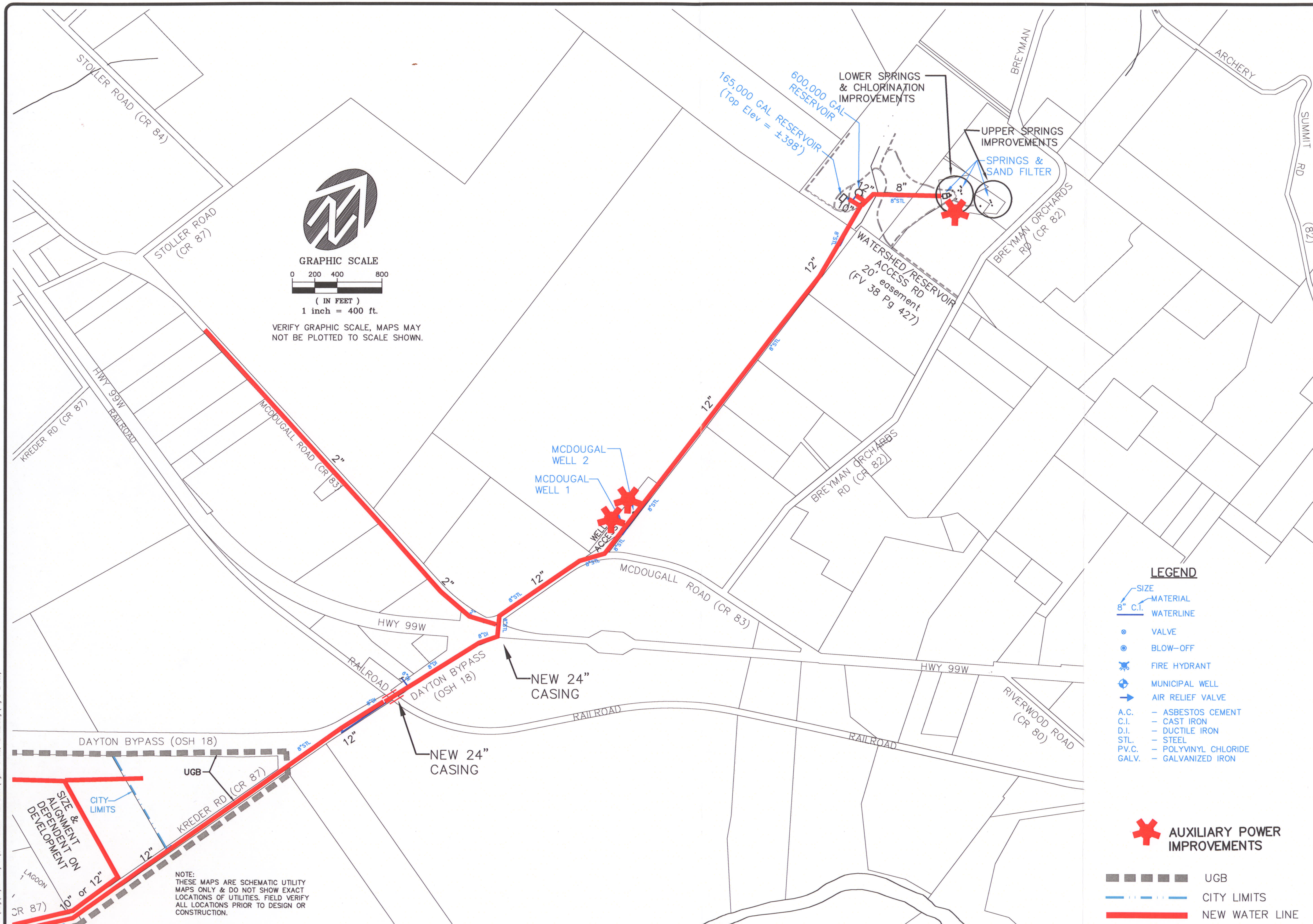
CITY OF DAYTON, OREGON

RECOMMENDED TRANSMISSION & DISTRIBUTION IMPROVEMENTS

SHEET
FIG. 8-1

JOB NUMBER
2609.4050.0



Dec 17, 2010 - 3:30pm R:\Dwg\Dayton City of\Water Master Plan 2009\Figure 8.1&2, 12.1&2.dwg (Fig 8-2 tab)

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CITY OF DAYTON, OREGON

RECOMMENDED TRANSMISSION & DISTRIBUTION IMPROVEMENTS

SHEET
FIG. 8-2

JOB NUMBER
2609.1006.0

9

CHAPTER 9

WATER STORAGE EVALUATION

Chapter Outline

WATER STORAGE EVALUATION

CHAPTER 9

9.1 INTRODUCTION

The emphasis of this chapter is shifted from the existing water storage inventory of Chapter 4 and placed on the hydraulic design and performance of existing and future reservoirs.

Although closely integrated with the overall water distribution system as discussed in Chapter 8, this report presents water storage as a separate discussion to focus on several key issues unique to this subset of the distribution system. Recommended budget numbers to cover the capital costs for the recommendations presented in this chapter appear in Chapter 12.

9.2 EVALUATION CRITERIA

Per ODWP rules, engineers are responsible for planning and designing stable and durable reservoirs that meet demands and protect the quality of stored water. Some of the evaluation criteria used utilized in the analysis and recommendations of this chapter are discussed below.

9.2.1 Storage Volume Categories

The primary function of water storage is to provide a reserve of water to equalize daily variations between supply and consumer demand, to serve fire-fighting needs, and to meet system demands during an emergency interruption of supply. The overall storage within a system can be divided into the several categories. The following sub-paragraphs define these storage allocation categories. Evaluation of how these categories apply to the Dayton water system are discussed in Section 9.3.

9.2.1.1 Operational Storage

Storage volume within the upper elevation of a tank used by the system operators to control the start and stop of the sources or pumps which fill the reservoir. The operational storage volume is not counted as part of the "effective storage" volume (discussed below), since emergency conditions are as likely to begin when water level is at the bottom of the operational storage range as when it is at the top of the range. The overall elevation difference (storage volume) required by the pump control system is determined by the type of instrumentation, the number of sources or pumps that fill the reservoir, and operator preferences.

9.2.1.2 Equalization Storage

Storage that is utilized to meet short term consumer demands that exceed the production capacity of the supply sources. As previously discussed, water demands vary throughout the day based on the water use patterns of the community, as well as over multiple days. Demand fluctuations are influenced the relative mix of residential, commercial and industrial use, as well as by the weather. Commercial and light industrial use tends to be relatively constant through the normal daytime hours (with light to no use at night), while residential use fluctuates between relatively high flows in the morning, low flows during the day, higher flows in the evening, and lowest flows at night. The equalization storage volume required is typically determined by one of two methods, as follows.

(1) The first method utilizes a percentage of the maximum day demand (MDD), generally 20 to 40%.

(2) The second method is to determine the deficit between the peak hour demand (PHD) and the available supply for a determined duration, typically 2 to 4 hours.

9.2.1.3 Standby Storage (Emergency Storage)

Storage that is required to meet demand during emergency situations such as power outages, supply pipeline failures or natural disasters (often termed as emergency storage). The amount of emergency storage provided can be highly variable depending upon the reliability and diversity of supply sources, an assessment of risk and the desired degree of system reliability.

For water systems with multiple sources, it is often set as the difference between 48 hours at ADD and the capacity of the supply sources which are "*continuously available to the system*" with the largest single source out of service (ie. "2 times ADD" minus "2 times daily capacity of all *continuously available* sources except the largest source").

Sources that are "*continuously available to the system*" means sources that comply with <u>all</u> of the following.

(1) Source is <u>either</u> gravity feed to the storage reservoir, <u>or</u> is equipped with adequate and functional pumping equipment, and the source is provided with adequate and functional treatment equipment (if required).

(2) The pumping and/or treatment equipment is regularly used (or is exercised regularly to ensure its integrity, if not regularly used).

(3) Water is available from the source year-round (only source capacity during lowest flow period is to be counted).

(4) The source activates automatically based on pre-set parameters (ie. reservoir level, water system pressure, or other conditions).

(5) Pumped source provided with on-site auxiliary backup power equipment (with an automatic transfer switch), or there is a separate dedicated mobile generator for each source which is equipped with a manual transfer switch.

Sources which do not comply with these requirements cannot be reasonably considered to be available during a major emergency, including a system wide power outage, particularly if sources are located in rural areas where restoration of power may take some time.

9.2.1.4 Fire Suppression Storage

Storage that is required to satisfy the largest design fire flow demand in the system. Fire storage volume is calculated by multiplying the design fire flow rate by its required duration. For a given fire flow, the Oregon Fire Code stipulates a required design duration (OFC Table B105.1).

9.2.1.5 Dead Storage

The volume of unusable water stored in a reservoir that either cannot be withdrawn, or which lies below the minimum recommended operating level of the reservoir (ie. the minimum level required to maintain required suction pressure on pumps, etc). Dead storage that is not available without violating the recommended operating conditions of distribution or fire pumps cannot be counted for the purposes of water storage planning (even if it is physically possible

9.2.1.6 Pumped Storage

Stored water that lies below the normal hydraulic head level of the distribution system (ie. in ground storage tanks). This is water that must be pumped into the distribution system or into an elevated tank before it is available in the distribution system. If the pumps (which move this stored water into the distribution system) are not available during an emergency, the pumped storage water is also unavailable.

9.2.1.7 Effective Storage

As noted above, the total volume is a reservoir often does not equal the effective volume available to the water system. The effective storage volume is defined as the reservoir volume below the bottom of the operational storage level, minus any dead storage. In the case of Dayton, a significant percentage of the WTP storage reservoir is currently classified as dead storage, as discussed in Section 9.3.

9.2.2 System Pressure

In most municipal distribution systems, the service pressure is determined by the elevation of the free water surface in the storage reservoirs serving the system. This is the case for the watershed service level (based on the watershed reservoir), as well as pressure maintained on the Dayton service level from flow through the PRV station. Service pressures begin with available static pressure created by elevated reservoirs and are reduced en-route to the consumer by friction losses in the pipe network. In such systems supplied by tanks set above the service level (either elevated tanks or tanks set on a hill), the overflow elevation of the reservoir is a critical design factor, as it directly controls the static pressure of the system.

For systems with ground storage tanks, service pressure is maintained by pumps which discharge into the distribution system (see discussions under Section 4.4.4). For systems which rely on pumps to maintain system pressure, the overflow elevation of the water reservoir is not a critical design factor.

Service pressures at the point of delivery typically range from 40 to 80 psi. Pressures below this range cause inaccuracies in customer meters and flow reductions during periods of high demand whereas pressures above this range can damage domestic plumbing systems. The Oregon Plumbing Specialty Code (OPSC) defines 80psi as the maximum unregulated pressure for domestic service. Service pressures above this range must be reduced with a pressure regulating valve. This plan recommends maintaining the operating pressure range in town between 40 and 80 psi.

9.2.3 Water Quality

There are no specific regulatory requirements for water turnover rates in storage facilities, but industry sources suggest a complete water turnover be accomplished every 3 to 5 days . Experiences with reservoirs with cement-based internal surfaces suggest a slightly higher turnover rate of 5-7 days .

Historically water storage facilities are operated at near full levels to maintain system pressure and maximize storage volumes for emergencies; however, in times of non-emergency the large storage volumes reserved for firefighting can create water quality problems. Degraded water quality in storage facilities is frequently the result of under utilization and poor mixing during filling cycles. As water ages, there is also a greater potential for disinfection by-product (DBP) formation.

In summary, excessive water age can result in a diverse set of problems ranging from the loss of residual disinfectant, problems with bacterial proliferation or regrowth, increased formation of DBPs, taste and odor problems, as well as temperature and pH instabilities.

The Dayton WTP reservoir includes a recirculation pump system to maintain mixing and to maintain chlorine residual levels in the reservoir water.

9.2.4 Reliability of Pumped Storage

Clearly, the provision of emergency backup power and redundant pumping is critical for systems that rely heavily on pumped storage. The Dayton WTP is provided with an auxiliary power generator to ensure the operation of the distribution service pumps, and the fire pump is powered by propane (with an on-site storage tank).

9.2.5 Redundancy

A lack of redundancy with regard to storage facilities is most frequently encountered when a reservoir must be taken off-line for cleaning, inspection or maintenance. While some of these procedures can be accomplished with a facility on-line, others (such as internal recoating) cannot. It is therefore recommended that the planning and construction of reservoir improvements provide the City operators with the flexibility to maintain these important facilities where feasible.

Storage redundancy is also critical in the wake of natural disasters. As discussed in previous chapters, seismic events present the largest natural disaster threat to these structures.

9.3 WATER STORAGE ANALYSIS

Effective storage volume, system pressures, water quality, and redundancy are some of the factors used to evaluate the suitability of existing water storage reservoirs, and provide recommendations for new reservoirs.

9.3.1 Storage Volume Assumptions

As previously noted, the only the effective storage volume can be counted when evaluating whether a water system meets the water storage goals. The overall storage within a system can be divided into the several categories. The following sub-paragraphs define these storage allocation categories.

9.3.1.1 Operational Storage Assumptions

For the purposes of this report, the operational storage range was assumed to be the upper 2 feet of each of the existing reservoirs. This equates to approximately ±104,000 gallons in the 94'φ WTP tank (±7% of total tank volume), and ±90,000 gallons in the two watershed tanks (68'φ steel, 61.7'x39' concrete), which is ±12% of the total volume of both watershed tanks. For purposes of planning, it was assumed that operational storage will account for 10% of the volume of any new reservoirs in the future.

9.3.1.2 Equalization Storage Assumptions

The discussion below summarizes the assumptions under each of the methods used to establish equalization storage volume.

Since the future status of the City's supply sources is highly uncertain, it is not feasible to utilize the equalization storage method that relies on the difference between the PHD and the source capacity. Therefore, for purposes of this analysis the first method was utilized (equalization storage was set at 25% of MDD).

9.3.1.3 Standby Storage Assumptions

Since the City's system has multiple supply sources, the City should (in theory) be able to set standby storage as the difference between 48 hours at ADD and the capacity of the supply sources with the largest single source out of service. However, since the only sources that can be counted as a credit to discount the amount of standby storage are those that are *continuously available to the system*, the only source that can be counted at present is the watershed springs, although those may not be able to be counted if the City is not able to address the GWUDI concerns raised by the ODWP (the existing sand filters are not adequate treatment to address the GWUDI or surface water treatment standards.

For purposes of the storage evaluation, the following scenarios were considered related to source availability.

(1) Scenario 1: Current conditions, springs available. Under this scenario, the City has only a single source *continuously available to the system*. Since the springs are thus the largest single source (which must be considered as if it were out of service), no source credits are available to offset the 2 day ADD standby storage requirement.

(2) Scenario 2: Current conditions, springs not available. Without the springs, the City has no sources *continuously available to the system*, with no source credits to offset standby storage requirements (same as Scenario 1).

(3) Scenario 3: Wellfield wells & McDougal wells provided with backup power, springs available. Under this scenario, the City sources would offset the standby storage, by accounting for the springs and all wells except for Well 3 (largest single source at 90 gpm), and excluding 11th Street & Flower Lane wells (which are assumed to not have generators under this scenario).

(4) Scenario 4: Wellfield wells & McDougal wells provided with backup power, springs not available. The source offset credit for this scenario is the same as Scenario 3 but without the springs.

(5) Scenario 5: All wells (except Post Office Well) provided with backup power, springs available. The source offset credit for this scenario accounts for all sources except for Well 3.

(6) Scenario 6: Same as scenario 5, except assuming a new finish water pump station will be constructed to recapture the dead storage in the WTP reservoir (after the end of the study period). The source offset credit for this scenario is the same as Scenario 5. This is presented for reference only.

Based on the design year ADD of 0.517 MGD (**Table 5-6**), the required standby storage volume is projected to be 0.952 MG at the end of the planning period (prior to applying any source credits). As shown in **Table 9-1** below, adding generators at the wells in order to utilize them as a source credit (for storage calculations) makes a tremendous difference on the overall storage requirements (see also tables under Section 9.3.2).

Table 9-1 Effect of Source Credit on Standby Storage Requirements

Storage Scenario	Total Standby Storage Required (2 x ADD at end of planning period) (MG)	Source Credit [1] (2 days production with largest single source out of service) MG (gpm)	Standby Storage Required with Source Credit (MG)
1. Current Conditions, springs available	1.034	0	1.034
2. Current Conditions, springs not available	1.034	0	1.034
3. Auxiliary power at Wellfield & McDougal Wells, springs available	1.034	0.544 (210)	0.490
4. Auxiliary power at Wellfield & McDougal Wells, springs <u>not</u> available	1.034	0.428 (165)	0.606
5. Auxiliary power at <u>all</u> wells, springs available	1.034	0.674 (260)	0.360
6. Auxiliary power at <u>all</u> wells, springs available, new finish water PS	1.034	0.674 (260)	0.360

(1) Assumes flowrates from all wells as noted on Table 6-1, excluding flows from Lafayette wells (Wells 2, 4 & 50% of 5), with watershed springs at summer flowrates of 45 gpm, and largest single source (Well 3 @ 90 gpm) out of service. Assumes sources operate 90% of time over 2 day period.

9.3.1.4 Fire Suppression Storage Assumptions

As discussed in Chapter 5, this report utilizes the design fire flows established by the City's PWDS. The design fire flow condition is a 4,000 gpm event with a duration of 4 hours, which equates to a total fire flow volume (FSS) of 960,000 gallons.

9.3.1.5 Dead Storage Assumptions

For Dayton's system, there are three sources of dead storage as follows (alternate values listed for the same tank are <u>NOT</u> additive, and the greatest value controls).

(1) The first applies to both steel reservoirs (WTP & watershed), and is the water between the reservoir floor and the height of the outlet pipe silt ring (based on City records, 6" for the WTP reservoir, 10" for watershed steel tank). This equates to approximately ±26,000 gallons in the WTP tank (94'φ), and ±22,600 gallons in the watershed steel tank (68'φ).

(2) The second consists of the fact that the elevation of the pump intakes for the Dayton distribution pumps (and the fire pump, discussed below) are higher than the floor of the reservoir (ie. if the water level drops too low, there is not enough head pressure left to drive water to the pump intakes with enough pressure to avoid cavitation at the impeller). The finish floor of the WTP was set 0.3 feet above the floor of the reservoir (building floor of 161.5' versus tank floor of 161.2'). The pump intakes for the distribution pumps are approximately 2½ feet above the building floor level.

Based on an assumed maximum of 500 gpm, headlosses through the 20", 12" & 10" suction piping, and the 4" pump suction riser to the pump intake, the recommended NPSH to minimize cavitation (ie. double the $NPSH_R$, see discussion below), the water level in the reservoir should not be drawn below a point ±8 feet above the eye of the fire pump impeller (ie. ±10.5' above the tank floor). Although the pumps will still operation at water levels below this point, cavitation damage to the impellers may result (as noted above, $NPSH_R$ is a point at which cavitation bubbles are occurring and beginning to significantly reduce the pump capacity.

The dead storage depth required to maintain adequate suction pressure to the Lafayette transfer pumps is similar to that required for the Dayton distribution pumps. Since the Lafayette transfer pumps are slightly higher head, they may required slightly more NPSH, but the difference will not be significant, and will be less that that required by the Dayton fire pump (discussed below).

For reference, $NPSH_R$ (Net Positive Suction Head Required) is defined as the suction head condition at which the cavitation bubbles at the impellor reach the point where cavitation-induced flow-blockage reduces the pump output head by 3% (and also the point beyond which the rate of cavitation blockage and potential damage accelerates), which means that while the pump may operate at $NPSH_R$ (although at a lower efficiency), cavitation damage may still result from operation under these conditions. The $NPSH_A$ (Net Positive Suction Head Available) should always be greater than $NPSH_R$ by a margin great enough to minimize or avoid formation of cavitation bubbles on the impellor vanes. There is currently no single standard defining an exact value for the NPHS margin required (ie. margin between NPSH $NPSH_R$ and $NPSH_A$) for pumps in general, although engineering guidelines (ie. ANSI/HI 9.6.1) do stress the importance of this margin and that it varies depending on the operating conditions of the pumps.

In concept, $NPSH_R$ is an operating point for a pump that is analogous to the "redline rpm" operating point for a vehicle engine. Although redline is the point beyond which catastrophic engine damage or failure is expected to occur in a very short period of time, it unrealistic to expect that routinely operating an engine at redline (or anywhere near redline) will not greatly shorten the engine life. Similarly, while a pump may be able to tolerate a rare period of operation close to the $NPSH_R$ point, routine operation at or near the $NPSH_R$ point will increase the risk of cavitation damage and premature failure of the pump impellor and/or volute.

(3) The third relates to the elevation of the Dayton fire pump (similar to the issues with the distribution pumps). It should be noted that for the fire pump, the maximum flowrates are higher (and the suction headlosses are correspondingly higher), the pump intake pipes are mounted slightly higher, and the $NPSH_R$ limitations are therefore slightly more restrictive. The center of the intake pipe to the fire pump is approximately 2½ feet above the building floor level (±3 feet to the top of the intake pipe).

Based on an assumed maximum of 4,000 gpm, headlosses through the 20" suction piping, the 16" pump suction riser, and 16x10" suction reducer, the estimated NPSH (Net Positive Suction Head) margin to minimize cavitation (ie. double the $NPSH_R$), the water level in the reservoir should not be drawn below a point ±10 feet above the eye of the fire pump impeller (ie. ±12.5' above the tank floor). Although the pumps will still operation at water levels below this point, cavitation damage to the impellers may result.

This results in recommended dead storage in the WTP reservoir (94'φ) of ±415,000 gallons (10.5' depth) for operation of the distribution service pumps (and the Lafayette transfer pumps), and ±650,000 gallons (12.5' depth) for operation of the fire pump. It should be noted that this recommendation is to minimize long term cavitation damage to the pump impellors, and does not mean that the reservoir level cannot be drawn below the level listed for the fire pump *under short term emergency conditions* (as long as it is only an intermittent, short-term condition). However, in no case should the water level be drawn lower than the dead storage limit for the distribution pumps (ie. ±10.5' above the reservoir floor), to avoid cavitation damage to the distribution pump impellors (and severe cavitation on the fire pump if were called to run under these conditions).

For purposes of this report, the bottom 10.5 feet of the WTP reservoir (415,000 gallons) is considered to be dead storage. This equates to ±28% of the total reservoir volume, or ±30% of the reservoir volume after discounting the operational storage as noted above. For reference, the fire pump dead storage would account for ±43% of the total reservoir volume, or ±46.5% of the reservoir volume after discounting the operational storage as noted above.

As noted in Section 4.2.2, the City of Lafayette paid 25% of the cost of the WTP to account for its use as a clearwell for the Lafayette transfer pumps. This appears to have been based on the dead storage volume in the reservoir that is required for the operation of the Lafayette transfer pumps.

9.3.1.6 Pumped Storage Assumptions

Since the WTP (and the associated distribution pumps) are provided with a permanent on-site auxiliary power generator, the pumped storage is considered to be available for use for purposes of the storage evaluation. Since all pump storage is available to the system, this category will not be considered further.

9.3.1.7 Effective Storage Assumptions

In the case of Dayton, a significant percentage of the WTP storage reservoir is currently classified as dead storage, as noted above. This dead storage (along with the operational storage) was discounted and not included in the storage volume evaluation and recommendations below. As shown in the tables under Section 9.3.2, construction of a new finish water pump station (with below grade canned vertical turbine pumps) will allow the City to utilize most of the existing dead storage in the WTP reservoir, which makes a tremendous difference on the overall storage requirements.

9.3.2 Storage Volume Evaluation

The total recommended storage in the system is the sum of operational, equalization, fire, and emergency storage (while discounting any dead storage). The first step in evaluating the need for additional storage is to evaluate the volume of existing storage that is available.

Discounting the operational storage and dead storage as noted above, the effective volume of the existing Dayton reservoirs is as listed in **Table 9-2** below.

Table 9–2 Effective Storage Volume, Existing Reservoirs

Existing Reservoir	Total Storage (gallons)	Operational Storage [(1)] (gallons)	Dead Storage (gallons)	Effective Storage (gallons)	% of Total Storage Available
WTP Steel Reservoir	1,500,000	±104,000	±415,000	981,000	65.4%
Watershed Concrete Reservoir	165,000	±54,000	0	111,000	67.3%
Watershed Steel Reservoir	600,000	±36,000	±22,600	541,400	90.2%
Totals	2,265,000	194,000	437,600	1,576,400	69.6%

(1) Assumes normal operating range of reservoirs (pump start & stop calls) consists of the upper 2 foot of each reservoir.