Section 7

Treatment System Evaluation and Recommendations

SECTION 7
TREATMENT SYSTEM EVALUATION AND RECOMMENDATIONS

7.1. Introduction & General Evaluation Criteria

This section develops and evaluates alternatives to adequately convey raw wastewater from the pump stations to the treatment plant as well as alternatives to treat and dispose of the projected flows and loads throughout the planning period. A wide range of alternatives were evaluated as part of the planning effort. This section begins with an analysis of the various forcemains that convey wastewater from the pump stations to the WWTP. The forcemain analysis was included in this section because the recommended forcemain improvements are strongly influenced by the location of the treatment facilities.

This section addresses the following key questions:

- What are the existing forcemain deficiencies?
- What are the existing treatment system deficiencies?
- What forcemain and treatment system components are likely to become deficient during the planning period?
- How shall the existing and projected deficiencies be corrected?

The existing and projected forcemain and treatment system deficiencies are presented along with a set of basic alternatives, or tools, for addressing each of the individual deficiencies. The basic alternatives are assembled into sets of primary alternatives that each address all of the existing and projected treatment system deficiencies. A total of seven primary alternatives were evaluated for initial screening, five of which were evaluated further and finally reduced to the four principal alternatives. For the sake of brevity, only the principal alternatives are described in this section. A present worth analysis for the principal alternatives is presented as well as a final recommended treatment plan. Should the City choose not to implement the recommended plan, the remaining alternatives, or permutations thereof, may be reevaluated and implemented.

7.2. Force Main Capacity Analysis and Identification of Alternatives

The purpose of this subsection is to evaluate the capacity of the existing pump station forcemains under existing and projected conditions. Key design criteria for sewer force mains include maximum velocity, minimum intermittent velocity and detention time. If the velocity in a force main exceeds 5-6 feet per second the friction losses in the pipe can become excessive leading to high pumping costs. High pipe velocities also increase the effects of pressure transients, which increase the wear and tear on the piping and can ultimately lead to premature failures. When pumping raw sewage, a minimum velocity of 2 to 2.5 feet per second must be maintained at least on an intermittent basis to prevent solids accumulation in the piping. In sewage force mains, excessive detention time can lead to corrosion problems caused by the generation of hydrogen sulfide both in the pipeline and at the discharge point. Low pH values can also cause problems with treatment processes. To minimize these problems a maximum detention time of 16 hours is preferred.

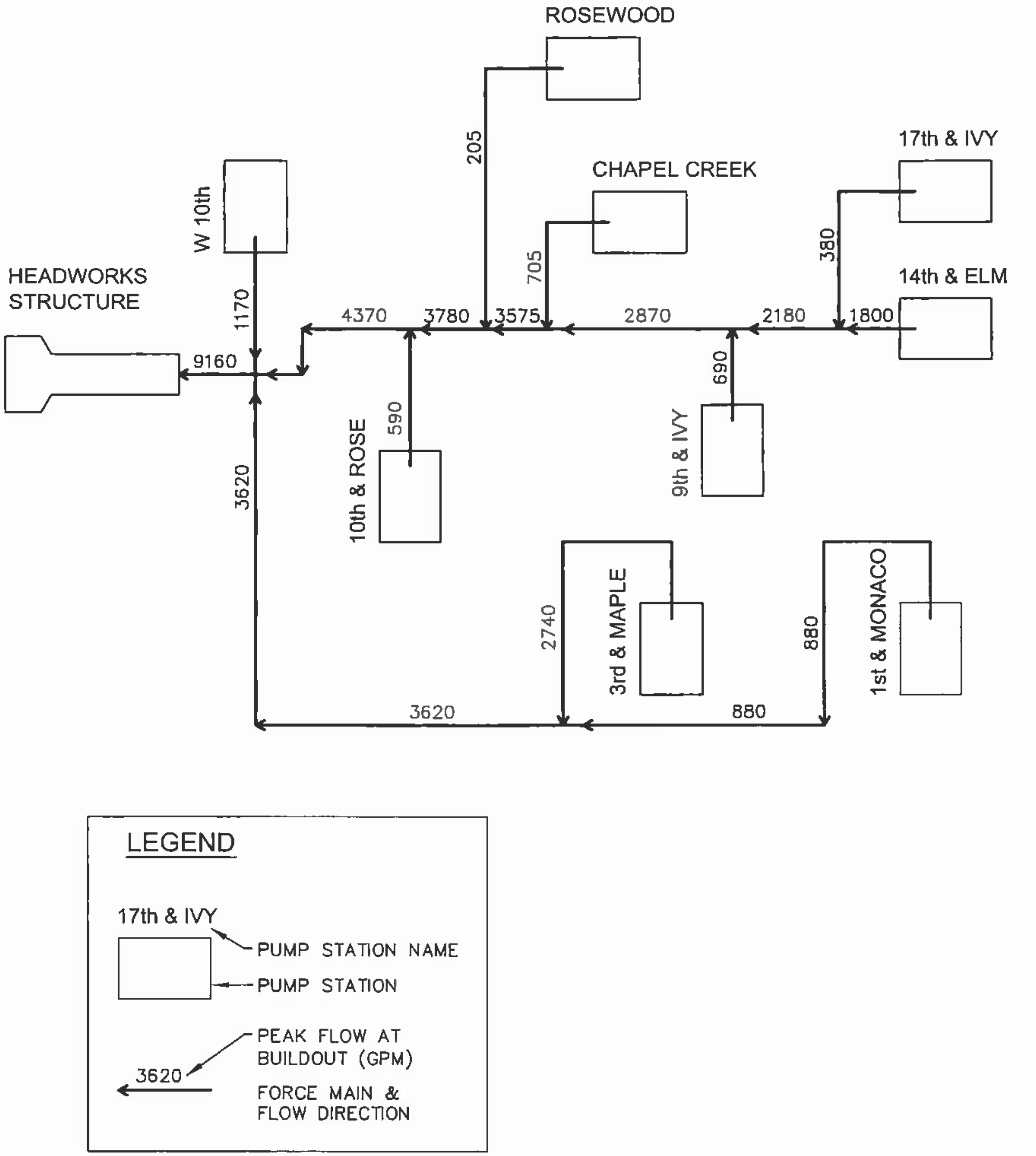
The forcemains were evaluated against these criteria for existing and projected peak hourly flows. The existing capacities of each of the forcemain segments are listed in **Table 7-1**. The reader is encouraged to refer to **Figure 4-1** when reviewing how sewage is currently routed through the system. The primary forcemain lacks the capacity to convey existing peak flows from the Chapel Creek pump station to the WWTP. The 9th & Ivy forcemain also lacks the capacity to convey the existing peak flows. The 3rd & Maple, 1st & Monaco, and Rosewood forcemains will lack the capacity to convey the peak flows associated with growth during the planning period. Two basic alternatives for increasing the capacity of the forcemains were considered. These include replacing the forcemain with a larger size pipe and rerouting forcemain discharge points from their existing location to new locations. As described later in this section, two alternative treatment plant locations were evaluated. These include the existing treatment plant site, and a new site east of the City. The routing of flow through the various forcemains is highly dependent upon the location of the wastewater treatment plant. Since two treatment plant locations were considered, two basic forcemain configurations were also considered. Schematic representations of the two configurations are shown in **Figures 7-1** and **7-2**. The required pipeline capacities for each segment of the proposed forcemains are also listed in **Table 7-1**. The reader is encouraged to refer to **Figures 7-1** and **7-2** when reviewing **Table 7-1**. Evaluations of the forcemain alternatives are included in the evaluations of each of the treatment system alternatives.

TABLE 7-1
Summary of Existing Forcemain Capacity Analysis

Forcemain Segment	Size (Length) [in] [feet]	Existing Capacity [gpm]	Existing Peak Flows [gpm] (1)	Required Buildout Capacity with WWTP in Existing Location [gpm](2)	Required Buildout Capacity with WWTP in New Location [gpm] (3)
Individual Pump Station Forcemains					
• 14th & Elm Forcemain	16 (150)	± 3,500	± 1,970	± 1,800	± 1,800
• 9th & Ivy Forcemain	6 (1230)	± 500	± 690	± 690	± 690
• 3rd & Maple Forcemain	6 (3290)	± 400	± 340	± 2,740	± 2,740
• Chapel Creek Forcemain	6 (950)	± 700	± 440	± 700	± 705
• 10th & Rose Forcemain	6 (310)	± 650	± 350	± 590	± 650
• 17th & Ivy Forcemain	6 (840)	± 500	± 300	± 380	± 380
• 1st & Monaco Forcemain	4 (480)	± 230	± 80	± 880	± 880
• Rosewood Forcemain	3 (340)	± 125	± 125	± 205	± 205
• W 10th Forcemain	-	-	-	±1,170	±1,170

(1) Existing peak flows based on existing flow routing as shown in **Figure 4-1**. Peak flows are assumed to be the greater of either: a) the existing flows from the basins or b) the actual pump station discharge rates.
(2) Projected peak flows based on proposed flow routing as shown in **Figure 7-1**.
(3) Projected peak flows based on proposed flow routing as shown in **Figure 7-2**.

TABLE 7-1 **Summary of Existing Forcemain Capacity Analysis (Continued)**					
Forcemain Segment	Size (Length) [in] [feet]	Existing Capacity [gpm]	Existing Peak Flows [gpm] (1)	Required Buildout Capacity with WWTP in Existing Location [gpm]	Required Buildout Capacity with WWTP in New Location [gpm]
Existing Primary Forcemain					
• From Connection to New Primary Forcemain to 14th & Elm P.S. Discharge	-	-	-	Segment Not Required	±5,540
• 14th & Elm P.S. Discharge to Future 17th & Ivy P.S. Discharge	16 (590)	± 3,500	± 1,800	± 1,800	±3,740
• Future 17th & Ivy P.S. Discharge to 9th & Ivy P.S. Discharge	16 (910)	± 3,500	± 1,890	± 2,180	±3,360
• 9th & Ivy P.S. Discharge to 3rd & Maple P.S. Discharge	16 (665)	± 3,500	± 2,580	± 2,870	±2,770
• Existing 3rd & Maple P.S. Discharge to Chapel Crk. P.S. Discharge	16 (1500)	± 3,500	± 2,920	± 2,870	±2,770
• Chapel Crk. P.S. Discharge to Proposed Rosewood P.S. Discharge	16 (150)	± 3,500	± 3,360	± 3,575	±1,965
• Proposed Rosewood P.S. Discharge to 10th & Rose P.S. Discharge	16 (1850)	± 3,500	± 3,485	± 3,780	±1,760
• 10th & Rose P.S. Discharge to W 10th P.S. and New Primary Forcemain Connection Point	16 (4185)	± 3,500	± 3,835	± 4,370	±1,170
• From Primary Forcemain Connection Point to Existing WWTP	24 (9)	± 12,000	± 3,835	± 9,160	Segment Not Required
Proposed New Primary Forcemain Across South Portion of the City					
• New WWTP Site East of Town to connection with Existing Primary Forcemain	-	-	-	Segment Not Required	±9,160
• From Connection with Existing Primary Forcemain to 1st & Monaco Pump Station Discharge	-	-	-	Segment Not Required	± 3,620
• 1st & Monaco P.S. Discharge to 3rd & Maple P.S. Discharge	-	-	-	± 880	± 2,740
• 3rd & Maple P.S. Discharge to Connection with Existing Primary Forcemain	-	-	-	± 3,620	Segment Not Required
(1) Existing peak flows based on existing flow routing as shown in **Figure 4-1**. Peak flows are assumed to be the greater of either: a) the existing flows from the basins or b) the actual pump station discharge rates. (2) Projected peak flows based on proposed flow routing as shown in **Figure 7-1**. (3) Projected peak flows based on proposed flow routing as shown in **Figure 7-2**.					



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City of Junction City
Sanitary Sewer Facilities Plan

PROPOSED FLOW ROUTING WITH WWTP IN EXISTING LOCATION

FIGURE
7-1

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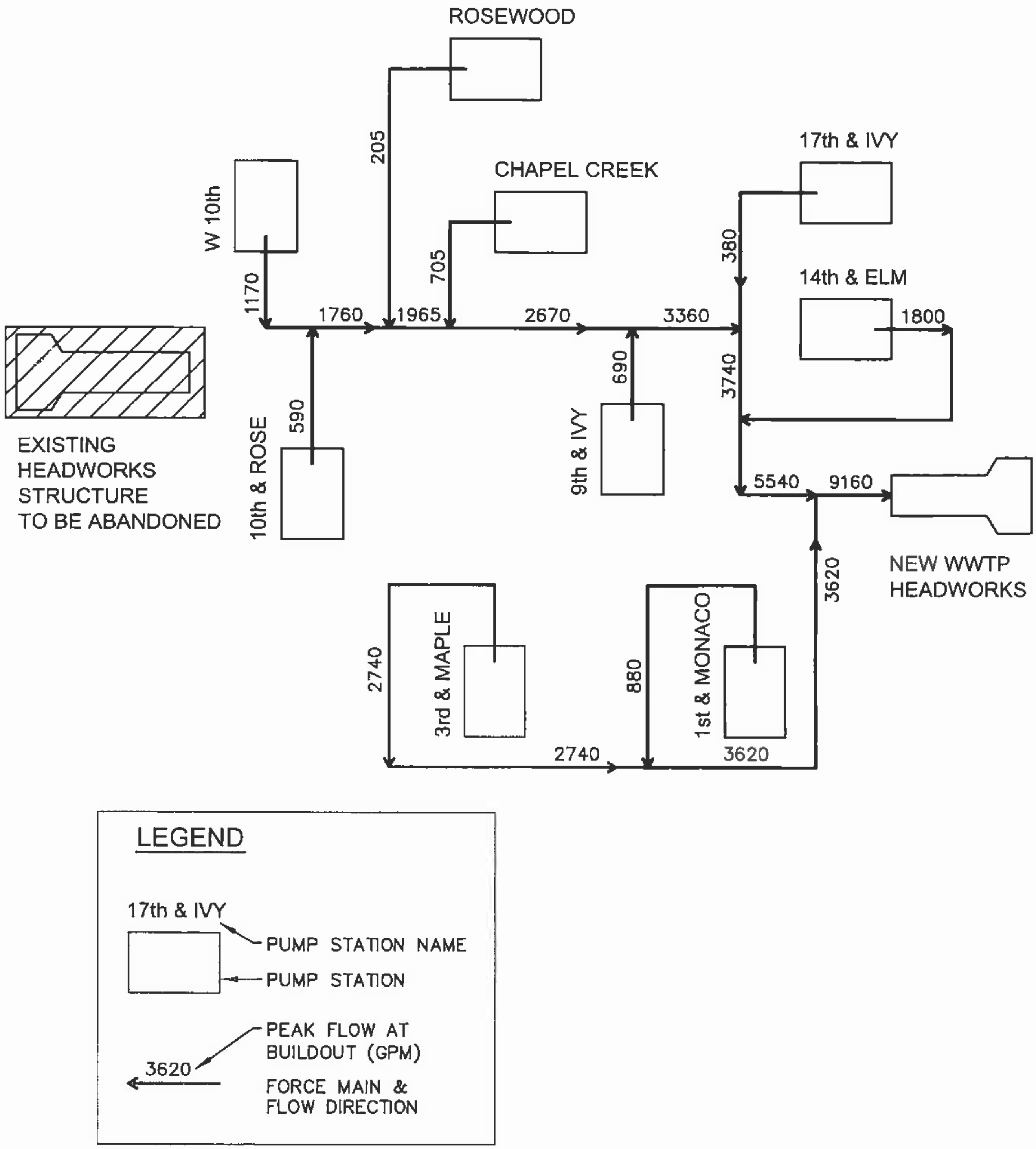
971.3160.0



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City of Junction City

Sanitary Sewer Facilities Plan

PROPOSED FLOW ROUTING WITH WWTP IN NEW LOCATION

FIGURE 7-2

JOB NUMBER 971.3160.0

7.3. Identification of Treatment System Deficiencies

The purpose of this section is to determine the components of the treatment system that are or will become deficient during the planning period. A list of known treatment system deficiencies was developed in **Section 4**. This section is intended to supplement that listing. One of the primary goals of this section is to present an overall list of deficiencies that must be addressed by the City during the planning period. Treatment system deficiencies are typically the result of aging or outdated equipment and systems, or systems that lack the capacity to accommodate increases in wastewater flows and organic loading due to growth in the community.

7.3.1 Hydraulic Loading Capacity

The hydraulic structures, pipelines and unit processes must have the hydraulic capacity to convey anticipated peak flows throughout the design period. DEQ guidelines require that the flow resulting from a 5-year 24-hour storm event must be conveyed to the plant without and overflow. The facilities of concern include:

- Headworks
- Influent Measurement Equipment
- Distribution Piping
- Transfer Pipe
- Outlet Piping
- Disinfection System
- Effluent Measurement Equipment
- Outfall

Wastewater is pumped to the headworks where it is normally distributed to both lagoons. Therefore, the headworks, influent measurement equipment, and distribution piping must be capable of conveying and measuring the peak hourly flows delivered to the plant from the pump stations. The existing peak flows may be estimated by assuming all of the pumps at the eight major pump stations are on. A hydraulic mode of the system was used to estimate the peak flow under these conditions. With all of the existing pumps at the individual pump station on, the model predicts a steady-state flow rate of 4.6 mgd at the plant. The projected peak hourly flow from the pump stations at the end of the planning period is difficult to estimate at this time. As recommended in **Section 6**, several pump stations must be upgraded. Therefore, the peak pumping rates cannot be known until a detailed design for these facilities is performed. Nonetheless, some general conclusions may be drawn from an analysis of the existing facilities.

Headworks and Influent Flow Measurement Equipment. Flow enters the existing headworks and passes through an 18-inch Parshall flume. The flume is approximately three feet tall with one foot of freeboard over the flume. Based on Parshall flume tables, a flow depth of 2.5 feet corresponds to a flow of approximately 16 mgd. The existing peak hourly flow to the WWTP with all existing pumps on is approximately 4.6 mgd. The projected peak hourly flow from the collection system at the end of the planning period is approximately 7.5 mgd. It is important to note that this is the flowrate to the pump stations and not the actual discharge from the stations. Common pump station design practice is to size the pumps such that the peak pump station discharge rate is slightly higher than the peak inflow to the station. As such, the peak flow to the headworks will likely be higher than peak hourly flows from the gravity collection system. One can imagine scenarios where all of the pumps at the pump stations are on at the same time. The likelihood of such a scenario is rare. Nonetheless, it could happen.

In order to estimate the peak flow to the headworks at buildout with all pumps on, a hydraulic model of the proposed improvements was used. Based on some preliminary pump selections, the model showed the estimated peak flow to the headworks with all pumps on is approximately 16 MGD. Though this value is at the upper end of the measurement range for the existing Parshall flume, the flume should still provide reliable flow measurement. As such, no upgrades to the headworks and flow measurement equipment are required during the planning period.

Distribution Piping. Flow is directed to one of the two ponds through 20-inch HDPE discharge piping with four outlets at constant spacing across the north end of each lagoon. The size of the outlets are 10-inch, 12-inch, 12-inch, and 20-inch from the inside edge to the outside edge of each lagoon respectively. The discharge pipe to each lagoon was analyzed using culvert hydraulic analysis principles. The maximum depth of water over the top of the headworks discharge piping was assumed to be 4 feet. This equates to 1 foot of submergence on the discharge side of the parshall flume. The maximum lagoon water surface elevation was assumed to be 322 feet. This provides two feet of freeboard to the top of the lagoon dikes. Based on these conditions, culvert hydraulic analysis techniques were used to analyze the flow in the pipe. Under inlet control the discharge from the four ports was calculated to be 3.4, 3.0, 1.5, and 0.9 MGD respectively for a total flow of 8.7 MGD. Under outlet control conditions, the discharge from the four ports was estimated to be 4.2, 3.9, 2.2, and 1.6 MGD respectively for a total flow of 11.8 MGD. These calculations show that the flow of water through the pipeline is inlet controlled. Therefore the maximum discharge rate through the pipeline is assumed to be approximately 8.7 MGD. Since there are two pipes (i.e., one to each lagoon), the total discharge capacity from the headworks is approximately 17.4 MGD.

Based upon the previous discussion, the peak instantaneous flow rate to the headworks will occur under the buildout condition with all pumps on. As described above, a preliminary estimate of this value is approximately 16 MGD. This is less than the estimated capacity of the distribution piping. As such, no modifications to the piping will be required during the planning period.

Transfer Piping. There is no transfer piping between the two lagoons. The lagoons are run in series at the present time. There is a 12-inch diameter overflow pipe, but this pipe lacks capacity, depth, and the valving required to operate the lagoons in series. In order to isolate the two primary lagoons from one another, the alternatives that include expanding the existing plant include the construction of new transfer piping.

Outlet Piping. The outlet piping is 12-inch diameter piping that has been in operation since the plant was originally constructed in 1968. This piping lacks the capacity required to convey projected plant effluent flows at the end of the planning period. In addition, water is withdrawn from a single point in each lagoon. This contributes to the short circuiting problem in the lagoons. In order to correct the short circuiting problem, and to provide the required outlet capacity to convey projected plant effluent flowrates, new outlet piping is recommended.

Disinfection System. At the present time, chlorine contact time is provided in a 15,000 gallon concrete contact chamber and in the unnamed ditch that runs from the contact chamber to Flat/Crow Creek along the south edge of the WWTP site. It is unlikely that DEQ will permit the use of the drainage ditch to provide chlorine contact time in the future. Given this fact, only the concrete contact chamber may be relied upon in the future to provide the required contact time. This tank lacks the capacity to provide adequate contact time for the projected plant effluent flows. As such additional tankage or piping will be required to provide the necessary contact time. Effluent is currently chlorinated by a chlorine gas feed system. The equipment was installed in 1968 and is becoming obsolete. This equipment should not be relied upon to provide adequate service through the planning period. Therefore, the recommended improvements include a new chlorine contact chamber and new disinfection equipment. The need to dechlorinate effluent prior to surface water discharge will depend on the final outfall configuration and water quality modeling in the mixing zone.

7.3.2 Organic Loading Capacity

The facultative lagoons provide primary and secondary treatment of the waste stream. The organic loading capacity of the lagoons is finite. If this capacity is exceeded compliance problems will result. As discussed in detail in **Sections 4 and 5**, a high-strength industrial user is currently contributing a large organic load to the City's treatment plant. The lagoons currently lack the capacity to treat the domestic loading and the loading from this industrial user. As such, it is strongly recommended that the City continue to work with the industrial user to remove this high-strength waste stream from the City's system. Eliminating this user from the system increases the life of the existing lagoon facilities from an organic loading standpoint. The following analysis demonstrates this point.

In Western Oregon, facultative lagoon design loading rates of up to 35 pounds BOD per acre per day with a maximum of 50 pounds BOD per acre per day to the first cell are typically considered acceptable. For design purposes, a loading rate of 30 pounds per acre per day over the entire plant will be used to develop each of the treatment alternatives considered herein. Each of the existing lagoons are approximately 23.5 acres in size. Therefore, the plant has the capacity to adequately treat approximately 1,410 pounds of BOD per day (i.e., 30 lbs/acre/day * 47 acre = 1,410 lbs./day). Based upon the information presented in **Section 5**, the projected domestic loading rates (not including the high strength industrial user) are listed in **Table 7-2**.

TABLE 7-2
Organic Loading Requirements

Year	BOD Loading (ppd)	Existing Lagoon Area (acres)	Existing Organic Capacity(1) (ppd)	Additional Organic Capacity Required (ppd)	Additional Lagoon Area Required(1) (acres)	Total Lagoon Area Required (acres)
2004	1078	47	1410	0	0	47
2005	1173	47	1410	0	0	47
2010	1327	47	1410	0	0	47
2015	1498	47	1410	88	3	50
2020	1692	47	1410	282	9	56
2025	1912	47	1410	502	17	64
2029	2108	47	1410	698	23	70

(1) Based on an aerial loading rate of 35 pound of BOD per acre.

As demonstrated in **Table 7-2,** if the high-strength industrial user is removed from the system, the existing lagoons have the capacity to provide adequate treatment through the year 2010. Therefore, it is critical that the City continues to work with the industrial user to remove their waste stream from the system. One of the fundamental assumptions used to develop the recommendations in this plan is that untreated, high-strength, industrial waste will not be discharged to the City's system. Therefore, the City must adopt and diligently enforce this policy. Based on the information presented in **Table 7-2,** the selected alternative must provide additional organic treatment capacity.

7.3.3 Crow/Flat Creek Outfall

The existing surface water discharge is a single port outfall to Crow/Flat Creek. For all practical purposes, Crow/Flat Creek is agricultural drainage ditch immediately upstream and downstream of the treatment plant outfall. Flowrates in Crow/Flat Creek are not sufficient to promote the dilution and mixing necessary to comply with current regulatory standards. In short, Crow/Flat Creek is an unacceptable receiving stream for treated effluent from the City of Junction City. As such, future disposal of wastewater to Crow/Flat Creek was not considered. All of the alternatives evaluated herein include disposal to the Willamette River.

7.4. Summary of Treatment System Deficiencies

Based on the discussions in **Section 4** and the information presented above the existing treatment system deficiencies are summarized in **Table 7-3**.

TABLE 7-3
Summary of Treatment System Deficiencies

Location	Description of Deficiency
Treatment Plant	
Lagoons	Lack organic treatment capacity to adequately treat existing organic loads.
Lagoons	Lack organic treatment capacity to adequately treat projected waste loads.
Transfer Piping	Lack of Facility
Outlet Piping	Lack capacity to convey projected plant inflows. Promotes short-circuiting.
Disinfection Equipment	End of useful life. Lacks capacity to adequately dose projected plant effluent flows.
Effluent Flow Measurement Equipment	End of useful life.
Chlorine contact chamber	Lacks capacity to provide adequate contact time for projected plant effluent flows.
Land application facilities	End of useful life.
Outfall/Receiving Stream	Inadequate to comply with existing mixing zone regulations.
Force mains	
16" Primary Forcemain	Lacks capacity to convey existing peak flows from 10th & Rose Pump Station to the WWTP.
9th & Ivy Forcemain	Lacks capacity to convey existing peak flows.
3rd & Maple Forcemain	Lacks capacity to convey peak flows associated with growth.
10th & Rose Forcemain	May reach the end of its useful life during the planning period.
17th & Ivy Forcemain	May reach the end of its useful life during the planning period.
1st & Monaco Forcemain	Lacks capacity to convey peak flows associated with growth.
Rosewood Forcemain	Lacks capacity to convey peak flows associated with growth.

7.5. General Treatment System Alternatives

A broad range of alternatives must be considered as part of the planning for major improvements to wastewater treatment systems. These alternatives generally include no action, expansion of the existing wastewater treatment plant (WWTP), or construction of a new WWTP. The alternative of regional treatment via pumping to Eugene is also considered for discussion purposes. Discussions of each of these general approaches are presented below.

7.5.1 No Action

The No Action alternative must be considered in the facilities planning process to help establish the need for action. Under this alternative, no significant changes would be made to the existing treatment facilities, and the City would continue to operate the existing WWTP as well as possible.

While this is an alternative, it is not considered feasible for the planning period considering the status of the current treatment facility and the projected increases in flows and loadings. If the existing system deficiencies are not addressed, the plant will continue to violate discharge permit requirements. By entering into an MAO with the DEQ, the City has essentially taken the No Action alternative off the table. The No Action alternative is therefore not recommended and will not be considered further.

7.5.2 Regional Treatment

The only city or service district close to Junction City that it would even be conceptually feasible to approach about regional treatment is Eugene. Even though regional treatment typically has the benefits of reducing capital and O&M costs in some cases, this alternative is not economically feasible based on the following. The total cost for the new pump stations and force main required to convey wastewater from Junction City to Eugene will likely far exceed the cost to expand the existing WWTP or construct an entirely new plant.

- The force main length would be a minimum of 12 miles in length from Junction City to the Eugene Regional Wastewater Reclamation Plant (WWRP). All of the pump stations would have to be reconstructed to provide the required discharge rates at the head conditions required to convey wastewater to Eugene.
- Junction City would have to buy capacity in the Eugene Regional WWRP and pay a portion of the operation and maintenance costs. In essence, Junction City would have to pay Eugene the avoided cost for them having to upsize their WWRP to accommodate the flows from Junction City. Although there might be some incremental savings of scale based on the larger size of the Eugene WWRP, the capital and O&M costs required to pump wastewater to the Eugene WWRP would exceed any cost savings.

The regional treatment alternative is therefore not recommended and will not be considered further.

7.5.3 Construct New WWTP

Under this alternative, consideration was given to constructing an entirely new plant. The new plant would utilize none of the existing treatment facilities. This alternative includes abandoning the existing facilities and constructing a new plant east of the City. The primary advantage of this alternative is that wastewater is generally pumped in the direction of disposal. Alternatives that include upgrading the existing plant require pumping all the wastewater collected in the City to the existing treatment plant site west of the City, providing the required treatment, and pumping the treated effluent back across the City to the Willamette River. A new WWTP on the East Side of the City would eliminate large sections of forcemain piping since wastewater would be pumped in a general easterly direction from the City, to the treatment plant, to the point of disposal (i.e., Willamette River). Another advantage of this alternative is that the City could capture a significant amount of value through the sale of the existing WWTP site. The existing WWTP site is inside the UGB and adjacent to a large parcel of land zoned for residential development. If the WWTP site was rezoned for residential development, the land value would increase, and the City would receive greater value from the sale of the land.

Several potential sites for a new WWTP between the City and the Willamette River were screened for further evaluation. Two sites were evaluated further. The first site is located near the northeast corner of the UGB. The site is bounded by the Union Pacific Railroad Tracks to the north and west, a line parallel to 18th Avenue to the south, and Lovelake Road to the East. This site will hence forth be referred to as the North Site. The second site is located further south and is generally bounded by Strome Road to the West and River Road to the East. This site will hence forth be referred to as the South Site. Construction of a WWTP at the North Site is complicated by flooding and wetland issues. The North Site is generally bisected by a broad floodway that conveys water during high flow events. Three large culverts under the Union Pacific Railroad tracks mark the route of the floodway through the site. The site is further encumbered by a number of well established wetlands. Based on a preliminary inspection of the South Site, it appears that it is much more suitable for the construction of a WWTP than the North Site due to the relative absence of flooding and wetland issues. As such, the South Site was selected for further analysis.

Two operational configurations were considered for the new plant. The first includes a summer hold winter discharge operational scheme. The second includes winter discharge with summer land application. These alternatives are discussed in greater detail later in this section.

7.5.4 Upgrade Existing Treatment Plant

Under this alternative, consideration was given to upgrading the existing plant. This alternative includes expanding the hydraulic storage and organic treatment capacity of the existing plant, constructing new lagoon effluent piping, new disinfection equipment, and a new effluent pump station and pipeline, and a new outfall in the Willamette River. For the most part, the existing headworks and lagoons would remain in service and the new facilities would provide the required additional hydraulic storage capacity, organic treatment capacity, and effluent disposal requirements. The existing lagoons are in relatively good condition and are well suited to incremental expansion. Therefore, this alternative has the benefit of retaining the residual value remaining in the existing facilities. Two permutations of this alternative are considered. The first includes converting to a summer hold winter discharge operational scheme. The second includes retaining the existing winter discharge summer land application operational scheme. These alternatives are discussed in greater detail later in this section.

7.6. Primary and Secondary Treatment Alternatives

As described above, one of the primary deficiencies in the existing treatment facilities is the inability to provide the required organic treatment capacity throughout the planning period. This section, therefore, presents the alternatives that were evaluated to increase the organic treatment capacity of the plant.

7.6.1 Mechanical Treatment

As used herein, mechanical treatment refers to primary treatment followed by some type of suspended or attached growth biological treatment process such as activated sludge, sequencing batch reactors, oxidation ditches, trickling filters, rotating biological contactors, etc. As discussed below, a year-round discharge to the Willamette River is not considered feasible for Junction City. As such, any mechanical treatment alternative must include a storage lagoon to store treated or untreated wastewater during non-discharging periods. In Western Oregon wastewater collection systems that accumulate large amounts of I/I, hydraulic storage rather than organic treatment requirements typically controls the size of lagoon facilities. In other words, a wastewater lagoon sized to provide hydraulic storage will generally be large enough to provide sufficient organic treatment. Therefore, if a storage lagoon is required to store wastewater during non-discharging periods, mechanical treatment is not necessary. All of the required biological treatment will occur in the storage lagoon. In essence, the need to provide summer storage eliminates the need to consider a purely mechanical treatment facility.

7.6.2 Facultative Lagoons

This alternative includes utilizing facultative lagoons to provide the required treatment. For the alternatives that include expanding the existing plant, a new facultative lagoon would be constructed to provide the added capacity. Alternatives that include the construction of a new plant would include new facultative lagoons. As the City's experience with the existing facultative lagoon system demonstrates, this treatment technology is relatively simple and inexpensive to maintain and operate. The power requirements are minimal, and essentially no rotating machinery is required. Therefore, power and maintenance costs are very low. The drawback of this alternative is that it tends to require the greatest amount of land area.

One of the primary decisions to be made when designing wastewater lagoons is whether to use a natural clay liner, a bentonite-enhanced liner, or a synthetic liner in the construction of the lagoon bottom. In order to meet current seepage requirements, synthetic liners are usually required. The existing lagoons are unlined. As described in **Section 4**, the seepage rate from the lagoons is below the current DEQ maximum. Without a detailed geotechnical report, it is difficult to determine if unlined lagoons are feasible at this time. The conservative approach with respect to budgeting includes assuming that synthetic liners are necessary. Eliminating the synthetic liner would present significant cost savings. As such, a detailed analysis of the soil conditions should be performed at the predesign stage.

Based on a design aerial loading rate of 30 pounds per acre per day, the information in **Table 7-2** suggests that the minimum facultative lagoon area required to treat 2029 loading rates is approximately 70 acres. It is important to note that this does not include storage requirements. The amount of hydraulic storage required is influenced by a number of factors including inflow rates, discharge rates, lagoon size, precipitation, seepage rates and evaporation rates. In Western Oregon, hydraulic storage requirements rather than organic treatment typically control the size of lagoons for communities with standard domestic waste strength. Therefore, lagoons that are properly sized from a hydraulic storage point of view should provide the required organic treatment capacity. The hydraulic storage requirements for each alternative are determined in **Section 7.8**.

7.6.3 Partially-Mix Lagoons

Partially mixed lagoons are typically deeper and more heavily loaded organically than facultative lagoons. Oxygen is supplied directly by floating mechanical aerators, submerged diffused aerators, or by floating mechanical mixers that enhance surface reaeration and algae growth. Key design parameters include the amount of aeration and mixing, total horsepower requirements, and aerator or mixer spacing. The aeration is designed to meet the oxygen requirements for BOD removal and in some cases, nitrification. Only a moderate degree of mixing is provided so that solids are not maintained in suspension as in the activated sludge process.

Several permutations of the partially mixed lagoon alternative were considered. These included converting one or both of the existing facultative lagoons to partially mixed lagoons, constructing a new partially mixed lagoon at the upstream end of the treatment plant, or constructing all new partially mixed lagoons. Two alternatives for supplying oxygen were also considered. These included floating mechanical aerators and floating solar powered mixers. In order to ensure mixing during periods of heavy cloud cover and at night, the solar powered mixers must be provided with backup power to be considered a viable alternative. The power costs for floating mechanical aerators is substantially higher than for solar powered mixers. Therefore, aerators were dropped from further consideration.

A preliminary screening of the partially mixed lagoon alternatives showed that the organic treatment capacity of the existing lagoons could be sufficiently increased by adding mechanical mixers. Therefore, the primary advantage of this alternative is that no additional lagoons would be required for treatment purposes. However, the existing lagoons, do not have the required hydraulic storage capacity required through the planning period even if summer land application is continued. As such, an additional lagoon is required to provide the required hydraulic storage. This demonstrates a fundamental point worth noting. Hydraulic storage rather than organic treatment determines lagoon size in small waste water systems in Western Oregon that are heavily influenced by infiltration and inflow and that have typical domestic waste strengths. As such, there is no benefit to increasing the organic treatment capacity of the lagoons and any alternative that includes mechanical enhancements aimed at increasing the organic treatment capacity of the lagoons will inevitably be more expensive than a facultative lagoon. For these reasons, the partially mixed alternative was removed from further consideration. This logic applies to partially aerated lagoons and completely mixed lagoons as well. As such, these alternatives were also not considered.

7.6.4 Constructed Wetlands

Constructed wetlands are generally defined as systems designed for wastewater treatment in an area where natural wetlands do not exist. There are two different types of constructed wetlands. Free water surface wetlands consist of a relatively shallow channel along which the wastewater flows. Subsurface flow wetlands consisting of a layer of permeable media through which the wastewater flows. Both systems utilize emergent aquatic vegetation that promote microbial growth. Both systems also include some type of barrier beneath the wetland bed to prevent groundwater contamination. Primary treatment is required for constructed wetland systems.

Similar to the lagoon alternatives, constructed wetlands require a large area, especially for communities that experience high levels of precipitation. Recent case studies have shown that constructed wetlands can produce high quality effluent. Constructed wetlands can also be used for polishing in conjunction with facultative lagoons or other secondary treatment processes, further reducing BOD and TSS concentrations.

Constructed wetlands are designed as flow through systems that do not provide hydraulic storage. Therefore, they are only feasible in Junction City in conjunction with an additional storage lagoon. Since flow through constructed wetlands must be maintained to promote the health of the aquatic vegetation, effluent from the lagoon must be recycled during periods when the plant is not discharging to either the river or a land application facility.

A number of constructed wetland configurations were considered. However, due to the complications discussed above and the large land area requirements, none of the configurations were cost effective. Therefore, constructed wetlands were removed from further consideration.

7.7. Advanced Treatment Alternatives

Advanced treatment is not expected to be required to meet discharge requirements to the Willamette River during the permitted winter discharge period and will not be considered further.

7.8. Effluent Disinfection Alternatives

Several effluent disinfection alternatives were considered including liquid hypochlorite, chlorine tablets, gas chlorine, onsite chlorine generation, Ozone, and ultraviolet light. Due to the shielding effects that result from high solids content of lagoon effluent, ultraviolet radiation will not provide reliable disinfection. Ozone disinfection of wastewater is rarely used in the United States, and is cost prohibitive for smaller systems such as Junction City. Due to the size of the projected chlorine demands, the chemical costs associated with sodium hypochlorite liquid and solid chlorine are prohibitive when compared to gas chlorine or onsite generation. Therefore, gas chlorine and onsite generation are considered to be the two feasible alternatives.

Onsite chlorine generation systems are generally much more expensive and more complex than gas chlorine systems. The added costs and complexity associated with onsite generation are generally justified by the added safety. Onsite generation systems utilize electricity to create a weak chlorine solution from a Sodium Chlorite salt brine mixture. All of the chemicals used or generated as part of this process are relatively safe when compared to gas chlorine. Gas chlorine systems are much less expensive and are simpler to operate, but are more hazardous. The City currently operates a gas chlorine feed system and has been doing so since 1968. The existing public works staff is familiar the operation and associated hazards of gas chlorine systems. Therefore, the added safety associated with an onsite generation system does not justify the added costs. As such, onsite generation was eliminated from further consideration and gas chlorine is the recommended disinfection alternative.

7.9. Hydraulic Storage/Effluent Disposal Alternatives

As described above, all of the alternatives include a new discharge to the Willamette River. Flat/Crow creek is an unacceptable receiving stream in today's regulatory climate. In addition to the Willamette, the Long Tom River was also considered. The Long Tom River is currently listed on the 303d list. This means that it does not meet water quality standards. The Willamette River is also listed on the 303d list. In general, the DEQ discourages new discharges to water quality limited streams. However, under certain circumstances the DEQ can permit new discharges to water quality limited streams. If studies show that the proposed discharge does not significantly degrade the receiving stream, new discharges may be permitted. The Long Tom River is designated as "cool water" habitat whereas the Willamette is designated for "Salmonid Use." As such, the ambient criteria for the Long Tom River are less stringent than for the Willamette River. Nonetheless, the Willamette River is still considered to be the most feasible receiving stream due to the fact that it is much larger than the Long Tom. Due to its larger size and more turbulent nature, the Willamette River is a better receiving stream with respect to mixing, dilution, and oxygen sag. Therefore, the Long Tom River was eliminated from further consideration as a receiving stream. All of the alternatives analyzed herein include a winter discharge to the Willamette River.

A significant amount of work must be done to gain regulatory approval for a new Willamette River outfall. The City will have to obtain a new NPDES permit from the DEQ. In addition, the City will have to obtain permits from the U.S. Army Corps of Engineers and the Oregon Division of State Lands in order to place the new outfall piping in the stream. The permitting process can be time consuming. Therefore, project schedules should be developed to provide sufficient time for obtaining all necessary permits.

The need for hydraulic storage is driven by effluent disposal practices. For example, effluent from the existing plant is only discharged during the irrigation season and during the winter months. Consequently, all flow to the plant that occurs between the end of the discharge season and the beginning of the irrigation season must be stored. The required storage volume is provided in the lagoons. The volume of storage depends on the method of disposal. For year around discharge no storage is required, for winter discharge summer land application some storage is required, and summer hold winter discharging systems require the most storage capacity. Therefore, hydraulic storage and effluent disposal are not mutually exclusive issues. As such, they are considered together in the development of treatment alternatives.

7.9.1 Year-round Discharge to Willamette River

Under this alternative, consideration was given to reducing the need for summertime hydraulic storage by discharging to the Willamette River during the summer months. In order to discharge during the summer, current regulations require an effluent with BOD and TSS concentrations below 10 mg/L. This would require an effluent polishing step at the downstream end of the existing lagoons. In addition, the City would have to demonstrate to the DEQ that the summertime discharge met all other effluent quality standards (i.e., temperature, DO, toxic substances, etc.). The primary drawback of this alternative is that it would add significant operational complexity and operational cost to the WWTP. The polishing step would require significantly more operator attention and expertise than the existing facility. Therefore, operating costs would increase. The Willamette River is water quality limited for temperature during the summer months. Therefore, the process of gaining regulatory approval for this alternative is likely to be long and expensive and have only a marginal chance of success. Discussions to date with DEQ personnel suggest this alternative is not desired or considered feasible. Therefore, this alternative was eliminated from further consideration.

7.9.2 Winter Discharge Summer Effluent Reuse

Under this alternative consideration was given to eliminating or reducing the need for hydraulic storage by discharging to a land application system during the summer holding period. A large number of reuse alternatives were evaluated. Key factors that were considered in developing each of the alternatives are listed as follows.

7.9.2.1 Overall Project Approach

One of the fundamental decisions that must be made when evaluating an effluent reclamation project is whether the project will be approached from a "wastewater disposal" philosophy or a "resource optimization" philosophy. With the "wastewater disposal" approach, the dominating priority is to dispose of excess wastewater. Using this approach, a municipality will generally desire to discharge the maximum amount of water that can be agronomically utilized on the lowest cost reuse site without much consideration to profitability or crop health. Crop selection criteria are developed to optimize water consumptive rates with little value placed on the agricultural crop. With the "resource optimization" approach, the dominating priority is to maximize the profitability of the agricultural crop by optimizing the use of the wastewater resource. In other words, wastewater is applied at rates that will produce the greatest profitability. The recommended approach is a resource optimization approach. The primary advantage of this approach is the greater potential for profitability. This approach also tends to be more environmentally friendly and is more compatible with the exclusive farm use zoning.

7.9.2.2 Ownership Alternatives

A number of ownership alternatives were considered. These include public versus private land ownership, public versus private operation, and land lease. The ownership alternatives are critical when evaluating the capital costs, operation and maintenance costs, and when evaluating revenue from potential crop sales. For example, capital costs for a publicly owned reuse facility are much higher than for land that is simply leased to an agricultural user. Likewise, O&M costs are higher for a publicly maintained facility than for a privately maintained facility. Finally, the division of crop revenue between the private owner/operator and the municipality must also be considered. Based on circumstances in Junction City, two ownership alternatives were evaluated for further consideration. The first alternative includes public ownership of the land, crops, and all reuse facilities with operation contracted to a private farm operator. This alternative involves significant capital resource and risk but has the greatest potential for profitability. The second alternative includes a typical lease arrangement of irrigated land to a farm operator. The City would provide water at pressure to the lessee. The lessee would be responsible for all crop and irrigation costs, all operation costs, and would have full ownership of all proceeds from the sale of the harvest crops.

7.9.2.3 Wastewater Quantity and Quality Requirements

Each of the reuse alternatives were evaluated based on the volume of treated effluent required as well as the effluent quality requirements. Based on the preliminary evaluation, the effluent from the plant was determined to be suitable for effluent reuse. Depending on the type of irrigation system, the effluent from a lagoon system may require filtering to prevent clogging the distribution system.

7.9.2.4 Crop Selection Alternatives

Key crop selection considerations include, agronomic application rates, harvesting schedule, history of use for land application facilities, replanting requirements, and market conditions. Several different crops were considered including grass pasture, alfalfa, corn silage, grass silage, hybrid poplar, Christmas trees, peppermint, and ornamental trees.

7.9.2.5 Irrigation System Alternatives

A number of irrigation alternatives were evaluated. These included center pivot, travelling gun, wheel line, hand set lines, aluminum solid set lines, polyethylene solid set lines with micro-spray heads, and polyethylene solid set lines with drip emitters. Based on the two crops identified as having potential for profitability, the appropriate types of irrigation systems are solid set type systems using either low cost polyethylene laterals with micro-spray heads, or aluminum laterals with bronze sprinkler heads.

7.9.2.6 Recommended Reuse Alternative

The reuse alternatives were screened separately from the overall wastewater treatment facility. A total of three reuse alternatives were selected for further evaluation. These are listed as follows.

1) Land lease of City-owned land with a summer irrigation source to a private agricultural operator.

2) City-owned hybrid poplar agricultural facility with a contract operator.

3) City-owned grass seed agricultural facility with a contract operator.

Based on discussions with City personnel, the preference is to lease the land to a private agricultural user that will have full control of crop selection and may keep all proceeds from crop sales (reuse alternative 1). This option requires less initial capital investment, and poses the least amount of risk to the City. The drawback is that the City surrenders all potential profits from the sale of the crop. City personnel based this decision on the belief that the City should remain in the business of supplying a wastewater utility to its customers only. The capital investment required to establish an agricultural enterprise would likely require higher user fees. City personnel do not believe the modest potential profitability justifies the increased cost and risk to the users.

7.9.2.7 Land Application Land Area Requirements

As discussed above, the recommended reuse alternative includes the City purchasing the required land and constructing an irrigation pump station and distribution piping. The land would then be leased to an agricultural operator who would have control over crop selection, would provide all irrigation equipment, and would keep all proceeds from the sale of the harvested crops. Since the City would not have any control over crop selection, it is difficult to determine the exact land requirements at this time.

As described above, several crops were evaluated and determined to be suitable for effluent reuse. Typical irrigation and nutrient requirements for these crops are listed in **Table 7-4**.

TABLE 7-4
Suitable Crops For Effluent Reuse

Crop	Annual Water/Nutrient Requirements			
	Water (in/ac)	N (lbs/ac)	P (lbs/ac)	K (lbs/ac)
Grass Pasture	18-27	75-150	30-70	200-300
Grass Silage	18-27	130-300	20-80	200-300
Alfalfa	22-33	10-40	80-100	80-100
Hybrid Poplar	24-36	80-100	60-150	150-500
Christmas Trees	12-20	60-120	90-180	100-200
Peppermint	18-24	200-250	100-150	120-200

As shown in **Table 7-4**, water requirements vary widely from a minimum of 12 inches per year to a maximum of 36 inches per year depending on the crop. Since the recommended reuse alternative includes leasing the land to an agricultural producer, the City will not have control over crop selection. Therefore, the City must take care to ensure that enough land is purchased to facilitate the required wastewater disposal and yet allow the greatest flexibility from an agricultural producers point of view. If the City does not purchase enough land, the agricultural producer and the City may be at odds. The goal of the agricultural producer is to maximize profits while the goal of the City is to dispose of wastewater. If market forces dictate that the agricultural user harvest a crop with low water requirements and the land application area is relatively small, the agricultural user may not be able to profitably dispose of the total quantity of wastewater required to meet the City's needs. In order to guard against this scenario, it is recommended that the City take a conservative approach with respect to effluent disposal and error on the side of too much land at this time. It is also recommended that the City plan the land application facilities for incremental expansion. As part of the initial project, it is recommended that the City purchase enough land to meet the land application needs of the community in the year 2020 assuming that a crop requiring low water consumption is planted. In this way, should agricultural markets lean toward crops that utilize large amounts of water, the facilities that were sized to meet the 2020 land application requirements will end up meeting the needs further into the future. The City may then delay expansion of the land application facilities as appropriate.

The amount of land required may be estimated by performing a dry-season water balance. The volume of water that must be land applied is the total dry season inflow less evaporation. It is assumed that the dry season corresponds to May 1 through October 31. Based on preliminary discussions with DEQ, the City may be permitted to discharge to the river during the month of May during wet years when flows are high. For planning purposes, it was assumed that no discharge would be allowed during the month of May. This approach is appropriate since high river flows in May are not reliable. If the facilities were sized to rely on discharge during the month of May, hydraulic storage shortfalls would be more likely to occur during years when May river flows are not sufficient to accommodate effluent discharge

The volume of wastewater lost through evaporation is highly dependent upon lagoon size. To aid in the evaluation of the treatment plant alternatives, a dry-season mass balance was performed for a range of lagoon sizes. The land application area required for crops with low (i.e., 20 in/year) and high (i.e., 36 inches per year) water demand is listed as a function of lagoon size in **Table 7-5.** The information in **Table 7-5** will be used in the following sections to determine the land application requirements for each of the alternatives. The calculations are based on the following assumptions.

- Dry Season Duration 184 days
- Plant inflow equal to Average Dry Weather Flow
- Average Net Summer Evaporation 15 inches
- No seepage

TABLE 7-5 Land Application Area Requirements							
	Year	2010		2020		2029	
	ADWF (MGD)	0.653		0.833		1.039	
	Agronomic Irrigation Rate (in/year)	20	36	20	36	20	36
Total Lagoon Area (acre)	45	188	104	249	138	318	177
	50	184	102	245	136	315	175
	55	180	100	241	134	311	173
	60	176	98	237	132	307	171
	65	173	96	234	130	303	169
	70	169	94	230	128	300	166
	75	165	92	226	126	296	164
	80	161	90	222	124	292	162
	85	158	88	219	121	288	160
	90	154	85	215	119	285	158
	95	150	83	211	117	281	156
	100	146	81	207	115	277	154

The recommended land application implementation plan consists of two stages. For the first stage, the amount land required is based on 2020 flow projections and annual crop water usage of approximately 20 inches. For the second stage, the amount of land required is based on 2029 flow projections and annual crop water usage of 20 inches. The first stage will also include the construction of the irrigation pump station and primary distribution pipelines. Since these facilities are not suitable for incremental expansion, it is recommended that they be sized for the effluent flows at buildout rather than at the end of the planning period.

7.9.2.8 Land Application Hydraulic Storage Requirements

Treated effluent may only be applied at agronomic rates. As such, effluent cannot be land applied during wet periods. Under a summer land application winter discharge operational scheme, surface water discharge is not permitted during the months May and June. These months are often too wet to create conditions suitable for irrigation at agronomic rates. As such, wastewater must be stored to allow the ground to dry to the point that it can accept irrigation. Lagoons are typically used to store the wastewater. At the end of the winter discharge season, operators draw the water level down to minimum depths. After the cut-off for the winter discharge season, all discharge from the plant ceases and the Lagoons are allowed to fill until land application can begin. Hydraulic storage rather than the organic treatment, usually controls the lagoon sizing in Western Oregon. A water balance was performed to evaluate the hydraulic storage requirements, and consequently the size of lagoons required. The results the water balance are presented in **Table 7-6**. Lagoon area requirements are listed for expansion of the existing plant as well as the construction of a new plant. The existing lagoons were constructed to provide a maximum water depth of 6 feet. As such only 4 feet is available for storage. It is assumed that all new lagoons will be constructed with a maximum water depth of 8 feet and provide 6 feet of usable storage. Due to the relatively shallow depth of the existing lagoons, alternatives that include expanding the existing treatment plant require more total lagoon area than alternatives that include constructing a new plant with new 8-foot deep lagoons. The information in **Table 7-6** is used later in this section for the development of complete treatment system alternatives. The water balance is based on the following assumptions.

- Storage duration 60 days
- Plant inflow equal to Average Annual Flow
- Existing lagoon area 47 acres
- Existing lagoon storage depth 4 feet
- New lagoon storage depth 6 feet
- Average May and June rainfall 3.75 inches
- Average May and June evaporation 7 inches
- No seepage
- No discharge

TABLE 7-6
Lagoon Area Required to Provide Hydraulic Storage For Summer Land Application

Year	AAF (MGD)	For Expansion of Existing WWTP		For Construction of New WWTP
		Additional Lagoon Area Required (acres)	Total Lagoon Area Required (acres)	Total Lagoon Area Required (acres)
2004	1.090	4	51	35
2005	1.155	6	53	37
2010	1.262	9	56	41
2015	1.380	13	60	45
2020	1.514	17	64	49
2025	1.666	21	68	54
2029	1.801	25	72	58

7.9.3 Winter Discharge Summer Storage

This disposal alternative includes storing the flows during the dry season in the lagoons with winter discharge to the Willamette River. From an operational point of view this alternative is simple and reliable. Rather than discharging to a land application facility, this alternative includes storing summer flows in the lagoons. It is assumed that the winter discharge period will begin November 1 and end April 30. Therefore, all plant inflows that occur between May 1 and October 31 must be stored in the lagoons. Based on preliminary discussions with DEQ, the City may be permitted to discharge to the river during the month of May during wet years when flows are high. For planning purposes, it was assumed that no discharge would be allowed during the month of May. This approach is appropriate since high river flows in May are not reliable. If the facilities were sized to rely on discharge during the month of May, hydraulic storage shortfalls would be more likely to occur during years when May River flows are not sufficient to accommodate effluent discharge.

A water balance was performed to evaluate the hydraulic storage requirements, and consequently the size of lagoons required. The results the water balance are presented in **Table 7-7**. Lagoon area requirements are listed for expansion of the existing plant as well as the construction of a new plant. The existing lagoons were constructed to provide a maximum water depth of 6 feet. As such only 4 feet is available for storage. It is assumed that all new lagoons will be constructed with a maximum water depth of 8 feet and provide 6 feet of usable storage. Due to the relatively shallow depth of the existing lagoons, alternatives that include expanding the existing treatment plant require more total lagoon area than alternatives that include constructing a new plant with new 8-foot deep lagoons. The information in **Table 7-7** is used later in this section for the development of complete treatment system alternatives. The water balance is based on the following assumptions.

- Duration of summer storage period (May-Oct.) — 184 days
- Plant inflow equal to Average Dry Weather Flow
- Existing lagoon area — 47 acres
- Existing lagoon storage depth — 4 feet
- New lagoon storage depth — 6 feet
- Average net summer evaporation — 15 inches
- No seepage
- No Discharge

TABLE 7-7
Lagoon Area Required to Provide Hydraulic Storage for Summer Storage

Year	ADWF (MGD)	For Expansion of Existing WWTP		For Construction of New WWTP
		Additional Lagoon Area Required (acres)	Total Lagoon Area Required (acres)	Total Lagoon Area Required (acres)
2004	0.530	7	54	41
2005	0.577	11	58	45
2010	0.653	17	64	51
2015	0.738	23	70	57
2020	0.833	31	78	65
2025	0.942	39	86	73
2029	1.039	47	94	81

7.9.4 Maximum Winter Discharge Rate

The maximum projected plant discharge rate must be estimated to determine the size of all facilities downstream of the lagoons. Lagoons act as flow equalization basins. Therefore, all facilities downstream of the lagoons must be sized to accommodate the maximum discharge rate from the plant rather than the peak inflow to the plant. The maximum discharge rate occurs in the winter when wastewater is discharged to the receiving stream. Performing a wet-season mass balance shows that the total volume of wastewater that must be discharged is the sum of the stored water in the lagoon, the wet season plant inflows, and the precipitation that falls on the lagoon. The size of the lagoons has a strong influence on the required plant discharge rate since precipitation that falls on the lagoons must be discharged. To aid in the evaluation of the treatment plant alternatives, a wet season mass balance was performed for a range of lagoon sizes. These calculations were performed for both the expansion of the existing plant and the construction of a new plant. The results of this analysis are presented in **Table 7-8**. The information in **Table 7-8** is used later in this section for the development of complete treatment system alternatives. The calculations are based on the following assumptions.

- Duration of wet season discharge period (Nov.-April) 181 days
- Actual days wastewater is discharged 150 days
- Plant inflow equal to Average Wet Weather Flow
- Existing lagoon area 47 acres
- Existing lagoon storage depth 4 feet
- New lagoon storage depth 6 feet
- November-April 20% exceedence Rainfall 53.5 in.
- No seepage
- No Evaporation

TABLE 7-8 Maximum Plant Discharge Rate							
Year AWWF (MGD)		2010 1.811		2020 2.207		2029 2.577	
		Peak Discharge Expansion of Existing WWTP (MGD)	Peak Discharge New WWTP (MGD)	Peak Discharge Expansion of Existing WWTP (MGD)	Peak Discharge New WWTP (MGD)	Peak Discharge Expansion of Existing WWTP (MGD)	Peak Discharge New WWTP (MGD)
Total Lagoon Area (acre)	50	3.201	3.405	3.595	3.799	4.041	4.245
	55	3.315	3.519	3.708	3.912	4.155	4.359
	60	3.428	3.632	3.822	4.026	4.268	4.472
	65	3.542	3.746	3.935	4.139	4.382	4.586
	70	3.655	3.859	4.049	4.253	4.495	4.699
	75	3.769	3.973	4.162	4.366	4.609	4.813
	80	3.882	4.087	4.276	4.480	4.722	4.926
	85	3.996	4.200	4.389	4.594	4.836	5.040
	90	4.110	4.314	4.503	4.707	4.949	5.154
	95	4.223	4.427	4.617	4.821	5.063	5.267
	100	4.337	4.541	4.730	4.934	5.177	5.381

7.9.5 Mass Load Limits

The mass load limits for the various disposal alternatives may be estimated by performing a wet season water balance for the proposed treatment plant. This analysis shows that the total volume of wastewater that must be discharged is the sum of the stored water in the lagoon, the wet season plant inflows, and the precipitation that falls on the lagoon. The size of the lagoons has a strong influence on the required mass loads since precipitation that falls on the lagoons must be discharged. To aid in the evaluation of the treatment plant alternatives, a wet season mass balance was performed for a range of lagoon sizes. These calculations were performed for both the expansion of the existing plant and the construction of a new plant. The results of this analysis are presented in **Table 7-9 and 7-10**. The information in these tables is used later in this section for the development of complete treatment system alternatives. The calculations are based on the following assumptions.

- Duration of wet season discharge period (Nov.-April) — 181 days
- Plant inflow equal to Average Wet Weather Flow
- Existing lagoon area — 47 acres
- Existing lagoon storage depth — 4 feet
- New lagoon storage depth — 6 feet
- November-April Average Rainfall Depth — 36.6 in.
- Effluent BOD concentration — 30 mg/L
- Effluent TSS concentration — 50 mg/L
- No seepage
- No Evaporation

TABLE 7-9 Effluent Organic Mass Load							
Year AWWF (MGD)		2010 1.811		2020 2.207		2029 2.577	
		BOD Mass Load Expansion of Existing WWTP (ppd)	BOD Mass Load New WWTP (ppd)	BOD Mass Load Expansion of Existing WWTP (ppd)	BOD Mass Load New WWTP (ppd)	BOD Mass Load Expansion of Existing WWTP (ppd)	BOD Mass Load New WWTP (ppd)
Total Lagoon Area (acre)	50	632	674	714	756	806	849
	55	652	695	734	776	827	869
	60	673	715	754	797	847	889
	65	693	735	775	817	867	910
	70	714	756	795	837	888	930
	75	734	776	815	858	908	950
	80	754	797	836	878	928	971
	85	775	817	856	899	949	991
	90	795	837	877	919	969	1012
	95	815	858	897	939	990	1032
	100	836	878	917	960	1010	1052

TABLE 7-10 Effluent Solids Mass Load							
Year AWWF (MGD)		2010 1.811		2020 2.207		2029 2.577	
		TSS Mass Load Expansion of Existing WWTP (ppd)	TSS Mass Load New WWTP (ppd)	TSS Mass Load Expansion of Existing WWTP (ppd)	TSS Mass Load New WWTP (ppd)	TSS Mass Load Expansion of Existing WWTP (ppd)	TSS Mass Load New WWTP (ppd)
Total Lagoon Area (acre)	50	1053	1124	1189	1260	1344	1414
	55	1087	1158	1223	1294	1378	1448
	60	1121	1192	1257	1328	1412	1482
	65	1155	1226	1291	1362	1446	1516
	70	1189	1260	1325	1396	1479	1550
	75	1223	1294	1359	1430	1513	1584
	80	1257	1328	1393	1464	1547	1618
	85	1291	1362	1427	1498	1581	1652
	90	1325	1396	1461	1532	1615	1686
	95	1359	1430	1495	1566	1649	1720
	100	1393	1464	1529	1599	1683	1754

7.10. Biosolids Treatment and Disposal Alternatives

Biosolids are collected in the lagoons. As described in **Section 4**, the biosolids were removed from the existing lagoons in 2001. Therefore, it is not anticipated that the biosolids will need to be removed during the planning period. Furthermore, all of the alternatives evaluated are lagoon-based systems with no biosolids treatment facilities. As such, the biosolids treatment and disposal alternative which is most feasible is the removal of biosolids and beneficial land application. However, as long as the City prevents high-strength industrial discharge into the City's system, biosolids removal will not be required during the next planning period.

7.11. Routine Maintenance for WWTPs

A routine maintenance program for the WWTP is just as important to the operation of the sewerage system as the systematic cleaning, inspection and rehabilitation program for the gravity collection system. In many respects, this program is even more critical, since major mechanical or control failures at the WWTP results in bypasses of untreated or partially treated wastewater.

The City currently has a file system with the status and maintenance history of all of the major components in the WWTP and influent pump stations. The City should continue their policy of preventative maintenance on system components. Following the construction of the new WWTP improvements, the City should implement a policy to update and revise the O&M manuals as system components are replaced or upgraded.

7.12. Development of Principal Treatment System Alternatives

The existing treatment system deficiencies are listed in **Table 7-3**. The purpose of this subsection is to develop complete alternatives that address these deficiencies and that will provide reliable service through the planning period. The alternatives described above were compared against the deficiencies to develop the complete treatment system alternatives listed in **Table 7-11**. A brief description of each alternative follows.

TABLE 7-11
Principal Treatment System Alternatives

Alternatives	
1. Expand Existing WWTP, Winter Discharge to Willamette River, Summer Storage	
Deficiency	Method of Addressing Deficiency
Pump Station Forcemains	Upsize individual pump station forcemains. Upsize Existing 16-inch primary forcemain. Construct new primary forcemain to WWTP to convey flows from 1st & Monaco and 3rd & Maple.
Hydraulic Capacity	Additional facultative lagoons sized to provide required storage volume.
Organic Capacity	Remove high-strength industrial user from system. Hydraulically sized facultative lagoon sufficient to provide organic treatment through planning period.
Transfer and Outlet Piping	Construct new facilities
Disinfection System, Effluent Flow Measurement and Sampling Equipment	Construct new facilities
Land Application Facilities	Abandon
Outfall/Receiving Stream	Construct new effluent pump station, forcemain, and Willamette River Outfall.

2. Expand Existing WWTP, Winter Discharge to Willamette River, Summer Effluent Reuse	
Deficiency	Method of Addressing Deficiency
Pump Station Forcemains	Upsize individual pump station forcemains. Upsize Existing 16-inch primary forcemain. Construct new primary forcemain to WWTP to convey flows from 1st & Monaco and 3rd & Maple.
Hydraulic Capacity	Additional facultative lagoons sized to provide required storage volume.
Organic Capacity	Remove high-strength industrial user from system. Hydraulically sized facultative lagoon sufficient to provide organic treatment through planning period.
Transfer and Outlet Piping	Construct new facilities
Disinfection System, Effluent Flow Measurement and Sampling Equipment	Construct new facilities
Land Application Facilities	Purchase additional land new WWTP. Construct new pump station and new distribution mains. Lease irrigated land to private agricultural operator.
Outfall/Receiving Stream	Construct new effluent pump station, forcemain, and Willamette River Outfall.
3. Construct New WWTP, Winter Discharge to Willamette River, Summer Storage	
Deficiency	Method of Addressing Deficiency
Pump Station Forcemains	Upsize individual pump station forcemains. Construct new primary forcemain from 14th and Elm to new WWTP site.
Hydraulic Capacity	Additional facultative lagoons sized to provide required storage volume.
Organic Capacity	Remove high-strength industrial user from system. Hydraulically sized facultative lagoon sufficient to provide organic treatment through planning period.
Transfer and Outlet Piping	Construct new facilities
Disinfection System, Effluent Flow Measurement and Sampling Equipment	Construct new facilities
Land Application Facilities	Abandon
Outfall/Receiving Stream	Construct new effluent pump station, forcemain, and Willamette River Outfall.
4. Construct New WWTP, Winter Discharge to Willamette River, Summer Effluent Reuse	
Deficiency	Method of Addressing Deficiency
Pump Station Forcemains	Upsize individual pump station forcemains. Construct new primary forcemain from 14th and Elm to new WWTP site.
Hydraulic Capacity	Additional facultative lagoons sized to provide required storage volume.
Organic Capacity	Remove high-strength industrial user from system. Hydraulically sized facultative lagoon sufficient to provide organic treatment through planning period.
Transfer and Outlet Piping	Construct new facilities
Disinfection System, Effluent Flow Measurement and Sampling Equipment	Construct new facilities
Land Application Facilities	Purchase land adjacent to new WWTP site. Construct new pump station and new distribution mains. Lease irrigated land to private agricultural operator.
Outfall/Receiving Stream	Construct new effluent pump station, forcemain, and Willamette River Outfall.

7.12.1 Alternative 1 – Expand Existing WWTP, Winter Discharge, Summer Storage.

This alternative includes expanding the existing WWTP. All wastewater collected in the City would be pumped to the existing headworks structure. The City would construct new facultative lagoons to provide the organic treatment and hydraulic storage capacity. Two 23.5 acre lagoons would be constructed on the south side of the existing lagoons. The new lagoons would have minimum and maximum water depths of 2 and 8 feet respectively. Plant inflows would be equally split between the two existing cells and two parallel treatment trains would exist. Effluent from the existing primary cells would be routed to the secondary cells. The secondary cells would act as polishing lagoons.

Based on the wetland delineation performed as part of the NEPA environmental report for the project, most of the land proposed for the two new lagoons showed characteristics of jurisdictional wetland. For details of the delineation work, the reader is referred to the stand-alone NEPA environmental report that accompanies this document. The construction of the new lagoons south of the existing lagoons will impact approximately 53 acres of wetlands. Based on discussions with the Oregon State Division of State Lands, wetland credits are currently available at a mitigation bank near Springfield. The cost per acre of credit is $48,000. Therefore, a total of $2,544,000 is included in the construction cost for this alternative for wetland mitigation. Due to the expense of the wetland mitigation, additional study to identify modifications to the recommended plan that may decrease wetland impact is warranted during the predesign phase of the project. A more detailed discussion of the wetland issue including potential modifications to the recommended plan that may reduce the wetland impacts is included in **Section 8.4**.

The improvements would include new transfer piping to convey wastewater from the existing cells to the new cells. A transfer structure would be constructed in each of the existing lagoons to control the flow of wastewater through the transfer piping. The transfer structures would be designed to permit the withdrawal of water from multiple elevations. Outlet structures in the two secondary cells would be constructed to control effluent flowrates. From the outlet structures, effluent would be routed to the effluent pump station. End suction centrifugal or vertical turbine pumps would be used to pump the treated effluent to the Willamette River. The pumps and pump discharge piping would be located in a room inside a larger wastewater treatment plant building. In addition to the pumps and discharge piping, the pump room would also house the pump control valves, an in-line effluent flow meter and the pump control panel. The wastewater treatment plant building would have separate rooms to house an auxiliary power unit, chlorination equipment, electrical controls, laboratory, restroom, and office space.

A gas chlorine feed system would be used to disinfection plant effluent. Chlorine contact time would be provided in the effluent forcemain. The effluent forcemain would discharge into a multiport diffuser. The diffuser would be designed to provide the mixing required to satisfy ammonia and chlorine toxicity requirements. Therefore, no dechlorination facilities are envisioned. A preliminary layout of the proposed improvements is shown in **Figure 7-3**. The recommended design criteria for this alternative are included in **Table 7-12**. A cost analysis for this alternative is included in **Section 7.12**.

TABLE 7-12
Treatment Alternative 1 Design Criteria
Expand Existing Plant, Winter Discharge to Willamette River, Summer Storage

Category	Design Criteria
• Design Year • ADWF, AAF, AWWF • Average BOD and TSS Loading	• 2029 • 1.039 mgd, 1.801mgd 2.577 mgd • 2,110 ppd, 2,300 ppd
Regulatory Requirements • Winter Discharge Period • Summer Storage Period • Effluent BOD and TSS Concentrations • Effluent E.coli	 • November 1 – April 30 • May 1 – October 31 • 30 mg/L, 50 mg/L • 126 / mL
Facultative Lagoons • Existing Lagoon Area • Existing Lagoon Max Water Depth, Min Water Depth • New Lagoon Area • Total Lagoon Area • New Lagoons Max Water Depth, Min Water Depth	 • 47 acres • 6 feet, 2 feet • 47 acres • 94 acres • 8 feet, 2 feet
Effluent Flow and Load • Peak Discharge Rate • Mass Load Limits BOD/TSS (1)	 • 5.3 MGD • 990 ppd BOD, 1650 ppd TSS
Disinfection • Type • Contact Chamber • Contact Volume • Contact Time at Peak Discharge Rate • Design Effluent Chlorine Residual • Peak Chlorine Consumption	 • Chlorine Gas Disinfection • 22,000 feet of 20-inch effluent forcemain. • 358,800 gallons • 97 minutes • 1.5 mg/L • 65 pound per day
New Forcemain Diameters • 9th & Ivy • 17th & Ivy • 3rd & Maple • 10th & Rose • Rosewood • 1st & Monaco • Existing Primary F.M. (Chapel Creek to West 10th and New Primary F.M. connection point) • New Primary F.M. (3rd & Maple, West 10th and Existing Primary F.M. connection point)	 • 8 inch • 6 inch • 14 inch • 8 inch • 6 inch • 10 inch • 18 inch • 16 inch
(1) Mass load limits based upon the AWWF plus an allowance for disposal of summer accumulations of treated wastewater as well as winter stormwater impacting the lagoon surface.	

7.12.2 Alternative 2 – Expand Existing WWTP, Winter Discharge, Summer Reuse.

This alternative includes expanding the existing WWTP. All wastewater collected in the City would be pumped to the existing headworks structure. The City would construct a new facultative lagoon to provide the organic treatment and hydraulic storage capacity. A single new 23.5 acre lagoon would be constructed on the south side of the existing lagoons on land owned by the City. The new lagoon would have minimum and maximum water depths of 2 and 8 feet respectively. Plant inflows would be equally split between the two existing cells and the new lagoon would act as a polishing lagoon.

Based on the wetland delineation performed as part of the NEPA environmental report for the project, most of the land proposed for the two new lagoon is jurisdictional wetland. For details of the delineation work, the ready is referred to the stand-alone NEPA environmental report that accompanies this document. For cost comparison purposes, it was assumed that the construction of the new lagoon will impact approximately 27 acres of wetlands. Based on discussions with the Oregon State Division of State Lands, wetland credits are currently available near Springfield. The cost per acre of credit is $48,000. Therefore, a total of $1,296,000 is included in the construction cost for this alternative for wetland mitigation.

Effluent from the existing cells would be routed to the new polishing lagoon cell. The improvements would include new transfer piping to convey wastewater from the existing cells to the new cell. A transfer structure would be constructed in each of the existing lagoons to control the flow of wastewater between the two primary cells and the new polishing lagoon. The transfer structure would be designed to permit the withdrawal of water from multiple elevations. An outlet structure in the secondary cell would be constructed to control effluent flowrates. From the outlet structure, effluent would be routed to either the effluent pump station for winter discharge, or to a chlorine contact chamber for summer land application.

During the winter discharge season, end suction centrifugal or vertical turbine pumps would be used to pump the treated effluent to the Willamette River. The pumps and pump discharge piping would be located in a room inside a larger wastewater treatment plant building. A gas chlorine fee system would be used to disinfection plant effluent. Chlorine contact time would be provided in the effluent forcemain between the WWTP site and the outfall. The effluent forcemain would discharge into a multiport diffuser. The diffuser would be designed to provide the mixing required to comply with chlorine toxicity requirements. Therefore, no dechlorination facilities are envisioned at this time.

During the summer months, plant effluent would be land-applied. Effluent from the secondary lagoon would be routed to a chlorine contact chamber. Chlorine would be added at the upstream end of the chamber. Vertical turbine or end suction centrifugal pumps would be installed at the downstream end of the contact chamber. These irrigation pumps would pressurize a distribution system that included irrigation risers at strategic locations around the reuse site. The irrigated land would be leased to an agricultural operator who would have complete control over crop selection and would keep all proceeds from the agricultural operation. It is recommended that the City plan the land application facilities for incremental expansion. As part of the initial project, it is recommended that the City purchase enough land to meet the land application needs of the community in the year 2020 assuming that a crop requiring low water consumption is planted. In this way, should agricultural markets lean toward crops that utilize large amounts of water, the City may delay the purchase of additional land for reuse accordingly. A total of 230 acres of land would be required as part of the first stage. The second stage would include the development of an additional 70 acres.

As previously mentioned, the project would include the construction of a new wastewater treatment plant building. In addition to the effluent pumps and controls, the building would have separate rooms to house an auxiliary power unit, chlorination equipment, electrical controls, laboratory, restroom, and office space.

A preliminary layout of the proposed improvements is shown in **Figure 7-4**. The recommended design criteria for this alternative are included in **Table 7-13**. A cost analysis for this alternative is included in **Section 7.12**.

TABLE 7-13
Treatment Alternative 2 Design Criteria
Expand Existing Plant, Winter Discharge to Willamette River, Summer Land Application

Category	Design Criteria
• Design Year • ADWF, AAF, AWWF • Average BOD and TSS Loading	• 2029 • 1.039 mgd, 1.801mgd 2.577 mgd • 2,110 ppd, 2,300 ppd
Regulatory Requirements • Winter Discharge Period • Summer Storage/Land Application Period • Effluent BOD and TSS Concentrations • Effluent E.coli	 • November 1 – April 30 • May 1 – October 31 • 30 mg/L, 50 mg/L • 126 / mL
Facultative Lagoons • Existing Lagoon Area • Existing Lagoon Max Water Depth, Min Water Depth • New Lagoon Area • Total Lagoon Area • New Lagoons Max Water Depth, Min Water Depth	 • 47 acres • 6 feet, 2 feet • 23.5 acres • 70.5 acres • 8 feet, 2 feet
Effluent Flow and Load • Peak Discharge Rate to River • Mass Load Limits BOD/TSS (1)	 • 4.5 MGD • 890 ppd BOD , 1480 ppd TSS
Disinfection • Type • Winter Discharge Contact Chamber • Winter Discharge Contact Volume/Min.Contact Time • Summer Land Application Contact Chamber • Design Effluent Chlorine Residual • Peak Chlorine Consumption	 • Chlorine Disinfection • 22,000 feet of 18-inch effluent forcemain. • 290,650 gallons/93 minutes • Baffled concrete tank sized to provide 30 minutes of contact time at peak irrigation rates. • 1.5 mg/L • 56 pound per day
Effluent Reuse System • Seasonal Water Consumption • Effluent Reuse Area	 • 20 inches/acre/season • Stage I: 230 acres, Stage II 70 acres
New Forcemain Diameters • 9th & Ivy • 17th & Ivy • 3rd & Maple • 10th & Rose • Rosewood • 1st & Monaco • Existing Primary F.M. (10th & Rose to West 10th and New Primary F.M. connection point) • New Primary F.M. (3rd & Maple West 10th and Existing Primary F.M. connection point)	 • 8 inch • 6 inch • 14 inch • 8 inch • 6 inch • 10 inch • 18 inch • 16 inch
(1) Mass load limits based upon the AWWF plus an allowance for disposal of fall season accumulations of treated wastewater as well as winter stormwater impacting the lagoon surface.	

7.12.3 Alternative 3 – Construct New WWTP, Winter Discharge, Summer Storage.

This alternative includes the construction of a new plant east of the City at the site identified above. This alternative includes abandoning the existing WWTP site and selling the land on the open market. Under this alternative, all wastewater collected in the City would be pumped to a new headworks structure at the new WWTP. For all practical purposes, the new WWTP would be identical to that envisioned for Alternative 1, but in a new location. The City would construct new facultative lagoons to provide the organic treatment and hydraulic storage capacity. Four new 21 acre lagoons would be constructed at the new WWTP site. The lagoons would have minimum and maximum water depths of 2 and 8 feet respectively. A new headworks structure with new flow measurement flume would be constructed. The grinder, sampler, and flow meter at the existing plant would be salvaged. The headworks would be designed to split plant inflows between the first two primary cells. Effluent from these cells would be routed to the two secondary cells. In this way, two parallel treatment trains would exist. The secondary cells would act as polishing lagoons.

The improvements would include transfer piping to convey wastewater from the primary cells to the secondary cells. A transfer structure would be constructed in each of the primary lagoons to control the flow of wastewater through the transfer piping. The transfer structures would be designed to permit the withdrawal of water from multiple elevations. Outlet structures in the two secondary cells would be constructed to control effluent flowrates. From the outlet structures, effluent would be routed to the effluent pump station. End suction centrifugal or vertical turbine pumps would be used to pump the treated effluent to the Willamette River. The pumps and pump discharge piping would be located in a room inside a larger wastewater treatment plant building. In addition to the pumps and discharge piping, the pump room would also house the pump control valves, an in-line effluent flow meter and the pump control panel. The wastewater treatment plant building would have separate rooms to house an auxiliary power unit, chlorination equipment, electrical controls, laboratory, restroom, and office space.

A gas chlorine fee system would be used to disinfection plant effluent. Chlorine contact time would be provided in the effluent forcemain between the WWTP site and the Willamette River. The effluent forcemain would discharge into a multiport diffuser. The diffuser would be designed to provide the mixing required to satisfy ammonia and chlorine toxicity requirements. Therefore, no dechlorination facilities are envisioned. A preliminary layout of the proposed improvements is shown in **Figure 7-5**. The recommended design criteria for this alternative are included in **Table 7-14**. A cost analysis for this alternative is included in **Section 7.12**.

TABLE 7-14
Treatment Alternative 3 Design Criteria
Construct New WWTP, Winter Discharge to Willamette River, Summer Storage

Category	Design Criteria
• Design Year • ADWF, AAF, AWWF • Average BOD and TSS Loading	• 2029 • 1.039 mgd, 1.801mgd 2.577 mgd • 2,110 ppd, 2,300 ppd
Regulatory Requirements • Winter Discharge Period • Summer Storage Period • Effluent BOD and TSS Concentrations • Effluent E.coli	 • November 1 – April 30 • May 1 – October 31 • 30 mg/L, 50 mg/L • 126 / mL
Facultative Lagoons • Lagoon Configuration • Total Lagoon Area • Max Water Depth, Min Water Depth	 • Two parallel trains each with a primary and secondary Lagoon • 4 @ 21 acres/each = 84 acres • 8 feet, 2 feet
Effluent Flow and Load • Peak Discharge Rate • Mass Load Limits BOD/TSS (1)	 • 5.0 MGD • 990 ppd BOD , 1650 ppd TSS
Disinfection • Type • Contact Chamber • Contact Volume • Contact Time at Peak Discharge Rate • Design Effluent Chlorine Residual • Peak Chlorine Consumption	 • Chlorine Gas Disinfection • 3,800 feet of 20-inch effluent forcemain. • 108,000 gallons • 31 minutes • 1.5 mg/L • 63 pound per day
New Forcemain Diameters • 9th & Ivy • 17th & Ivy • 3rd & Maple • 10th & Rose • Rosewood • 1st & Monaco • New Primary F.M. (14th & Elm to New South Primary F.M. connection point) • New Primary F.M. (1st & Monaco to New North Primary F.M. connection point) • New Primary F.M. (Primary F.M. connection point to new WWTP Site)	 • 8 inch • 6 inch • 14 inch • 8 inch • 6 inch • 10 inch • 24 inch • 16 inch • 30 inch
(1) Mass load limits based upon the AWWF plus an allowance for disposal of summer accumulations of treated wastewater as well as winter stormwater impacting the lagoon surface.	

7.12.4 Alternative 4 – Construct New WWTP, Winter Discharge, Summer Reuse

This alternative includes the construction of a new plant east of the City at the site identified above. This alternative includes abandoning the existing WWTP site and selling the land on the open market. Under this alternative, all wastewater collected in the City would be pumped to a new headworks structure at the new WWTP. For all practical purposes, the new WWTP would be identical to that envisioned for Alternative 2, but in a new location. The City would construct new facultative lagoons to provide the organic treatment and hydraulic storage capacity. A total of three new lagoons would be constructed. All of the lagoons would be 23.5 acres in size. Flow would be equally split between the first two primary cells. Effluent from the primary cells would be routed to the third 23.5 acre polishing lagoon. All of the new lagoons would have minimum and maximum water depths of 2 and 8 feet respectively.

The improvements would include transfer piping to convey wastewater from the two primary cells to the secondary cell. A transfer structure would be constructed in each of the primary cells to control the flow of wastewater to the secondary cell. The transfer structures would be designed to permit the withdrawal of water from multiple elevations. An outlet structure in the secondary cell would be constructed to control effluent flowrates. From the outlet structure, effluent would be routed to either the effluent pump station for winter discharge, or to a chlorine contact chamber for summer land application.

During the winter discharge season, end suction centrifugal or vertical turbine pumps would be used to pump the treated effluent to the Willamette River. The pumps and pump discharge piping would be located in a room inside a larger wastewater treatment plant building. A gas chlorine fee system would be used to disinfection plant effluent. Chlorine contact time would be provided in the effluent forcemain between the WWTP site and the outfall. The effluent forcemain would discharge into a multiport diffuser. The diffuser would be designed to provide the mixing required to comply with chlorine toxicity requirements. Therefore, no dechlorination facilities are envisioned at this time.

During the summer months, plant effluent would be land-applied. Effluent from the secondary lagoon would be routed to a chlorine contact chamber. Chlorine would be added at the upstream end of the chamber. Vertical turbine or end suction centrifugal pumps would be installed at the downstream end of the contact chamber. These irrigation pumps would pressurize a distribution system that included irrigation risers at strategic locations around the reuse site. The irrigated land would be leased to an agricultural operator who would have complete control over crop selection and would keep all proceeds from the agricultural operation. It is recommended that the City plan the land application facilities for incremental expansion. As part of the initial project, it is recommended that the City purchase enough land to meet the land application needs of the community in the year 2020 assuming that a crop requiring low water consumption is planted. In this way, should agricultural markets lean toward crops that utilize large amounts of water, the City may delay the purchase of additional land for reuse accordingly. A total of 235 acres of land would be required as part of the first stage. The second stage would include the development of an additional 70 acres.

As previously mentioned, the project would include the construction of a new wastewater treatment plant building. In addition to the effluent pumps and controls, the building would have separate rooms to house an auxiliary power unit, chlorination equipment, electrical controls, laboratory, restroom, and office space.

A preliminary layout of the proposed improvements is shown in **Figure 7-6**. The recommended design criteria for this alternative are included in **Table 7-15**. A cost analysis for this alternative is included in **Section 7.12**.

TABLE 7-15 Treatment Alternative 4 Design Criteria Construct New WWTP, Winter Discharge to Willamette River, Summer Effluent Reuse	
Category	Design Criteria
• Design Year • ADWF, AAF, AWWF • Average BOD and TSS Loading	• 2029 • 1.039 mgd, 1.801mgd 2.577 mgd • 2,110 ppd, 2,300 ppd
Regulatory Requirements • Winter Discharge Period • Summer Storage/Land Application Period • Effluent BOD and TSS Concentrations • Effluent E.coli	 • November 1 – April 30 • May 1 – October 31 • 30 mg/L, 50 mg/L • 126 / mL
Facultative Lagoons • Lagoon Configuration • Total Lagoon Area • Max Water Depth, Min Water Depth	 • Two primary and one secondary lagoon • 3 @ 23.5 acres/each = 70.5 acres • 8 feet, 2 feet
Effluent Flow and Load • Peak Discharge Rate to River • Mass Load Limits BOD/TSS (1)	 • 4.7 MGD • 930 ppd BOD , 1550 ppd TSS
Disinfection • Type • Winter Discharge Contact Chamber • Winter Discharge Contact Volume/Min.Contact Time • Summer Land Application Contact Chamber • Design Effluent Chlorine Residual • Peak Chlorine Consumption	 • Chlorine Disinfection • 3,800 feet of 20-inch effluent forcemain. • 108,000 gallons /34 minutes • Baffled concrete tank sized to provide 30 minutes of contact time at peak irrigation rates. • 1.5 mg/L • 58 pound per day
Effluent Reuse System • Seasonal Water Consumption • Effluent Reuse Area	 • 20 inches/acre/season • Stage I: 230 acres, Stage II 70 acres
New Forcemain Diameters • 9th & Ivy • 17th & Ivy • 3rd & Maple • 10th & Rose • Rosewood • 1st & Monaco • New Primary F.M. (14th & Elm to New South Primary F.M. connection point) • New Primary F.M. (1st & Monaco to New North Primary F.M. connection point) • New Primary F.M. (Primary F.M. connection point to new WWTP Site)	 • 8 inch • 6 inch • 14 inch • 8 inch • 6 inch • 10 inch • 24 inch • 16 inch • 30 inch
(1) Mass load limits based upon the AWWF plus an allowance for disposal of fall season accumulations of treated wastewater as well as winter stormwater impacting the lagoon surface.	

7.13. Evaluation of Principal Treatment System Alternatives

As described above, four alternatives have been identified to address the treatment system deficiencies. In this subsection, each alternative is compared to arrive at the best treatment plan. The project costs for each of the alternatives are listed in **Table 7-16**. The basis for the cost estimates is described in **Section 3**. Both capital and annual power costs required for pumping are estimated for each alternative. For comparison purposes, it was assumed that only power costs would vary amongst the alternatives. Annual labor and material costs were not included in the analysis. Each of the alternatives has similar basic mechanical components. As such, it is believed the labor and material costs will be approximately the same for each alternative. The capital costs are the total project costs including construction costs, engineering and surveying costs, administration costs, legal costs, permitting costs, and financing costs. A detailed breakdown of the capital costs is presented in **Table E-2** of **Appendix E**.

TABLE 7-16
Cost Comparison of Principal Treatment Alternatives

Item	Estimated Probable Cost	Oversize Cost Required for Future Growth
Alternative 1: Expand Existing WWTP, Winter Discharge to Willamette River, Summer Storage		
Treatment System	$17,191,000	$12,929,000
Forcemains	$4,354,000	$3,892,000
Alternative 1: Total Project Cost	**$21,545,000**	**$16,822,000**
Estimated Annual Power Costs Associated with Pumping	**$95,000**	**N/A**
Alternative 2: Expand Existing WWTP, Winter Discharge to Willamette River, Summer Land Application		
Treatment System	$16,583,000	$12,258,000
Forcemains	$4,354,000	$3,892,000
Alternative 2: Total Project Cost	**$20,937,000**	**$16,150,000**
Estimated Annual Power Costs Associated with Pumping	**$98,000**	**N/A**
Alternative 3: Construct New WWTP, Winter Discharge to Willamette River, Summer Storage		
Treatment System	$15,910,000	$9,056,000
Forcemains	$4,926,000	$2,888,000
Alternative 3: Total Project Cost	**$20,836,000**	**$11,944,000**
Estimated Annual Power Costs Associated with Pumping	**$69,000**	**N/A**
Alternative 4: Construct New WWTP, Winter Discharge to Willamette River, Summer Land Application		
Treatment System	$17,896,000	$11,287,000
Forcemains	$4,926,000	$2,888,000
Alternative 4: Total Project Cost	**$22,822,000**	**$14,175,000**
Estimated Annual Power Costs Associated with Pumping	**$72,000**	**N/A**
(1) Costs are in 2006 dollars and assume dry weather construction, publicly bid project, ENR 20 cities index = 7883. See Section 3.7 for basis of project cost estimates (i.e., 15% construction contingency, 20% engineering, 10% legal, permits, easement, and administration)		

The advantages and disadvantages of each of the treatment system alternatives are listed in **Table 7-17**.

TABLE 7-17
Comparison of Principal Treatment Alternatives

Advantages	Disadvantages
Alternative 1: Expand Existing WWTP, Winter Discharge to Willamette River, Summer Storage	
• Same as existing treatment technology	• Higher pumping costs
• Simple to operate	• Largest lagoon area
• Lowest land acquisition requirements	• Less suitable for expansion beyond planning period
• Treatment facilities remain in existing location	• High wetland impacts
• Captures residual value in existing facilities	• More forcemain piping to construct and maintain
• Higher potential for cost savings associated with using clay liner in lieu of synthetic liner for lagoon construction	• Inefficient wastewater conveyance
Alternative 2: Expand Existing WWTP, Winter Discharge to Willamette River, Summer Land Application	
• Same as existing treatment technology	• Higher pumping costs
• Simple to operate	• Large lagoon area
• Treatment facilities remain in existing location	• Less suitable for expansion beyond planning period
• Captures residual value in existing facilities	• Treatment plant adjacent to future residential land
• Higher potential for cost savings associated with using clay liner in lieu of synthetic liner for lagoon construction	• More forcemain piping to construct and maintain
	• Reliance on land application and agricultural operator
	• Greater land requirements
	• Inefficient wastewater conveyance
Alternative 3: Construct New WWTP, Winter Discharge to Willamette River, Summer Storage	
• Same as existing treatment technology	• Requires relocating treatment plant to new site
• Simple to operate	• High land acquisition requirements and uncertainty
• Lowest pumping costs	• Greater political challenges
• Small lagoon area	• Greater land use issues
• Less forcemain piping to construct and maintain	• Does not capture residual value of existing facilities
• Potential for revenue from land sale of existing WWTP site.	• Lower potential for cost savings associated with using clay liner in lieu of synthetic liner for lagoon construction
• More suitable for expansion beyond planning period	
• Efficient wastewater conveyance	
Alternative 4: Construct New WWTP, Winter Discharge to Willamette River, Summer Land Application	
• Same as existing treatment technology	• Requires relocating treatment plant to new site
• Simple to operate	• Highest land acquisition requirements
• Low pumping costs	• Greater political challenges
• Smallest lagoon area	• Greater land use issues
• Less forcemain piping to construct and maintain	• Does not capture residual value of existing facilities
• Potential for revenue from land sale of existing WWTP site.	• Lowest potential for cost savings associated with using clay liner in lieu of synthetic liner for lagoon construction
• More suitable for expansion beyond planning period	• Reliance on land application and agricultural operator
• Efficient wastewater conveyance	

The costs for the treatment alternatives are similar. Alternative 3 is the least costly at approximately $22.8 million dollars. Alternative 4 is the most costly at approximately $22.8 million. This spread is not believed to be significant given the uncertainty associated with planning level estimates of this nature. In other words, considering the margin of error for the methods used to estimate the total project costs, all four alternatives cost approximately the same. No single alternative stands out on purely a cost comparison basis. The treatment alternatives that include expanding the existing plant (i.e., Alternatives 1 and 2) capture the residual value of the existing treatment facilities. This is demonstrated in **Table 7-16** by a comparison of the oversize cost required for growth. Alternatives 1 and 2 (i.e., the "expansion alternatives") utilize the existing facilities to meet existing demands. The additional capacity is necessary to accommodate growth only. Whereas in Alternatives 3 and 4 (i.e., the "relocation alternatives") completely new facilities must be constructed to meet existing demands. Alternatives 1 and 2 would permit a greater portion of the project to be funded through future SDC fees. This would lessen the burden on the existing ratepayers.

For all practical purposes, the costs of the four alternatives are approximately equal. Therefore, the selection of a preferred alternative must be based on factors other than cost. As mentioned above, one advantage of the expansion alternatives is that a greater portion of the project may be funded through the collection of SDC fees. There are several other advantages associated with the expansion alternatives. The community is accustomed to the WWTP in its current location. The local residents are aware that the treatment plant exists. The need to expand the existing plant is a reasonable fact that is easily justifiable. On the other hand, relocating the treatment facilities would involve introducing a potentially controversial land use into a new area. The relocation alternatives would likely fall under increased scrutiny from the nearby property owners. Therefore, gaining public approval for the relocation alternatives would be more difficult than for the expansion alternatives. The relocation alternatives also include a significant amount of land acquisition. Virtually all of area between the City and the Willamette River is utilized for farming by well-established agricultural operators. Therefore, the relocation alternatives would involve displacing at least one long-standing agricultural operation. The relocation alternatives also involve locating an urban use (i.e., municipal wastewater treatment) outside of the City's UGB. Statewide land use goals generally discourage urban uses outside of urban areas. As such, the relocation alternatives would be more difficult to permit. For these and other reasons, the relocation alternatives are likely to be generally more difficult to implement than one of the expansion alternatives. As such, the expansion alternatives are generally believed to be more appropriate than the relocation alternatives. Though the costs of the two expansion alternatives are similar, it is likely that Alternative 1 would be slightly less costly than Alternative 2. Alternative 2 would also require the acquisition of significantly more land than Alternative 1. Therefore, Alternative 2 is likely to be more difficult to implement. Since Alternative 1 includes no land application facilities operation is more simple and requires less labor.

In order to select a preferred alternative, the authors worked with City Staff and the City Council. Draft versions of the above information were presented to Public Works Committee and the City Council at two separate public meetings in the spring of 2005. Through this work, the City reached a general consensus that Alternative 1 is the most feasible and in the best long-term interest of the City. As such, Alternative 1 is the preferred alternative that will be carried forward through the remainder of this document.