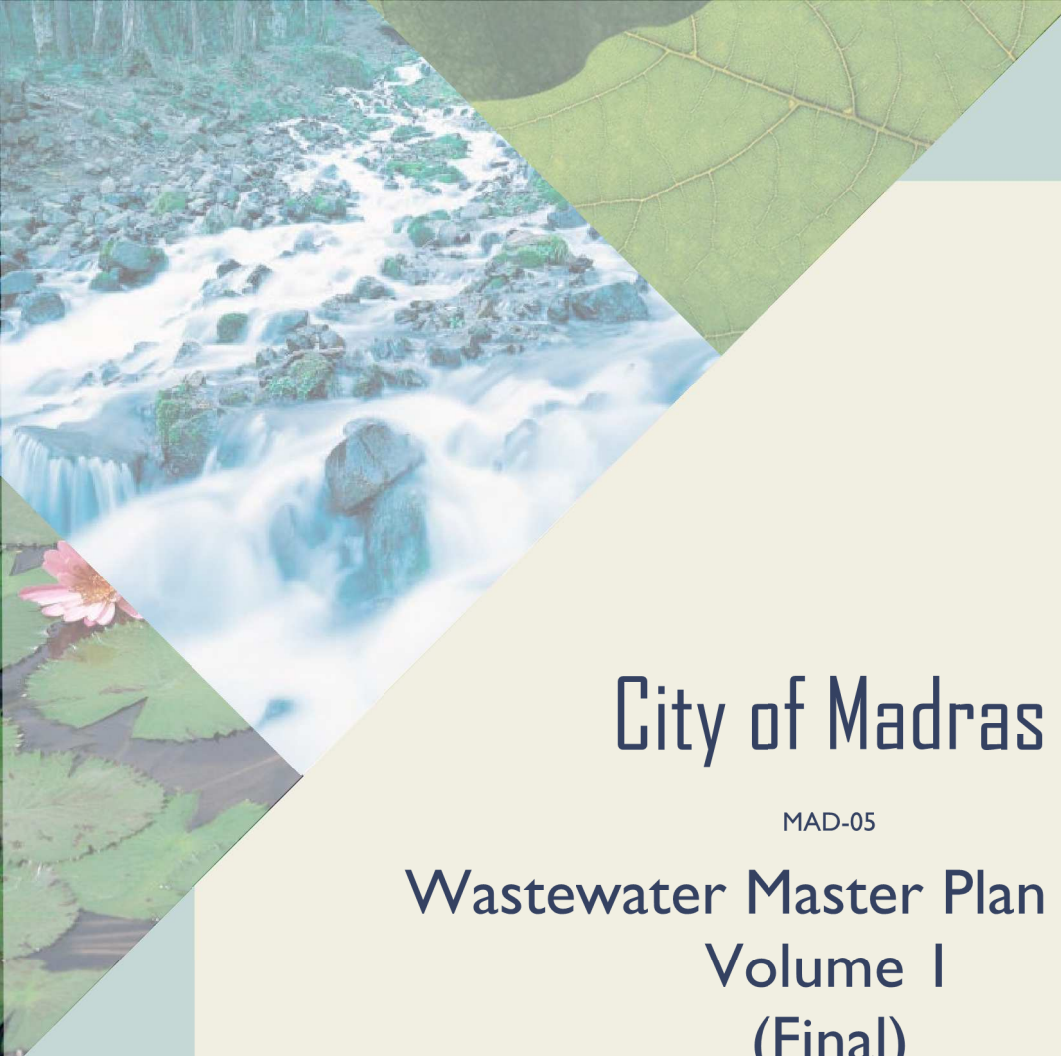




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City of Madras

MAD-05

Wastewater Master Plan Update Volume I (Final)

May 2018

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CITY OF MADRAS

WASTEWATER MASTER PLAN UPDATE

VOLUME ONE (FINAL)



EXPIRES 12-31-2019

May 2018

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WASTEWATER MASTER PLAN UPDATE
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EXECUTIVE SUMMARY

S.1 INTRODUCTION

General. This Wastewater Master Plan (WWMP) Update addresses the City's service needs for a 20-year planning period and presents recommendations for meeting those needs. The City had a comprehensive WWMP prepared in 1996 that provided an analysis of the City's wastewater (WW) facilities and many of the recommendations from that original study have been implemented. This update builds on the results of the 1996 study to the extent they still apply.

Planning Area. This study addresses needs for the area inside the City's Urban Growth Boundary (UGB), with a few exceptions. The City serves two customers outside the UGB: the State's Deer Ridge Correctional Institution (DRCI) and the Madras Municipal Airport. Also, the WW treatment, effluent storage and effluent recycling facilities addressed in this study extend beyond the UGB.

Regulations for Wastewater Facilities. The City operates the WW facilities according to a Water Pollution Control Facility permit and the governing rules issued by the State Department of Environmental Quality (DEQ). These rules mainly pertain to treatment requirements at each wastewater treatment plant (WWTP) and recycling practices for the treated effluent and biosolids.

If the City were to pursue a discharge of treated effluent to Willow Creek, water quality standards (WQS) established by the DEQ for the Deschutes Basin would apply. The City would also need to obtain a different type of operating permit from the DEQ under the National Pollution Discharge Elimination System (NPDES).

Guidelines on Airport Impacts. The existing WWTP located adjacent to the municipal airport is affected by Federal Aviation Administration (FAA) guidance that recommends constraints on land uses near airports. These FAA guidelines would impact potential WWTP expansions due to concerns about the attraction of wildlife and any expansion plans would require FAA review.

S.2 EXISTING FACILITIES

S.2.1 *Descriptions*

General. Madras began providing centralized WW service in 1975 with the construction of a collection system and WWTP. Since then, the City has constructed expansions that have reshaped these facilities. The main elements of the existing facilities serving Madras are listed below:

- Two separate collection systems that receive and convey sewage to the City's two WWTPs.
- The North and South Wastewater Treatment Plants (NWWTP and SWWTP).
- Storage ponds and transmission systems that support effluent recycling as irrigation water on farmland and the municipal golf course.
- A supervisory control and data acquisition (SCADA) system.
- Onsite WW treatment and disposal systems on properties not served by or connected to a sewer.

Collection Systems. The City's WW facilities include a Main Collection System serving most of the City and a smaller Industrial Park Collection System. The smaller system serves the industrial/commercial area near the airport plus a group of residences north of Birch Lane and east of the Municipal Golf Course.

Major components of the two collection systems include approximately 208,000 linear feet of gravity sewers, 878 manholes, five pump stations, and close to 43,800 linear feet of force mains.

NWWTP. The NWWTP was constructed as part of the City's original WW facilities and is located on the southwest side of the airport. Upgrades to the plant were completed in the 1990s, but most original components remain in service. The plant capacity is 0.50 million gallons per day (MGD).

The main treatment process is a lagoon system that provides biological treatment. During cold weather, lagoon effluent is pumped to an open pond for storage. During the growing season, clarification and disinfection equipment provide additional treatment for both the lagoon system effluent and the stored effluent from the pond. Following clarification and disinfection, the effluent is recycled through irrigation of City-owned farmland and the Desert Peaks Golf Course.

SWWTP. The SWWTP was constructed in 2001 as part of the improvements recommended in the 1996 WWMP. The plant site is located on the south side of "J" St., between Willow Creek and Grizzly Road. The City expanded the facility in 2008 to treat flows from the DRCI and the current capacity is 0.54 MGD. The plant is configured to accommodate another expansion.

A sequencing batch reactor (SBR) process provides biological treatment and cloth filters polish the SBR effluent. Disinfection equipment provides final treatment before the effluent is either pumped to a storage pond or used to irrigate the adjacent, City-owned farmland.

S.2.2 Condition Assessments

Gravity Sewers and Force Mains. Available information indicates the sewers, force mains and appurtenances in the two collection systems are generally in adequate to good condition. Our analysis also shows the pipes have adequate capacity to handle existing WW flows generated in the City, plus the design flows from the DRCI when that facility is fully occupied. Age should only become a concern for the existing pipes near the end of the 20-year planning period.

Pump Stations. Age and condition concerns for each collection system pump station (PS) are outlined below. The City will need to implement PS renovations during the planning period.

- **"B" Street PS North.** Originally constructed in 1975; overall in poor to good condition. The main areas of concern are the age of original components from the 1970s and problems with pump clogging due to increased amounts of non-degradable materials such as flushable wipes.
- **"B" Street PS South.** Constructed in 2001; overall in adequate to good condition. The main area of concern is rusting of valves and piping in the valve vault.
- **"B" Street Standby Generator Facilities.** Constructed in 1999; overall in good condition.
- **South 97 PS.** Age not available; overall in adequate to good condition. The pumps operate infrequently due to low incoming flows. The main area of concern is rusting of valves.
- **Golf Course PS.** Constructed in 1996; overall in poor to adequate condition. The main areas of concern are the age of the existing facility, the condition of the pumps and valves, and the lack of a SCADA unit for remote monitoring of alarms.
- **Demers PS.** Constructed in 1989-90; overall in poor to adequate condition. The pumps had to be replaced with used units in 2017 after the condition assessment was performed for this study. The main areas of concern are heavy corrosion and overall wear on components in the valve vault.

NWWTP. The condition of the NWWTP is generally adequate and a few pieces of equipment have recently been replaced. However, the replacement aerators the City recently installed are used units. Overall, due to the age of most components, the facility will require major renovations during the planning period to continue to provide reliable service. Since some condition issues would need to be addressed sooner than others, a phased upgrade would be appropriate.

Key condition issues with the NWWTP are summarized below.

- No influent screening to remove debris that can accumulate in the lagoons.
- Sediment buildup in the lagoons that reduces the available volume for treatment.
- Deteriorated components at the lagoon transfer/outlet structures.
- The age and the outdoor location of the clarifier process. Because the equipment is subject to freezing in cold weather, the City cannot operate the process during the winter.
- Some pumping equipment, valves and instrumentation are old and in need of replacement.
- The main electrical service components should be replaced within 10 years.

SWWTP. The overall condition of the SWWTP is good. Structural components and yard piping should generally be adequate for the next 20 years and beyond. The age of most equipment will not become a concern for about 10 years. However, there are some key condition and operating issues that should be addressed within the next 3 years. These are listed below.

- Control system hardware and software components are either obsolete or soon will be.
- The fine-bubble, diffuser tubes in the SBR basins are at the end of their service life.
- The SBR effluent control valves for the two SBR basins do not close tightly and leak.
- The filter backwash pump and skid are heavily corroded and the electric actuators on the control valve may also be subject to corrosion due to their location in a below-grade gallery.
- The performance and efficiency of the aerobic digester process can be improved by installing dissolved oxygen (DO) probes in the digesters and adding new DO control programming.
- The practice of hauling dilute liquid sludge (biosolids) from the SWWTP to the sludge drying beds at the NWWTP can fill up all available drying-bed capacity when unusually cold and wet weather occurs.

S.2.3 Maintenance Plan Recommendations

Recommendations for completing major maintenance tasks and corrective work were developed in cooperation with the City and are presented in this study. The items covered by the recommended plans are non-routine tasks that often must be performed by outside contractors or repair shops. Routine preventive maintenance tasks are not included in the plan recommendations.

S.3 BASIS OF PLANNING

S.3.1 Population and Dwelling Units

Table S-1 (following page) lists the population forecasts published by the Portland State University Population Research Center (PSU), as well as corresponding numbers of dwelling units (DUs). Census data shows the average unit occupancy was about 2.7 people per household (PPH) in 2010 and this unit occupancy is anticipated to decline in the future. Therefore, this study used 2.6 PPH to estimate the projected numbers of DUs.

Table S-1 Madras UGB Forecasts for Population and DUs			
Year	Population	Occupied DUs ⁽¹⁾	AAGR ⁽²⁾
2015	7,484 ⁽³⁾	2,880	---
2020	8,070	3,105	1.52%
2025	8,700	3,345	1.51%
2030	9,268	3,565	1.27%
2035	9,815	3,775	1.15%
2065	12,749	4,905	0.88%

(1) Forecasts for occupied DUs based on an average unit occupancy of 2.6 PPH.

(2) AAGR is calculated for each incremental 5-year period to 2035 and for the 30-year period from 2035 to 2065.

(3) PSU 2015 population estimate for entire area within UGB.

S.3.2 Wastewater Flows

Records for 2010 through 2014 indicate that the aggregate, average unit flow from all users, residential and nonresidential, is approximately 200 gallons per day (gpd)/DU. We applied a factor of 1.2 to this average unit flow to estimate a peak-monthly unit flow of 240 gpd/DU.

Consistent with recent planning forecasts, the WW flow projections in this study assume nonresidential growth will be proportional to residential growth. Therefore, the overall flow per DU is projected to remain constant over the planning period.

Table S-2 lists the WW flow projections used in this study. The WWMP assumes the DRCI will produce its full design flow throughout the planning period due to the absence of any population or flow projections from the Department of Corrections.

Table S-2 Wastewater Flow Projections for Madras UGB					
Year	Flows (MGD) ⁽¹⁾		DRCI Flows (MGD) ⁽²⁾	Total Flows (MGD)	
	Avg. Annual	Peak Month		Avg. Annual	Peak Month
2020	0.55	0.66	0.23	0.78	0.89
2025	0.67	0.80	0.23	0.90	1.03
2030	0.71	0.86	0.23	0.94	1.09
2035	0.76	0.91	0.23	0.99	1.14
2065	0.98	1.18	0.23	1.21	1.41

(1) Average annual and peak monthly flows based on 200 and 240 gpd/DU, respectively.

(2) DRCI flow based on dedicated capacity allotted to DRCI by agreement with ODOC.

To be conservative, the flow projections for the Year 2025 and beyond assume the entire population within the UGB will be served. The forecasts for 2020 assume sewer service will be extended to 25% of the projected population outside the City, but inside the UGB.

S.3.3 Wastewater Strength

We used estimates of the influent WW strength along with flow capacity to establish the basis of planning for WWTP expansions. Concentrations of five-day biochemical oxygen demand (BOD) and total suspended solids (TSS) are the parameters used for this purpose.

Data for both WWTPs and the DRCI show the following WW strengths are appropriate for planning treatment needs.

- **NWWTP – Combined flows:** 300 mg/L average BOD and TSS concentration
- **SWWTP – City flows:** 250 mg/L average BOD and TSS concentration
DRCI flows: 750 mg/L average BOD and TSS concentration

S.4 ANALYSIS OF COLLECTION SYSTEMS

S.4.1 Sewer Modeling

General. Computer models of the City's two collection systems were generated using the InfoSWMM program to analyze sewer-pipe capacities. Sewer data were obtained from the 1996 WWMP model, available City records, and field investigations. We executed models to analyze sewer capacities under both existing and projected flow conditions.

Estimated peak flows were input to the models using estimated distributions of tributary dwellings, the estimated average flow per DU, and a peaking factor. Proposed developments inside the UGB and input from the City were used as the basis for distributing future tributary DUs.

Most existing DUs not connected to the existing systems were added into the models of future conditions. Only the unserved, existing homes most distant from existing sewers were excluded because of the low probability sewers will be extended to these areas within 20 years.

Modeling Results – Main Collection System. The model of existing conditions confirmed the sewers have enough capacity to convey peak flows to the “B” Street Pump Stations and the WWTPs. The model of projected flows showed the following two sections of existing sewers will have insufficient capacity under future conditions.

- **North Wye Sewer.** The 8-inch interceptor sewer extending southwest from U.S. 97 at Cedar Street to the intersection of 4th and Maple Streets.
- **Culver Highway Sewer.** The 8-inch interceptor sewer extending along Marshall Street, Culver Hwy. and 1st Street, between “G” and “B” Streets.

Modeling Results – Industrial Park Collection System. The model of existing conditions confirmed the existing sewers have enough capacity to convey peak flows to the Demers PS and NWWTP. A model of future conditions with projected flows from the planned Willowbrook development also showed no capacity deficiencies in the existing sewers.

As part of industrial site readiness planning, we performed additional analyses to address the impacts of major industrial developments contributing high WW flows. We applied a 1.0-MGD average flow with a peaking factor to model future flows from three currently undeveloped areas in the industrial park. The modeling results show the existing sewers serving these three selected areas do not have enough capacities to convey peak flow from such major users.

The parallel sewers required to carry the major WW contributions we modeled from each area are listed below. These potential sewers are identified to support industrial site readiness planning. The size, length and alignment of a parallel sewer will depend on actual development plans.

- **East Area** – Approximately 4,800 linear feet of 15-inch parallel sewer along Cherry Lane.
- **South Area** – Up to 2,400 linear feet of 18-inch parallel sewer along Demers Drive.
- **North Area** – Approximately 2,240 linear feet of 18-inch parallel sewer along Airport Way.

S.4.2 Sewer Extensions

Main Collection System. Gravity sewers need to be installed in several unsewered areas to extend service to existing housing developments. New sewer extensions will also be needed in three locations to serve proposed housing developments not immediately adjacent to existing sewers. These proposed sewer extensions are listed below.

- **Extensions into Existing Unsewered Areas:** Bel Air/Herzberg Heights (~5,100 linear feet); Jefferson Street/North Unit (~2,900 linear feet); Mountain View/Sky Ridge (~4,500 linear feet); and Juniper Heights (~5,350 linear feet).
- **Extensions to Reach Proposed Developments:** Willow Creek North (~2,500 linear feet); Madras South Town Center (~2,400 linear feet); and Eldorado Estates/The Ridge (~1,800 linear feet).

The pipe size for all the above-listed sewers would be 8-inch. The sewer lengths listed for extensions to future developments exclude the sewers required within the developments since those would be installed by developers.

Industrial Park Collection System. The City has identified two sewer extensions in the Industrial Park to serve infill locations. These projects include 8-inch sewers extending along Hess Street and Mill Street for about 1,670 and 1,430 linear feet, respectively.

S.4.3 Pump Stations

The collection system pump stations and their associated force mains all have adequate capacity for existing conditions. However, PS improvements will be required during the planning period due to condition and/or capacity issues. The following paragraphs summarize projected PS needs.

- **“B” Street PS North.** The “B” Street North PS needs influent screening equipment to reduce maintenance costs from clogging. The existing PS will also need a renovation to replace and upgrade aging components in 11 to 15 years.
- **“B” Street PS South.** Due to age and wear, the City should plan to replace some components within 20 years. This PS will also need to be expanded when the SWWTP is expanded.
- **“B” Street Standby Generator.** The generator equipment is projected to require a major overhaul or replacement within 15 years. Renovation work on the Generator Building should also be scheduled at that time.
- **South 97 PS.** The City should plan to renovate this PS in 6 to 10 years to address age issues.
- **Golf Course PS.** The Golf Course PS will need to be replaced when development of the Willowbrook subdivision proceeds. If that development does not occur, the City should plan to replace the existing PS within 10 years due to its age and condition.
- **Demers PS.** The City will need to replace components in the valve vault and rebuild the pumps in 3-5 years due to condition issues. The City should also plan to replace the PS in 16-20 years.

S.5 TREATMENT ALTERNATIVES DEVELOPMENT AND SCREENING

S.5.1 *Background for Alternatives Development*

NWWTP. Key factors in developing alternatives for the NWWTP are outlined below.

- The existing lagoon system has relatively low operations and maintenance (O&M) costs.
- The facility is strategically located close to both the Industrial Park and farmland used for effluent recycling.
- The NWWTP requires renovation to keep the facility in operation for another 20 years.
- FAA guidelines identify WWTPs as potential wildlife attractants that should not be located near airports unless acceptable mitigation measures are implemented.
- The shallow depth to underlying rock in the area significantly impacts construction costs.

SWWTP. Key factors in developing alternatives for the SWWTP are outlined below.

- The plant has significant useful service life remaining and should provide reliable service for the next 20 years with scheduled renovations of existing equipment .
- The facilities are designed to accommodate another expansion with common-wall construction.
- The site accommodates gravity flow from sewers in the east and southeast parts of the City. Pumping flows from these gravity sewers to a remote site would require costly upgrades.
- The SBR process has higher O&M costs relative to a lagoon system, but requires far less land.

Effluent Uses. The City has always reused treated effluent for irrigation purposes and has never obtained a permit for a surface-water discharge. Incentives for the City to continue the practice of effluent recycling include a successful track record, the local availability of farmland, and a favorable and stable regulatory climate for this beneficial reuse.

Given the above-listed incentives, the scope of this study is based on the premise the City would continue the current practice of effluent recycling. Therefore, this WWMP Update includes only a preliminary examination of the issues impacting a surface-water discharge.

The City would need to have a separate study prepared to investigate the feasibility of a direct or indirect discharge to Willow Creek. Such a feasibility study would need to examine discharge options, the impacts of WQS, DEQ guidance, potential treatment requirements, site constraints, and related issues. At this point, it is uncertain whether DEQ would approve a Willow Creek discharge.

Biosolids Handling. Existing biosolids treatment systems include aerobic digesters at the SWWTP and sludge drying beds at the NWWTP. These facilities are generally in good condition and the digesters have adequate capacity for current solids production. However, liquid biosolids hauled from the SWWTP can fill the sludge drying beds to capacity when cold and wet weather prevents adequate drying from drainage and evaporation. This capacity limitation needs to be addressed.

The City spreads dried biosolids as Class B biosolids on City-owned farmland accordingly to an approved biosolids management plan. The rules and guidelines for land application of Class B biosolids have not changed significantly over the last decade.

S.5.2 Basis for Treatment and Storage Requirements

Influent WW Flow. Table S-3 lists the maximum-monthly and peak-hourly flow projections identified from the basis of planning. The net available treatment capacity listed to the right is the difference between the maximum-monthly flow projection and the existing combined WWTP capacity. Maximum-month flows are projected to reach existing capacity in about 10 years.

Table S-3 Wastewater Treatment Capacity Requirements			
Year	Projected Total Flows (MGD) ⁽¹⁾		Net Available ADF Capacity ⁽²⁾
	Maximum Monthly	Peak Hourly	
2020	0.89	2.43	0.15
2025	1.03	2.75	0.01
2030	1.09	2.91	– 0.05
2035	1.14	3.04	– 0.10
2065 (build-out)	1.41	3.76	– 0.37

(1) Flow projections assume full occupancy of the DRCI and nonresidential flows will increase in proportion with projected growth in residential flows.

(2) Based on the existing total ADF capacity of 1.04 MGD.

Treatment Level. The development of alternatives was based on the premise that the City would continue the following practices:

- Treat WW to produce Class B effluent for recycling by irrigation.
- Process biosolids to produce dried, Class B biosolids for recycling by spreading on farmland.

Effluent Storage. Since effluent can only be recycled during the growing season, all alternatives would require storage capacity for about 6 to 6.5 months of treated effluent. The effluent storage pond needed for each 0.5-MGD of treatment capacity would require a gross area of about 30 acres.

Solids Handling. The continued practice of land applying Class B biosolids as a dry material is the recommended option at this time and was used as the basis for alternatives development. This conclusion is based on the current conditions of the existing biosolids handling facilities, the availability of farmland, and the regulatory climate.

S.5.3 Overall Treatment Scenarios

We identified and screened five overall WW treatment scenarios for serving the City over the next 20 years. These alternative scenarios are summarized below:

- **Scenario One** – No expansion; renovate existing WWTPs to maintain current capacity.
- **Scenario Two** – Abandon both existing plants and construct one or two new WWTPs.
- **Scenario Three** – Abandon one WWTP and expand the other to treat all flows.
- **Scenario Four** – Maintain one WWTP at its current capacity and replace or expand the other.
- **Scenario Five** – Implement modifications to both WWTPs to expand the capacity of each.

Initial screening eliminated Scenarios One, Two and Three from further evaluation because they are not suitable for the City's planning needs. The City should plan for a small, capacity expansion to treat projected flows and additional expansions to serve potential growth in the Industrial Park. At this time, the City should also plan to maintain both WWTPs in service.

Our evaluation shows either Scenario Four or Scenario Five could fit projected WW service needs. However, accelerated industrial development may drive expansions to both plants. Therefore, to address industrial site readiness, we selected Scenario Five as the preferred overall treatment scenario for planning purposes. Table S-4 summarizes this recommended scenario.

Table S-4 Preferred Overall Treatment Scenario
Expand SWWTP to Treat Projected Flows and Dedicate NWWTP to Serve North Area 1. Phased improvements to the SWWTP. • Phase 1. Modify the plant to provide a small increase in capacity to 0.65-MGD and renovate the existing facilities as needed to maintain reliable service. • Phase 2. If necessary, construct the additional 0.5-MGD treatment module for which the SWWTP is designed to provide a total 1.14-MGD ADF capacity. This second phase would free up capacity at the NWWTP to serve industrial developments. 2. Renovate, retrofit, or replace the existing NWWTP to maintain the 0.5-MGD capacity. After SWWTP expansions, the NWWTP would be dedicated to serving the North Area and would be expanded as necessary to serve Industrial Park developments.

The proposed Phase 1 SWWTP expansion would provide an overall treatment capacity that meets the 20-year flow projection. The Phase 2 SWWTP expansion would free-up the entire NWWTP capacity to serve the North Area. This additional expansion would only be necessary if industrial development creates the need for more capacity than currently projected. Otherwise, Phase 2 for the SWWTP would not be justified.

S.5.4 Improvements to Existing NWWTP

An upgrade of the NWWTP is part of the overall treatment scenario because improvements must be made within the planning period to maintain reliable service. The plant could be renovated to keep existing treatment processes in service, retrofitted to modify existing processes, or replaced with a new facility.

The following options for upgrading the 0.5-MGD lagoon system were evaluated in coordination with the alternatives evaluation for future NWWTP expansions. Upgrades and/or replacements of the clarification and disinfection systems would be part of each approach.

1. **Full Renovation.** Renovate the existing lagoons to maintain the same capacity and footprint.
2. **Partial Renovation.** Renovate half the lagoon system to provide a 0.25-MGD and construct an adjacent treatment module to maintain or expand capacity.
3. **Retrofit to Maintain Capacity and Reduce Footprint.** Retrofit half the existing lagoon system with a new, higher-rate system to maintain a 0.5-MGD capacity in about half the existing lagoon footprint.
4. **Retrofit to Increase Capacity.** This retrofit option would convert both primary lagoons to aerated lagoons in two phases to double the plant capacity within the existing footprint.

5. **Complete Replacement.** Construct a new 0.5-MGD WWTP adjacent to the existing facility and abandon all existing components.

Modifications to the NWWTP must address FAA constraints on the siting of wildlife attractants near airports. Plans for any modifications would need to be submitted to FAA for review and, if costly mitigation measures are mandated, the City may need to investigate alternative sites.

S.5.5 Alternative Treatment Processes for NWWTP Expansions (Industrial Site Readiness Planning)

As a first step in determining the most cost-effective approach to expanding the NWWTP, we conducted a technology review to identify alternatives that should be screened for further evaluation. This review identified treatment processes that have been found to be suitable for the required plant size and treatment level in Madras.

We then screened the alternatives we identified by evaluating and comparing the processes with regard to O&M considerations, treatment reliability, safety considerations, and siting constraints. These criteria were used to eliminate processes that do not fit this application well and identify the alternatives that have the potential of being cost effective solutions for Madras.

Table S-5 lists the alternatives that remained after the initial review and screening efforts. These processes were further evaluated in a cost-effectiveness analysis using more detailed criteria.

Table S-5	
NWWTP Alternatives Recommended for Further Evaluation	
1. Preliminary Treatment at Headworks:	Mechanically-Cleaned Screen and Low-Energy, Vortex Grit Chamber
2. Secondary Treatment	a. Renovation of existing Partially-Aerated, Facultative Lagoon System (no stabilization) b. Aerated/Partially Mixed Lagoon System – retrofit and expansion (no stabilization) c. Oxidation Ditch Extended Aeration d. Compact Extended Aeration Process Configuration e. SBR Extended Aeration
3. Disinfection	a. Sodium Hypochlorite Bulk Storage and Feed b. Onsite Sodium Hypochlorite Generation and Feed c. Tablet Chlorination (Calcium Hypochlorite) Equipment
4. Biosolids Stabilization:	Aerobic Digestion
5. Biosolids Dewatering:	Sludge Drying Beds

S.6 EVALUATION OF TREATMENT ALTERNATIVES

S.6.1 Overview of Alternatives Evaluation

Evaluations of alternatives for NWWTP expansions were completed in two stages. We first evaluated the alternative processes that remained after screening. Then we evaluated alternatives for renovating, retrofitting or replacing the existing NWWTP in coordination with expansions.

A separate alternatives evaluation was also performed to identify the most cost effective method for handling SWWTP biosolids to address hauling requirements and drying-bed capacity limitations.

We evaluated alternatives by comparing probable present worth costs, O & M requirements, land use impacts, treatment reliability, operating flexibility, and sustainability. Probable present worth costs include estimates of construction costs, annual O&M costs and salvage values after 20 years. Our analysis used December 2016 dollars.

S.6.2 Development of Alternatives for NWWTP Secondary Treatment and Disinfection

Preliminary process sizing and layouts were developed for the NWWTP alternatives that remained after the screening effort and were used to estimate probable life-cycle costs. Each alternative secondary process requires the following support facilities to provide complete systems.

- a headworks for preliminary treatment;
- secondary clarifiers for biosolids separation and removal;
- aerobic digesters for biosolids stabilization;
- building to house accessory equipment and process controls; and
- sludge drying beds for biosolids dewatering and land application of Class B biosolids.

All disinfection alternatives include a building to house the process equipment, piping and controls.

S.6.3 Summary of Secondary Process Alternatives Comparison and Ranking

We compared the secondary alternatives using both our estimates of probable, present-worth costs and the nonmonetary criteria. The aerated lagoon process had the highest overall ranking based on these criteria and the SBR process, with similar life-cycle costs, was a close second. The other two processes, oxidation ditches and compact extended aeration, had significantly higher life-cycle costs and were ranked lower.

Based on our evaluation, an aerated lagoon system is the most cost-effective secondary treatment alternative to implement at the NWWTP. The main advantages to this process are the lower O&M requirements and the associated cost savings relative to the other alternatives. Therefore, we recommend that the City implement aerated lagoons as the secondary treatment process for a retrofit or replacement of the existing NWWTP and for plant expansions.

Overall, aerated lagoons and SBRs rank close in both costs and nonmonetary factors. If the Federal Aviation Administration (FAA) requires covers over lagoons and basins, then the SBR process could be worth further consideration due to its smaller footprint.

S.6.4 Summary of Disinfection Alternatives Comparison and Ranking

Using both monetary and nonmonetary considerations, our analysis identified the tablet chlorination system as the most cost effective disinfection alternative. A tablet chlorinator is estimated to have a lower life-cycle cost and about 20% lower O&M costs than the bulk storage and feed system the City currently uses.

S.6.5 *Evaluation of Alternatives for Existing NWWTP Facilities*

General. As previously described, the City will need to renovate, retrofit, or replace the existing facility within 15 years. We evaluated alternatives for upgrading the NWWTP in conjunction with an expansion to identify a preferred approach for addressing condition issues while maintaining industrial site readiness. Aerated lagoons were incorporated into the alternatives evaluation as the recommended secondary treatment process for expansions, consistent with our analysis of process alternatives.

The evaluation compared alternatives for providing an expanded NWWTP capacity of 1.0 MGD and also considered site requirements for an additional expansion to 1.5 MGD. The main considerations in evaluating alternative NWWTP upgrades are summarized below:

- the pace of development in the North Area;
- the need for flexibility to accommodate expansions for maintaining industrial site readiness;
- the impacts on the airport that expansions of open lagoons would have;
- the recommended secondary treatment alternative for NWWTP expansions; and
- the availability of farmland near the NWWTP for effluent recycling by irrigation.

Alternatives for Upgrading and Expanding the NWWTP. The alternative approaches to upgrading and expanding the existing NWWTP are summarized below.

1. Option 1 – Full Renovation of Lagoon System and Adjacent Expansion.

- Renovate the existing lagoons in phases to maintain a 0.5-MGD capacity in the same area.
- Expand the capacity to 1.0 MGD by constructing a 0.5-MGD aerated lagoon module and additional effluent storage capacity on adjacent land.

2. Option 2 – Partial Renovation Combined with Adjacent Expansion.

- Renovate half the lagoon system to provide a 0.25-MGD capacity. To expand to 1.0 MGD, construct a 0.75-MGD aerated lagoon module on adjacent land.
- The remainder of the existing lagoon system could be converted to an effluent storage pond as part of the expansion to reduce the required area outside the existing footprint for effluent storage.

3. Option 3 – Retrofit to 0.5-MGD Aerated Lagoons with Adjacent Expansion.

- Retrofit one primary cell of the existing lagoon system to convert it to a 0.5-MGD aerated lagoon system. This retrofit would use a more intensive process to reduce the volume required for a 0.5-MGD capacity.
- Adjacent aerated lagoons would be constructed for expansions and the remainder of the existing lagoon footprint would be converted to effluent storage.

4. Option 4 – Retrofit to Increase Capacity with Aerated Lagoons.

- This retrofit option would convert both existing primary lagoons to the higher-rate aerated lagoons in two phases to double the plant capacity to 1.0 MGD within the existing footprint.
- Effluent storage capacity would need to be constructed on adjacent land for the expansion.

5. Option 5 – Complete Replacement with New Facility.

- This option would replace the existing facilities with an entirely new 0.5-MGD WWTP. The replacement facilities would consist of the same aerated lagoon and disinfection processes we recommend for expansions.
- The existing lagoon footprint would be converted to effluent storage when the plant is expanded to 1.0 MGD.

Common Facilities. Each option would include new headworks, clarification and disinfection equipment, pumps, and valves. The clarifiers would be housed in a new process building for weather protection. The existing sludge drying beds would be expanded when necessary.

Probable Costs for NWWTP Alternatives. Our estimates show the probable present worth costs for Options 1 and 4 are essentially equal and the probable present worth costs for the other three options are estimated to be 5% to 11% higher.

Recommended Approach to NWWTP Upgrades and Expansions. We recommend the City plan to implement Option 4, a phased retrofit to upgrade and expand the NWWTP with aerated lagoons, for the following reasons.

- Option 4 is estimated to have lower O&M costs than the other alternatives while the life-cycle costs are estimated to be approximately equal to Option 1 and lower than the other options.
- Option 4 would eliminate the larger facultative lagoons (secondary cells) that would remain under both Options 1 and 2. Facultative lagoons not only require more area, they also are more prone to algae growth and more likely to act as wildlife attractants.
- There is considerable flexibility in how the aerated-lagoon retrofits can be configured and the types of equipment that can be installed. This provides flexibility in tailoring the design capacity of the retrofits to actual development conditions.
- Aerated lagoons receive more aeration and mixing than the existing lagoon system. This reduces algae growth and provides a less attractive environment for water fowl due to greater surface agitation.

We recommend the implementation of the 0.5-MGD retrofit in phases due to limitations in funding and differing priorities for component replacements. The recommended phasing, our estimates of probable project costs for each phase, and project timing are presented in the capital improvements plan.

S.6.6 *Evaluation of Biosolids Alternatives for Existing SWWTP*

Existing Biosolids Handling. Biosolids must be wasted from the SBR process at the SWWTP and treated in adjacent aerobic digesters. Currently, the treated (digested) biosolids are hauled in a tank truck as a dilute liquid (98.5 – 99.0% water) to the sludge drying beds at the NWWTP. This practice requires the hauling of approximately 500 truckloads of liquid biosolids annually. In the winter, cold and wet conditions can result in no net loss of water. Therefore, the drying beds must effectively store all of the liquid hauled over a period of up to three months.

SWWTP Biosolids Alternatives. We evaluated the following three alternatives for drying digested biosolids before land application by spreading.

- Alternative One – Continue Hauling Dilute Biosolids and Expand Sludge Drying Beds.

- Alternative Two – Add Biosolids Thickening Process at SWWTP and Reduce Scope of Sludge Drying Bed Expansion. Haul thickened liquid with 4 – 5% solids content (95 – 96% water) in a tank truck.
- Alternative Three – Add Biosolids Thickening and Dewatering Processes at SWWTP and Avoid Sludge Drying Bed Expansion. Haul material with about 20% solids content using a dump truck.

The City could potentially use an existing storage building at the SWWTP to house the process equipment required for Alternatives Two and Three.

Recommended SWWTP Biosolids Alternative. Alternative Two, the thickening process, is estimated to have significantly lower life-cycle costs than the other alternatives. This includes both lower capital costs and lower O&M costs. Alternative Two would also reduce fuel consumption and safety concerns relative to current practice by lowering the number of truck trips for hauling biosolids.

Given the clear monetary and nonmonetary benefits to adding a thickening process at the SWWTP, we recommend the City implement Alternative Two. Because the thickener will reduce O&M costs, we propose the City consider this a high-priority improvement.

S.7 EFFLUENT STORAGE AND RECYCLING

S.7.1 General

We developed estimates for future effluent storage and farmland requirements based on the assumption that the City will continue to recycle all effluent flows with no surface water discharge. The following paragraphs summarize our estimates of these projected needs.

S.7.2 Effluent Storage Volume Requirements

Volume Requirement. Our calculations show an effluent storage volume of 79 MG is required in Madras for each 0.5-MGD in average treatment capacity. This estimated volume is based on a water balance using records for the maximum annual precipitation (1983) and minimum annual evaporation (1993).

NWWTP Effluent Storage. Currently, the NWWTP includes one 79-million gallon (MG) storage pond. This means there is adequate volume for the existing capacity, but no surplus storage volume. Therefore, a plant expansion to 1.0 MGD would require the storage volume to be doubled.

SWWTP Effluent Storage. There are two existing storage ponds near the SWWTP with a total volume of 113 MG. This translates to a surplus storage volume of 28 MG at the current design capacity and a 10-MG surplus volume after the recommended expansion to 0.65 MGD. A Phase 2 expansion to 1.15 MGD would require 70 MG of additional storage volume. The existing site of the upper storage pond for the SWWTP has room for the future addition of another pond.

S.7.3 Existing Land Available for Effluent Recycling

NWWTP Sites. The City owns 246 acres of farm fields around the airport that can still potentially be used to recycle NWWTP effluent. The municipal golf course also has about 75 acres that can be irrigated with effluent. Not all of the irrigation demands for crops and the golf course are satisfied using effluent. Water from the North Unit Irrigation District (NUID) is used as a supplemental source.

Some farmland is designated for future development in the around the airport. Overall, about 135 gross acres of farmland would be lost, if airport developments occur and the NWWTP were expanded to a 1.5 MGD capacity. Alternatively, there are tentative plans to expand the golf course and potentially increase available land for irrigation by 43 acres.

SWWTP Sites. The City owns 304 acres of land near the SWWTP that can potentially be used for effluent recycling. Currently, 111 acres are available for irrigation. The other 193 acres are unimproved and are planned as a future golf course site that would be irrigated with recycled effluent. If the land is not developed as a golf course before the City needs it for SWWTP effluent recycling, this site could be used as farmland.

S.7.4 Farmland Requirements for Effluent Recycling

General. We estimated the amount of farmland required for future effluent volumes based on information presented in the effluent reuse plans previously prepared for the City. We also updated the information in these reuse plans based on input from the City regarding effluent uses.

Effluent use for irrigation was estimated using water demand for alfalfa as an average condition. Grass crops generally have higher water demands and grains generally require lesser amounts. Costs for securing additional irrigable land will depend on several factors and further study into available options will be needed to develop planning-level estimates of probable costs.

NWWTP Recycling Farmland Requirements. We project the NWWTP would have a farmland deficit of 37 acres when peak monthly flows reach 0.50 MGD. This estimate accounts for the potential loss of fields near the airport due to commercial/industrial development. Some of this deficit could be offset, if more of the golf-course water demand were met using recycled water. However, the City should plan to identify and purchase at least 42 more acres of farmland to use for irrigation within the planning period. This figure includes allowances for field access and buffers.

Our projections also show the NWWTP site would face a farmland deficit of about 300 acres and 610 acres at future flows of 1.0 MGD and 1.5 MGD, respectively. These deficits would be less, if more of the golf-course demand can be met with recycled water in the future. However, these deficits would each increase by about 20 acres, if the municipal golf course is not expanded as planned.

SWWTP Recycling Farmland Requirements. We project the City would need to make about 125 acres of the upper parcel available as irrigable farmland to recycle the current SWWTP capacity of 0.54 MGD. Therefore, the City should plan to make the future golf course parcel available for irrigation in 6 to 10 years or sooner, if flow contributions from DRCI begin to increase significantly.

With the entire upper parcel available for irrigation as farmland, the City would have a surplus of about 28 acres when peak monthly flows reach an expanded capacity of 0.65 MGD.

We project a farmland deficit of almost 180 acres if the City constructed a Phase 2 SWWTP expansion and peak monthly flows reached 1.15 MGD. This deficit could be reduced, if the future golf course is developed as planned and effluent is used to meet most or all of the irrigation demand. This is because water demands are higher for turf grass than alfalfa.

S.8 RECOMMENDED IMPROVEMENTS AND PLANNING CONSIDERATIONS**S.8.1 *Capital Improvements Plan***

The projects identified in the capital improvements plan (CIP) and the anticipated project timing are listed below. Actual project schedules will depend on a variety of factors.

- 1. Gravity Sewer Projects.** The CIP identifies 10 gravity sewer projects for the collection systems with an estimated total probable project cost of \$7,822,000. These improvements and the projected time frames for their implementation are listed below.
 - A. Within 5 years:** Three projects to bring service to unsewered residential areas (Bel Air/Herzberg Heights, Mountain View/Sky Ridge, and Jefferson Street/North Unit).
 - B. In 6-10 years:** Two sewer extension projects in the Industrial Park to serve unsewered pockets of land along Hess and Mill Streets.
 - C. In 11-15 years:** The North Wye sewer replacement to increase capacity for proposed developments and unsewered areas.
 - D. In 16-20 years:**
 - The Culver Highway parallel sewer to increase system capacity for proposed developments and unsewered areas .
 - The Juniper Heights project to bring service to that unsewered area.
 - The Willow Creek North sewer to extend service to future developments.
 - E. Industrial Site Readiness Planning:** The potential North Area-Airport Way parallel sewer project as the least-cost alternative to serve a major WW contributor in the Industrial Park. The project scope and timing are development dependent.
- 2. Pump Station and Force Main Projects.** The CIP identifies the following PS and force main projects for the two collection systems. The estimated total probable project cost of these improvements is \$5,555,000.
 - A. Within 5 years:** Add an influent mechanical screen at the “B” Street North PS.
 - B. In 6-10 years:** The Golf Course PS replacement with capacity expansion for the planned Willowbrook subdivision (project would be sooner if the Willowbrook development proceeds).
 - C. In 11-15 years:**
 - “B” Street North PS renovation with no capacity expansion.
 - Generator replacement at the “B” Street pump stations.
 - South U.S. 97 PS Renovation with no capacity expansion.
 - D. In 16-20 years:**
 - Demers PS replacement with allowance for future expansion(s).
 - “B” Street South PS renovation and expansion.
 - E. Industrial Site Readiness Planning:** Potential force main replacement project from Demers PS to NWWTP when PS capacity is expanded beyond 0.70 MGD. Project timing is development dependent.

3. **WWTP Projects.** The CIP identifies the following WWTP and effluent storage projects. The estimated total probable project cost of these improvements is \$77,315,000. These improvements are based on the City continuing the current practice of recycling Class B effluent on farmland and the golf course. The NWWTP projects also are contingent on FAA reviews and acceptance.

A. Within 5 years:

- NWWTP Phase 1A improvements to allow reliable year-round operation of the clarifier, disinfection and pumping equipment.

Alternate Phase 1A Project: Our CIP also identifies an alternate project with a more limited scope to allow reliable year-round operation of the NWWTP in the near term. Other work we included in the full Phase 1A improvements would then be completed separately as funds become available.

- A SWWTP biosolids thickening process to reduce biosolids hauling costs and reduce the amount of sludge drying bed capacity that is required.

B. In 6-10 years:

- A SWWTP expansion to increase average design capacity to 0.65 MGD and partial renovation to upgrade aging equipment.
- A SWWTP irrigation system expansion to allow effluent recycling and crop production on the 193-acre site of the future golf course east of the SWWTP site.

C. In 11-15 years:

- NWWTP Phase 1B improvements to retrofit the existing lagoon system with an aerated lagoon system and related facilities with no capacity expansion.
- NWWTP effluent transmission and irrigation system expansion for effluent recycling (add farmland, effluent pipeline, and irrigation system components).

D. In 16-20 years:

- SWWTP Phase 2 expansion and renovation to increase average design flow capacity to 1.15 MGD.
- SWWTP Phase 2 effluent storage and irrigation system expansions to increase effluent recycling capacity (add farmland, effluent pipeline, and irrigation system components).

E. Industrial Site Readiness Planning (scope and timing are development dependent):

- Potential NWWTP Phase 2 expansion
 - increase average design capacity to 1.0 MGD;
 - increase effluent storage by 79 MG; and
 - increase irrigation system capacities (add farmland, effluent pipeline, and irrigation system components).
- Potential NWWTP Phase 3 expansion
 - increase average design capacity to 1.5 MGD;
 - increase effluent storage by another 79 MG; and
 - increase irrigation system capacities (add farmland, effluent pipeline, and irrigation system components).

Table S-6 summarizes the estimated probable project costs for each major project category (sewers, PS and force mains, and WWTPs).

Table S-6 Capital Improvements Summary Estimates of Probable Costs and Anticipated Timing ⁽¹⁾		
Project Description	Probable Construction Cost	Probable Project Cost
A. Short-Term Projects (within 5 years)		
1. Gravity Sewer Projects	\$2,139,000	\$2,885,000
2. Pump Station and Force Main Projects	\$345,000	\$465,000
3. WWTP and Effluent Recycling Projects ⁽²⁾	\$1,931,000	\$2,610,000
Subtotal for Short-Term Projects	\$4,415,000	\$5,960,000
B. Mid-Term Projects (6 – 10 years)		
1. Gravity Sewer Projects	\$480,000	\$650,000
2. Pump Station and Force Main Projects	\$361,000	\$490,000
3. WWTP and Effluent Recycling Projects	\$1,303,000	\$1,760,000
Subtotal for Mid-Term Projects	\$2,144,000	\$2,900,000
C. Mid- to Long-Term Project (11 – 15 years)		
1. Gravity Sewer Projects	\$1,213,000	\$1,640,000
2. Pump Station and Force Main Projects	\$678,000	\$915,000
3. WWTP and Effluent Recycling Projects	\$7,267,000	\$9,810,000
Subtotal for Mid- to Long-Term Projects	\$9,158,000	\$12,365,000
D. Long-Term Projects (16 – 20 years)		
1. Gravity Sewer Projects	\$1,455,000	\$1,965,000
2. Pump Station and Force Main Projects	\$1,869,000	\$2,520,000
3. WWTP and Effluent Recycling Projects	\$16,771,000	\$22,640,000
Subtotal for Long-Term Projects	\$20,095,000	\$27,125,000
E. Potential Industrial Site Readiness Projects ⁽³⁾		
1. Gravity Sewer Projects	\$695,000	\$940,000
2. Pump Station and Force Main Projects	\$2,909,000	\$3,925,000
3. WWTP and Effluent Recycling Projects	\$43,199,000	\$58,315,000
Subtotal for Long-Term Projects	\$46,803,000	\$63,180,000
Total – Proposed & Potential Capital Projects	\$82,615,000	\$111,530,000

(1) Estimates of probable costs are updated to December 2017 dollars. Refer to Chapter 8 for more information on the basis for the estimates of probable cost.

(2) Subtotal includes the alternate NWWTP Phase 1A Improvements project rather than full Phase 1A upgrade described in Chapter 8.

(3) Potential improvements identified for industrial site readiness planning. Actual projects would be driven by development and must be reevaluated based on actual development proposal(s). Capital costs for development-driven projects would typically be covered by developers or other funding sources and not come from the City's capital improvements fund.

S.8.2 Additional Planning Considerations

Planning Effort for Surface-Water Discharge. Before the City proceeds with the Phase 1 SWWTP expansion and renovation, it may be worthwhile to investigate the feasibility of an indirect discharge to Willow Creek. An early step in this effort would be to solicit input from the DEQ regarding the prospects for a discharge permit. Depending on DEQ's input, the City might then want to proceed with a feasibility study to evaluate the potential cost effectiveness of an indirect discharge to Willow Creek through constructed wetlands.

Capacity, Management, Operations and Maintenance (CMOM) Program Planning. The intent of CMOM program planning is to support the City's efforts in properly managing, operating and maintaining the sewer systems and overall WW facilities it owns. The general goals of a formal CMOM program are as follows:

- Assure that adequate sewer capacity remains available.
- Develop a proactive (predictive) approach to system management, operations and maintenance.
- Provide quality customer service to generate and sustain support for adequate local investment in the WW facilities.

Since Madras has had centralized sewer service for over 40 years, the City has been following many recognized CMOM practices for an extended period. The City can improve current practices by conducting a thorough review of their CMOM practices, developing a written CMOM program, and establishing specific objectives for CMOM performance.

We recommend the City conduct an assessment of current CMOM practices to identify areas of improvement. The City should then establish performance objectives in coordination with current priorities for addressing areas of concern. The key is to clearly define the target objectives, develop a process for tracking progress, and determine whether staff can meet these objectives.

The information presented in this WWMP Update can be used as a tool in CMOM program planning and regular WWMP updates can help keep CMOM activities current.

Solar Power Array Funding and Partnering Opportunities. Solar photovoltaic (PV) panels have become increasingly cost competitive as a way to generate electricity onsite and reduce the amount of electricity purchased from a utility. Due to the availability of State and Federal incentives, it may be cost effective for the City to install an array of PV panels at either or both WWTP sites as a means for lowering power costs.

To take advantage of tax incentives, a City could enter into a power purchase agreement (PPA) with a private, solar service provider. Under a PPA, the service provider would furnish, install, own and maintain the power-generating system. In exchange, the City would agree to purchase the electricity produced by the system for the term of the PPA.

The main advantages to a PPA for the City include:

- low initial cost for a solar array;
- the ability to benefit from Federal tax benefits;
- a predictable cost of electricity for the term of the PPA;
- no need to deal with the solar system permitting, design, and construction; and
- no operations and maintenance responsibilities for the array.

Because the NWWTP is adjacent to the airport, a solar array installed at that plant site would need to be designed according to FAA standards and technical guidance. The FAA recognizes the benefits of installing solar power projects at airports and generally supports their implementation. To promote aviation safety, the FAA has studied the visual impacts of solar panels and has issued guidance to address technical considerations.

Master Plan Updates. We recommend the City update the WWMP on a regular basis, particularly when changes occur that are likely to have significant impacts on previous recommendations. In general, the City should plan to have a WWMP update prepared every 5 years.

S.9 WASTEWATER SYSTEM DEVELOPMENT CHARGE UPDATE

S.9.1 General Description

The City levies system development charges (SDCs) as one-time fees on new developments to recover a fair share of the capital costs of facilities that provide capacity for future growth. As part of this master planning effort, FCS GROUP performed an analysis to update the SDCs for wastewater services. A full analysis to update other sewer rates will be completed at a later date.

In general terms, the updated SDC calculation involves dividing eligible costs for capital improvements by the customer-base expansion that can be served by these improvements. The recommended SDCs for wastewater facilities include a reimbursement fee component, an improvement fee component, and an administrative cost component.

S.9.2 Customer Base

Different types of customers have differing service needs. Therefore, the existing and expanded customer base are each estimated on an equivalent basis using water meter size. A standard residential meter has a meter capacity equivalent (MCE) of 1.0 and larger meters have higher MCEs based on their relative capacity.

According to the current breakdown of water meter sizes in Madras, the City's wastewater facilities have an existing MCE of 3,074. Updated flow data and the total future capacity proposed in this Master Plan Update show the recommended wastewater facilities could serve another 5,779 MCEs in future growth. This available growth in customer base was used to calculate the reimbursement and improvement components of the SDCs.

S.9.3 Recommended SDC Components

Reimbursement Fee. The reimbursement fee is based on the City's eligible costs for constructing that portion of the existing wastewater facilities capacity that is unused and available for future growth. Currently, 30% of the existing capacity is available and the net eligible cost basis for this unused capacity is approximately \$3.4 million.

Improvement Fee. The improvement fee cost basis is the portion of the Capital Improvements Plan presented in Chapter 8 that constitutes net eligible costs for SDCs. Ineligible capital costs will include costs for projects not expected to be funded by the City and/or improvements that do not expand capacity. The net eligible cost basis for the improvement fee is estimated to be approximately \$24.7 million.

Administrative Cost Component. The annual administrative costs to comply with SDC rules include an amortized cost for the SDC study and costs to account for SDC expenditures.

S.9.4 Summary of Recommended SDCs

Table S-7 summarizes the wastewater SDC components and overall fee per MCE that FCS GROUP calculated from current planning information. The City should plan to periodically adjust these proposed SDCs to account for inflation.

Table S-7	
Recommended Wastewater System Development Charge	
A. Updated SDC Components per MCE	
Reimbursement Fee	\$586
Improvement Fee	\$4,281
Administrative Cost Recovery	\$279
B. Total Updated SDC per MCE	\$5,146
Net Decrease from Existing SDC	(\$143)

CHAPTER 1

INTRODUCTION

1.1 **PURPOSE AND SCOPE**

The purpose of this Wastewater Master Plan (WWMP) Update is to develop and present a plan for meeting the City's projected, 20-year wastewater (WW) service needs inside the planning area. The preparation of this study was approved and authorized by the Madras City Council on December 9, 2014. The scope generally consists of the following main elements.

1. Collect and review available background data associated with the existing WW facilities as well as with the management, operation and maintenance of those facilities.
2. Develop a WW facilities inventory and complete condition assessments of the facilities in coordination with City staff.
3. Establish the planning criteria and basis of design for WW facilities to identify projected needs, develop and evaluate alternative improvements, and select a recommended alternative.
4. Develop and execute a collection-system hydraulic model to identify capacity deficiencies under current and projected flow conditions, and analyze alternative sewer improvements.
5. Develop and evaluate alternative improvements and select the recommended improvements.
6. Prepare a Capital Improvements Plan (CIP) that lists the projects proposed to implement the recommended improvements.
7. Describe the elements of a Capacity, Management, Operations and Maintenance (CMOM) Program, identify CMOM goals, and address the steps the City should take to meet these goals.
8. Identify other planning issues the City may need to address in the future to sustain sewer services.
9. Use the CIP and data supplied by the City to calculate updated system development charges.
10. Prepare materials and participate with City staff in a public information efforts to solicit input and obtain buy in from the community.
11. Prepare a complete WWMP Report that presents the results of the study.

1.2 **BACKGROUND**

1.2.1 *Previous Master Planning Effort*

ACE Consultants completed a Wastewater System Master Plan for Madras in November 1996 that provided a comprehensive analysis of the existing facilities and the impacts from projected growth. The 1996 study recommended extensive modifications and expansions to the WW facilities that were largely implemented over the following five years.

Due to the comprehensive nature of the 1996 WWMP, this update builds on the information and conclusions presented in that study to the extent feasible. Most of the WW infrastructure constructed based on the recommendations of the 1996 Plan still has useful service life remaining.

As a result, this study has been prepared with the understanding that significant portions of the WW infrastructure, particularly major structures and pipelines, can continue to serve the City for the next 20 years. Alternatives that would require the abandonment of these relatively-new components were generally eliminated from consideration.

1.2.2 *General Description of Planning Area*

The planning area for this WWMP Update corresponds to the area currently lying inside the City's Urban Growth Boundary (UGB). Figure 1-1 shows both the current UGB and City Limits for Madras, as well as elevation contours at 10-foot intervals. This planning area encompasses approximately 6.0 square miles (3,850 acres). By comparison, the study area for the 1996 WWMP was 7.4 square miles and the UGB at that time encompassed 4.5 square miles.

This study addresses WW service requirements for existing and projected developments within the UGB shown in Figure 1-1. The City also currently provides WW service outside the UGB in two locations, the State's Deer Ridge Correctional Institution (DRCI) and the Madras Municipal Airport. Therefore, this study takes into account the ongoing service needs for these two developments as part of the master planning effort.

Service requirements for the DRCI, which is located about one mile east of the City limits, were established in a 2006 agreement between the City and Oregon Department of Corrections (ODOC). The airport is west of the Industrial Park at the north end of the City. The 2010 Airport Master Plan Update prepared by Century West did not identify expansion plans that would significantly affect WW contributions from the airport.

The City's WW treatment, effluent storage and effluent recycling facilities extend beyond the UGB in certain locations. Therefore, this study addresses siting considerations for expansions to WW facilities both inside and in adjacent areas immediately outside the UGB.

Current planning information on population and development within the Madras UGB are presented in Chapter 3 of this study. Descriptions of the Madras planning area, including information on local topography, climate, geology and soils are presented in the 1996 WWMP.

1.2.3 *History and Development of Existing Wastewater Facilities*

Municipal System. The following paragraphs provide a brief background overview of the City's WW facilities. Chapter 2 provides a more detailed description and inventory of these existing facilities.

The City of Madras originally constructed a collection system and wastewater treatment plant (WWTP) in 1973-75. Prior to the construction of this centralized system, residents and businesses in Madras had onsite disposal systems generally consisting of septic tanks with disposal wells and/or drainfields.

The main components of the original municipal facilities were a gravity sewer system serving what is now the central part of town, a pump station on "B" Street at 1st Street, a force main (pressure sewer), and a WWTP. The WWTP provided biological treatment in a series of stabilization ponds and the effluent was pumped into a spray irrigation system for reuse on nearby land. This original WWTP was constructed outside the City on land adjacent to the municipal airport.

North Area (Industrial Park). A small, North Unit sewer system and pump station were built adjacent to the airport in the 1940s by the U.S. War Department and were operated by the North Unit Irrigation District. At the time, the airport was an army airfield. The original pump station discharged WW through a force main and into a drainage ditch on airport land.

After the municipal WWTP was constructed in the 1970s, the sewers, pump station and force main in this north area were replaced and the WW was pumped from a pump station at Demers Lane to the City's WWTP. The north area is now part of the City's Industrial Park and the sewer system has been expanded to serve nearby developments.

Recent Developments. Following the adoption of the 1996 WWMP, the City implemented a series of upgrades that reconfigured the WW facilities. The result is flows from the main part of the City are now split and pumped to two separate WWTPs: the original plant, now called the North WWTP (NWWTP), and a newer South WWTP (SWWTP) in the southeast part of the City. The SWWTP was expanded in 2008 to receive flows from the DRCl.

Sludge drying beds were constructed at the NWWTP to receive biosolids from the SWWTP. The dried biosolids are land applied on farmland.

Onsite Sewage Disposal. A small proportion of the dwellings within the UGB continue to rely on septic tanks with disposal wells and/or drainfields to handle the WW they generate. Most of these properties are outside the City limits on County land, but some are also inside the City.

1.3 REGULATORY FRAMEWORK

1.3.1 *Operating Permit*

The City of Madras WW facilities operate under the requirements set forth in the Water Pollution Control Facility (WPCF) permit issued in 2006 by the Oregon Department of Environmental Quality (DEQ). This WPCF permit, attached as Appendix A, was issued under the authority of Oregon Administrative Rule (OAR) 340-045. These rules were published to conform to the federal National Pollution Discharge Elimination System (NPDES) established under the Clean Water Act.

The WPCF permit establishes the maximum quantities of treated effluent the City's WWTPs are allowed to produce and the minimum levels of treatment the WWTPs must provide. The permit also contains monitoring and reporting requirements for the influent to each plant, effluent discharged from each plant and biosolids produced at each plant.

The expiration date for the current permit was October 31, 2016 and the City applied for a permit renewal in August 2016. The terms and conditions of the previous permit remain in force until the issuance of a renewal permit by DEQ.

The City's WPCF permit does not allow the discharge of treated WW into Willow Creek or any other surface water body. Therefore, the City currently must reuse all treated effluent as recycled water and does so through spray irrigation onto farmland and the Desert Peaks Golf Course.

If in the future the City wants to implement a WWTP discharge into Willow Creek or into the groundwater, a separate NPDES permit would need to be obtained from DEQ. As with the WPCF permit, an NPDES permit would identify treatment, monitoring and reporting requirements for the WWTP that would produce the surface-water or groundwater discharge.

The permit would establish treatment requirements by identifying the discharge limits the WWTP would need to meet to comply with water quality standards (WQS) for the receiving waters. WQS are published by DEQ after they are approved by the United States Environmental Protection Agency (USEPA). Subsection 1.3.2 provides more information on the WQS that DEQ would use to regulate a discharge to Willow Creek.

1.3.2 Current Regulations

Water Recycling. The use of treated effluent from WWTPs as recycled water is regulated in Oregon by DEQ according to OAR 340-055 (latest revision 2008).

Where WWTP effluent is recycled, the WPCF permit identifies the class(es) of recycled water the WWTP is designed to produce. Table 1-1 (following page) summarizes the recycled water classifications defined in OAR 340-055 and lists the minimum treatment and monitoring requirements established for each class.

Table 1-1 Classes of Recycled Water ⁽¹⁾					
Description	Class A	Class B	Class C	Class D	Non-Disinfected
Former Water Reuse Level ⁽²⁾	IV	III	II	Enhanced I	I
Oxidized	Yes	Yes	Yes	Yes	Yes
Disinfection	Yes	Yes	Yes	Yes	
Filtered	Yes				
Turbidity Limit (NTU):					
24-Hour Mean	2				
5% of Time in 24-Hr. Period	5				
Max. at Any Time	10				
Monitoring Frequency	Hourly				
Total Coliform: ⁽³⁾					
7-day Median	2.2	2.2	23		
Max. in Any Sample	23	23			
Max. in 2 Consecutive Samples			240		
Monitoring Frequency	Daily	3/Week	1/Week		
E. Coli: ⁽³⁾					
30-Day Log Mean				126	
Max. in Any Sample				406	
Monitoring Frequency				1/Week	
Beneficial Purposes	More Allowable Uses			Fewer Allowable Uses	
Conditions on Use	Less Restrictive			More Restrictive	

(1) Taken from Table 3 in DEQ Internal Management Directive – Implementing Oregon’s Recycled Water Rules (June 19, 2009).

(2) Similar, but inexact correlation to levels in previous rules.

(3) Measured as Organisms/100 mL.

Table 1-2 (following page) identifies the allowable beneficial uses for each class. In general, Class A water must receive the highest level of treatment, but has the most allowable uses and the least restrictive conditions of use. The City’s WPCF permit was issued when an earlier version of OAR 340-055 was in force and lists both Level II and Level IV effluent classes as options for recycling. These levels approximately correlate to current recycled water Classes C and A, respectively.

The City has prepared an Effluent Reuse Plan for each WWTP and submitted these plans to DEQ for approval. Copies of these reuse plans are included as Appendices B-1 and B-2. The WPCF permit requires the City to submit an annual report to the DEQ by February 1 documenting compliance during the prior year with the reuse plans, the current OAR, and the permit.

The Effluent Reuse Plans must be updated and submitted to DEQ at the time of permit renewal. Updated plans must also be submitted if the City needs to change their recycling practices during the permit cycle.

Table 1-2 Recycled Water Beneficial Purposes					
Description	Class A	Class B	Class C	Class D	Non-Disinfected
Irrigation					
Fodder, fiber, seed crops not intended for human ingestion, commercial timber	Yes	Yes	Yes	Yes	Yes
Firewood, ornamental nursery stock, Christmas trees	Yes	Yes	Yes	Yes	No
Sod	Yes	Yes	Yes	Yes	No
Pasture for animals	Yes	Yes	Yes	Yes	No
Processed food crops	Yes	Yes	Yes	No	No
Orchards or vineyards if an irrigation method is used to apply recycled water directly to the soil	Yes	Yes	Yes	No	No
Golf courses, cemeteries, highway medians, industrial or business campuses	Yes	Yes	Yes	No	No
Any agricultural or horticultural use	Yes	No	No	No	No
Parks, playgrounds, school yards, residential landscapes, other landscapes accessible to the public	Yes	No	No	No	No
Industrial, Commercial, or Construction					
Industrial cooling	Yes	Yes	Yes	No	No
Rock crushing, aggregate washing, mixing concrete	Yes	Yes	Yes	No	No
Dust control	Yes	Yes	Yes	No	No
Nonstructural fire fighting using aircraft	Yes	Yes	Yes	No	No
Street sweeping or sanitary sewer flushing	Yes	Yes	Yes	No	No
Stand alone fire suppression systems in commercial and residential buildings	Yes	Yes	No	No	No
Non-residential toilet or urinal flushing, floor drain trap priming	Yes	Yes	No	No	No
Commercial car washing	Yes	No	No	No	No
Fountains when the water is not intended for human consumption	Yes	No	No	No	No

Surface Water Discharge. The effluent limitations that determine WWTP treatment requirements for a surface-water discharge are based on the effluent quality necessary to meet the published WQS outside a discharge mixing zone. The current WQS for Oregon water bodies, including the Deschutes Basin, are published in OAR 340-041.

Since Willow Creek flows into the Deschutes River at Lake Simtustus, a discharge to the creek would need to meet the WQS that apply to the mid-Deschutes Basin. Table 1-3 describes the current WQS for the mid-Deschutes.

Table 1-3 Key Numerical Water Quality Standards for Mid Deschutes Basin and Willow Creek ⁽¹⁾	
Water Quality Criterion	Limitation
1. Five-Day Biochemical Oxygen Demand (BOD ₅)	
Winter Season (November 1 – March 31)	
Maximum Monthly Average ⁽²⁾	30 mg/L
Maximum 7-Day Average ⁽²⁾	45 mg/L
Summer Season (April 1 – October 31):	
Maximum Monthly Average	10 mg/L
Maximum 7-Day Average	15 mg/L
2. Total Suspended Solids – Same as BOD ₅ for winter & summer	
3. Bacteria (E. Coli Organisms)	
Maximum Monthly Log Mean	126 per 100 ml
Maximum (single sample)	406 per 100 ml
4. Dissolved Oxygen (DO) – Minimum ⁽³⁾	8.0 mg/L
5. pH Range (Hydrogen Ion Concentration)	6.5 – 8.5
6. Temperature – Maximum 7-day average ⁽⁴⁾	18.0 degrees
7. Total Dissolved Solids – Maximum	500 mg/L
8. Chlorophyll-a – Average ⁽⁵⁾	0.015 mg/L

NOTES:

- (1) Numerical WQS as of January 2017 for Deschutes Basin reach between Bend Diversion Dam and Pelton Dam, including Willow Creek and other tributaries.
- (2) Representative numerical limits for required secondary treatment.
- (3) Numerical DO criterion for water bodies potentially supporting cold-water aquatic life. With adequate background data, allowable limits may be modified to 6.0 mg/L as an absolute minimum, 6.5 mg/L as a 7-day mean, and 8.0 mg/L as a minimum 30-day mean.
- (4) Numerical temperature criterion for stream potentially having salmon and trout rearing and migration uses.
- (5) Numerical chlorophyll-a limit intended to prevent nuisance phytoplankton growth.

Key non-numerical WQS that would potentially affect a proposed discharge into Willow Creek are briefly summarized below.

1. **Antidegradation – OAR 340-041-0004.** The Antidegradation Policy is generally intended to prevent unnecessary water quality degradation from new or increased sources of pollution. The policy also aims to protect, maintain, and enhance existing, surface water quality and thereby ensure the full protection of all existing beneficial uses. In order to obtain a new NPDES, the City would probably be required to submit a pollution fate analysis documenting that a new discharge would not degrade existing water quality.
2. **Effluent Dilution and Mixing – OAR 340-041-0007, Article (15)(a)(A)(i).** Effluent organic materials, as measured by BOD₅ concentration, must not exceed the dilution factor. This factor is defined as the ratio of receiving-stream flow to effluent flow. This criteria would prohibit the discharge of any measurable BOD₅ during those periods when there is no stream flow.

Since Willow Creek is a seasonal stream, it frequently has no background flow during summer, fall, and potentially even winter months. Therefore, in order to obtain a new NPDES, the City would either need an exemption from this criteria or limit discharges to periods when there is adequate stream flow. Even then, the potential for low stream flows may result in DEQ issuing a low BOD₅ discharge limit that would require an expensive treatment process.

Biosolids. DEQ regulates biosolids handling according to the requirements of OAR 340-050 (latest revision 1995) and the U.S. EPA's Biosolids Rule (40 CFR Part 503). The City has prepared, and amended, a Biosolids Management Plan according to the OAR that describes the City's practices for handling, treating and land applying the biosolids. Appendix B-3 includes a copy of this plan.

The WPCF permit requires the City to submit a report to the DEQ by February 19 that documents the biosolids handling activities during the prior year. The minimum required information in the report is outlined in OAR 340-050-0035.

The Biosolids Management Plan must also be updated and submitted to DEQ at the time of permit renewal or whenever the City changes the way the biosolids are handled.

Onsite Disposal. The onsite sewage disposal systems within the UGB are regulated by Jefferson County under agreement with DEQ according to OAR 340-071 (latest revision 2014). The rules require a property owner to obtain a permit for the construction of a new onsite system or the repair/replacement of an existing system.

If a property is within 500 feet of an existing sewer, then any new development on that property is required under the OAR to connect to the sewer and a permit for an onsite system will not be issued. Similarly, if an existing onsite system fails and a sewer is within 500 feet, then the system user must connect to the sewer.

1.3.3 *Federal Aviation Administration Standards for Airports*

Federal Aviation Administration (FAA) guidelines place constraints on potential expansions to the existing NWWTP due to the proximity of the airport to the plant site. The FAA has published guidance identifying WWTPs as potential wildlife attractants that should not be located near airports unless acceptable mitigation measures are implemented. As a result, a mitigation plan for deterring wildlife attraction must be developed as part of any NWWTP expansion and the FAA must accept both the expansion and mitigation plans. The FAA relies on the United States Department of Agriculture – Wildlife Services to review and approve mitigation plans.

1.3.4 *Potential for Regulatory Changes*

The OAR and federal rules that govern the operation of the City's WW facilities are periodically revised and reissued based on new legislation or other factors. As part of this study, HHPR contacted the DEQ Eastern Region office in Bend to ask about the potential for upcoming revisions to the governing regulations.

DEQ reported that the agency had no plans at the time this report was written to revise and reissue the OAR covering WPCF permits, effluent recycling, biosolids handling or onsite sewage disposal. There are also no indications the USEPA will be revising federal rules pertaining to effluent recycling or biosolids handling.

Municipalities such as Madras that recycle their WWTP effluent tend to experience fewer regulatory changes than WWTPs with surface-water discharges. Facilities with NPDES permits for surface-water discharges tend to experience more changes permit requirements.

Changes to NPDES permit requirements often stem from the promulgation of stricter WQS for a specific water body that then result in stricter WWTP effluent limitations. The mid-Deschutes Basin and Willow Creek are listed as being water quality impaired for certain criteria and the federal Clean Water Act currently requires regulating authorities to issue Total Maximum Daily Loads (TMDLs) that cover these criteria for such impaired water bodies.

The TMDLs identify the maximum amounts of certain pollutants that can be discharged into a basin without degrading water quality. No TMDLs have been proposed yet for the Deschutes Basin and DEQ has indicated that the issuance of the TMDLs is not a priority. The lack of TMDLs for the criteria not conforming to current WQS creates uncertainty regarding future effluent limits for a discharge into Willow Creek.

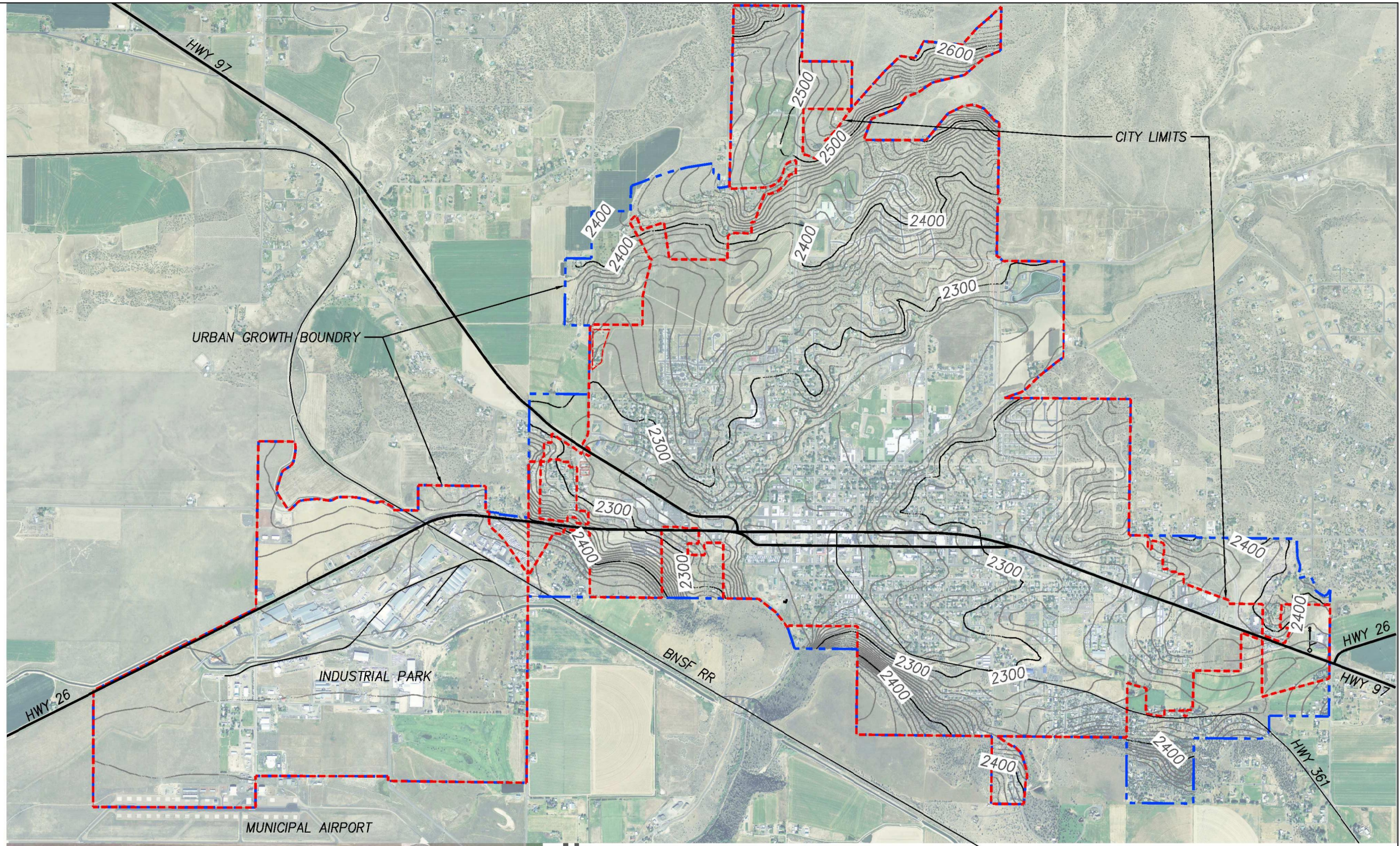
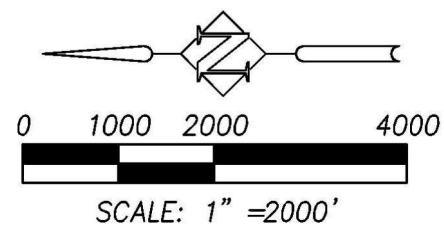
In general, it can be difficult from a regulatory standpoint to obtain an NPDES permit for a new surface-water discharge, particularly into a water body listed as water-quality impaired. This regulatory climate provides an incentive for communities such as Madras to continue the practice of effluent recycling.

1.4 **ACKNOWLEDGEMENTS**

HHPR and FCS Group appreciate the valuable input, data-collection efforts, and other support provided by City and County staff. We particularly wish to acknowledge the contributions of the following individuals:

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CHAPTER 2

EXISTING FACILITIES INVENTORY AND CONDITIONS ASSESSMENTS

2.1 FACILITIES DESCRIPTIONS

2.1.1. *General Description*

The major elements of the existing wastewater (WW) facilities serving Madras are listed below:

- collection systems that receive and convey sewage to the City's two treatment plants;
- the North and South Wastewater Treatment Plants (NWWTP and SWWTP);
- storage ponds and transmission systems that allow effluent recycling as irrigation water;
- a supervisory control and data acquisition (SCADA) system; and
- onsite WW treatment and disposal systems on properties not served by or connected to a sewer.

Each of these elements are described in the following paragraphs. Figure 2-1 shows their locations and Figure 2-2 shows a simplified schematic of the general flow patterns through the WW facilities.

2.1.2. *Collection Systems*

General. The City's WW facilities include two separate collection systems. The Main Collection System serves most of the City and the Industrial Park Collection System serves the Madras airport and a relatively small area to the east and southeast of the airport. Major components of the collection systems include gravity collector and interceptor sewers, manholes (MHs), pump stations, and force mains.

Main Collection System. This system includes 188,620 feet of gravity sewers with associated MHs, three pump stations, and 29,410 feet of pump station (PS) force mains. Gravity sewers in the Main Collection System are divided into three drainage basins: a central sewer basin that serves most of the City, a southeast sewer basin, and a small sub-basin at the south end of the City.

Most of the large central basin was the original sewer system for the City. The sewers in this basin convey flows to the "B" Street Pump Stations located at the northwest corner of NW 1st and "B" Streets. An onsite diversion structure splits incoming flows between the "B" St. North PS and "B" St. South PS. The North PS pumps flows through a 10-inch force main that extends to the NWWTP. The South PS pumps flows to the SWWTP via a 14-inch force main that discharges into a gravity sewer in McTaggart Road that is part of the SE sewer basin.

Flows in the SE basin are conveyed by gravity to the SWWTP. This basin consists of interceptor sewers in McTaggart Rd., "J" St., City View Drive, and Ashwood Rd., as well as tributary collector sewers. The Ashwood Rd. sewer receives the WW pumped from the Deer Ridge Correctional Institution (DRCI) and also includes an inverted siphon. Currently, there is limited development within the SE basin and the DRCI is not fully occupied. Therefore, most flow to the SWWTP comes from the "B" St. South PS.

The third basin in the Main Collection System consists of a short run of gravity sewer near the south City limit, on the east side of U.S. Highway 97. This sewer segment flows east to west and drains into the South 97 PS. A 4-inch force main carries the flows north from the PS to the central sewer basin described above.

Industrial Park Collection System. This system consists of 24,330 linear feet of gravity sewers with MHs, two pump stations, and 14,370 linear feet of force mains. The gravity sewers are divided into two drainage basins: one sewer system serves the Industrial Park and airport and a second, smaller sub-basin serves a residential area along Birch Lane.

The Golf Course PS is located at the west end of this smaller basin and pumps these residential flows north, through a 4-inch force main, to a manhole south of Adler St.

The Industrial-Park sewers are tributary to the Demers PS located south of the intersection of Cherry Lane and Demers Drive. The Demers PS directs flows through dual 4-inch force mains that connect to the 10-inch force main extending from the “B” St. North PS to the NWWTP.

2.1.3. Wastewater Treatment Plants

General. Each wastewater treatment plant (WWTP) includes liquid-stream processes to treat the wastewater and solids-stream processes to handle the solids produced as byproducts of the liquid-stream processes. The NWWTP was originally constructed in 1973 and remained the City’s sole treatment plant until 2001 when the SWWTP was constructed. Upgrades to the NWWTP were implemented in 1993 and 1998 and an expansion of the SWWTP was constructed in 2008.

NWWTP. This facility is located adjacent to the Municipal Airport and has an average design capacity of 0.5 million gallons per day (MGD). Figure 2-3 provides a site plan of the NWWTP. The plant consists of the following main processes:

- a five-cell lagoon system,
- mechanical clarification equipment with chemical addition,
- disinfection equipment that feeds sodium hypochlorite and a chlorine contact basin,
- an effluent storage pond,
- in-plant liquid-stream and sludge pumps,
- a sludge pond to store solids generated by the clarification equipment, and
- sludge drying beds.

The lagoon system includes two primary cells, two secondary cells, and a polishing cell that support the growth of microorganism for biological treatment. The primary cells are aerated, partially-mixed lagoons, whereas the secondary cells are unaerated and act as facultative lagoons. The polishing cell is also a facultative lagoon. The mechanical clarifier uses dissolved air flotation (DAF) to remove algae and other solids in the lagoon effluent. This reduces chlorine demand and prevents fouling of irrigation components.

PLC-based controls are provided for the process equipment. The clarifier is part of a proprietary process that includes a sand filter unit. But since the City does not need to produce Class A effluent, the filter has not been used recently and is no longer functional. The sludge drying beds are used to dry biosolids trucked from the SWWTP as a dilute liquid. Sludge from the sludge pond can also be pumped to the beds for drying. The dried materials are spread on farmland as Class B biosolids.

SWWTP. This facility is located on the south side of “J” St., between Willow Creek and Grizzly Road. A site plan of the SWWTP is shown in Figure 2-4. The plant consists of the following main processes:

- headworks for preliminary treatment,
- a sequencing batch reactor (SBR) process,
- an onsite storage lagoon,
- tertiary filters and chemical feed equipment,
- disinfection equipment that feeds sodium hypochlorite,
- influent, effluent and other in-plant pumps,
- aerobic digesters for biosolids processing, and
- two effluent storage ponds.

The plant headworks consist of influent screening and grit removal processes. The screenings and grit removed by these processes are disposed of at a landfill. The SBR process is an activated-sludge system that provides biological treatment. The filters remove suspended solids to reduce chlorine demand and prevent fouling of irrigation components. Because the City does not need to produce Class A effluent, chemicals are not added upstream of the filters to increase solids removal.

The SWWTP was designed to ultimately have three 0.5-MGD SBR modules and two of those modules have been built. But when the second module was added, the plant design was modified to treat more-highly concentrated WW from the DRCI. The design WW strength for the DRCI is three times the design strength for the City. As a result, the expansion increased the capacity to treat a higher organic loading, but did not significantly increase the flow capacity. Now the two SBR modules have a 0.54-MGD average design flow capacity rather than a 1.0-MGD capacity.

The agreement between the City and the State Department of Corrections stipulates that an average flow capacity of 0.23 MGD at the SWWTP is dedicated to serving the DRCI. Although the DRCI is not fully occupied, this study assumes it will be contributing the maximum allowed flow within five years to avoid reliance on capacity that is dedicated to the DRCI. This currently leaves the City with an available capacity of 0.31 MGD at the SWWTP.

2.1.4. *Effluent Recycling*

General. Both WWTPs are zero-discharge facilities and all treated effluent is recycled as irrigation water. The WW conveyed to each WWTP receives biological treatment throughout the year to produce oxidized effluent suitable for further treatment, storage, and recycling. Biological treatment helps reduce the risk of nuisance odors and vector attraction.

The City generally operates the two plants to split flows between them, but adjusts the proportions to make sure all stored effluent from each WWTP is recycled by the end of the growing season. The growing season extends from about April 15 to October 15. During the colder months, treated WW is stored in ponds. The treated WW is stored either as secondary effluent that will receive further treatment before recycling or as disinfected effluent ready to be recycled.

NWWTP. The influent WW flows by gravity through the NWWTP lagoon system for biological treatment and is then pumped to a single storage pond. When recycling occurs, the water in the storage pond is pumped through the DAF clarification process and then disinfected. The disinfected effluent is then pumped from the chlorine contact tank into the irrigation system for recycling. NWWTP effluent is currently used on nearby farm land and the Municipal Golf Course.

SWWTP. Wastewater entering the SWWTP is treated in the SBR process and then discharged to a post equalization basin and an onsite storage pond. The SBR effluent flows by gravity through the filters from the onsite pond and the post equalization basin. The filtered water is disinfected to produce recycled water that is then pumped either the irrigation systems for farm fields near the SWWTP or a remote, upper storage pond about 1.5 miles east of the plant. Recycled water stored in the upper pond flows by gravity down to the irrigation systems.

Water Recycling. Both WWTPs produce Class B effluent as defined in OAR 340-055 and the City follows the access, exposure, and notification requirements listed for Class B water. The City owns the farm lands that receive recycled water and leases it to growers with the understanding the crops will be irrigated with recycled water. The objective is to use all stored effluent by the end of each growing season so the storage ponds are empty before cold weather forces a halt to irrigation for the winter. Figure 2-1 shows the land currently irrigated with recycled water.

Irrigation practices follow the approved Effluent Reuse (Recycled Water Use) Plans for each WWTP. A plan was prepared for the NWWTP in 1993 and a similar plan was prepared for the SWWTP in 2004 and updated in 2007 when the SWWTP was expanded. These plans address irrigation practices, application rates, land requirements and other recycling considerations.

2.1.5. Onsite Wastewater Disposal Systems

There are estimated to be 106 residential properties within the City not currently served by sewers and assumed to have onsite septic systems for wastewater disposal. There are approximately 210 additional lots within the UGB, but outside the City limits, also not served by sewers and assumed to have onsite disposal systems. Figure 2-1 outlines the general areas where most of the onsite systems are located. These properties are generally outside the current City limits.

2.2 INVENTORIES OF WW FACILITIES

2.2.1. Collection Systems

Sewers and Force Mains. Table 2-1 summarizes the gravity sewer and force main lengths in each collection system broken down by size. The numbers of sewer MHs in each system are also listed. Appendix C provides a tabulation of the pipe and MH data for the existing sewer lines in the two collection systems. Figures 4-1A through 4-1D provide aerial maps of the service area with the locations of the sewer lines and MHs shown.

Table 2-1 Existing Collection-System Pipe Lengths			
1. Main Collection System:			
Gravity Sewers		Force Mains	
Pipe Size (in.)	Length (ft.)	Pipe Size (in.)	Length (ft.)
6	2,600	4	1,140
8	167,700	8	6,070
10	4,770	10	15,980
12	3,380	14	6,220
15	2,300	Total Length:	29,410
18	2,870		
Total Length:	183,620		
No. of Gravity Sewer MHs: 807			
2. Industrial Park Collection System:			
Gravity Sewers		Force Mains	
Pipe Size (in.)	Length (ft.)	Pipe Size (in.)	Length (ft.)
8	15,920	4	14,370
10	6,970		
12	1,440		
Total Length:	24,330		
No. of Gravity Sewer MHs: 71			

Gravity sewers constructed in the 1970s are asbestos cement (AC) pipe with rubber-gasketed joints. Since then, sewers and force mains have been constructed of polyvinyl chloride (PVC) pipe, also with rubber-gasketed joints.

Pump Stations. Table 2-2 summarizes the configurations and capacities of the collection system pump stations. Inventories of the main PS components are provided in Appendix D.

Table 2-2 Existing Collection-System Pump Stations	
1. Main Collection System:	
“B” Street South Pump Station	
Configuration:	Submersible
No. of Pumps:	Two
Design Capacity:	920 gpm @ 53 feet TDH (each pump)
Motor Size & Speed:	23 horsepower & 1,750 rpm
South 97 Pump Station	
Configuration:	Submersible
No. of Pumps:	Two
Design Capacity:	40 gpm @ 49.5 feet TDH (each pump)
Motor Size & Speed:	1.9 horsepower & 3,395 rpm
2. Industrial Park Collection System:	
“B” Street North Pump Station	
Configuration:	Wetwell/Drywell
No. of Pumps:	Four Total (two pairs in series)
Design Capacities:	770 GPM @ 165 feet TDH (each pump)
Motor Size & Speed:	60 horsepower & 1,785 rpm
Demers Pump Station	
Configuration:	Submersible
No. of Pumps:	Two
Design Capacities:	290 GPM @ 76.0 feet TDH (each pump)
Motor Size & Speed:	10 horsepower & 1,750 rpm
Golf Course Pump Station	
Configuration:	Submersible
No. of Pumps:	Two
Design Capacity:	Not Available
Motor Size & Speed:	1.7 horsepower & 1,750 rpm

The “B” St. North PS has a wetwell/drywell configuration and, as a result, is the only PS with an aboveground building. A standby generator set (genset) is installed in a separate building at the “B” St. site to supply backup power to both the North PS and South PS.

An outdoor chemical storage tank and an RV dump station are also located at the “B” St. PS site. The tank is used to store calcium nitrate that is fed into the South PS wet well to control odors at the force main discharge MH in McTaggart Road.

The rest of the City's pump stations have submersible pump assemblies and the structural components are almost entirely below ground. The enclosures for electrical and control panels are the only components mounted above-grade at these submersible stations. City personnel must bring a portable genset to these other pump stations to provide backup power during outages.

All the pump stations, except the existing Golf Course PS, contain radio telemetry equipment that communicates alarms and PS status to the SCADA system. The SCADA system records pump run times and alarm histories.

2.2.2. Wastewater Treatment Plants

Tables 2-3 and 2-4 identify the general design parameters and summarize the major process components for the NWWTP and SWWTP, respectively. Appendix E includes inventories of the equipment and other major components at each plant.

Table 2-3	
Existing NWWTP Design Parameters and Components	
1. Plant Influent Design Parameters	
Average Daily Flow:	0.50 MGD
Peak Hourly Flow:	1.35 MGD
Design Average BOD ₅ Concentration:	200 mg/L
Design Average TSS Concentration:	215 mg/L
2. Biological (Oxidation) Process	
Process:	Partially-Aerated/Facultative Lagoon System
Total System Surface Area & Volume:	28 acres & 50 MG
Average Organic Surface Loading Rate:	30 lbs. BOD ₅ /acre/day
Primary Lagoon Aerators (each cell):	10 units @ 5.0 hp each (50 hp/cell)
3. Clarification/Filtration Process	
Process Capacity:	1.0 MGD Avg. Recycling Flow
Clarification Process:	Krofta SupraCell (SPC) Dissolved Air Flotation (DAF) Clarifier
Filtration Process (Not Used):	Krofta Sandfloat (SASF 20) Filter
4. Disinfection Process	
Disinfectant:	Sodium Hypochlorite Solution
Maximum Feed Rate:	170 gpd
5. Effluent Storage Pond	
Total Storage Volume:	79 MG
Design Storage Capacity:	Five Months @ 0.5 MGD Avg. Flow
6. Intermediate Lift & Effluent Irrigation Pumps	
No. of Pumps:	Two (one intermediate & one effluent)
Design Pump Capacity:	700 gpm each
7. Sludge Drying Beds	
No. of Beds:	24
Bed Dimensions:	24 ft. x 100 ft. x 12 in. Deep (each)
Total Bed Area	57,600 square feet

Table 2-4
Existing SWWTP Design Parameters and Components

1. Plant Influent Design Parameters	
Average Daily Flow:	0.54 MGD
Peak Hourly Flow:	1.33 MGD
Design Average BOD ₅ Concentration:	465 mg/L
Design Average TSS Concentration:	465 mg/L
2. Headworks	
Influent Screening:	One Mechanically-Cleaned Fine Screen
Design Screen Capacity:	4.0 MGD
Grit Removal Process	One Low-Energy Vortex Circular Chamber
Design Grit Removal Capacity:	2.5 MGD
3. Biological (Oxidation) Process	
Process:	SBR, Activated-Sludge System
No. of SBR Basins:	Two
Design Effluent BOD ₅ Concentration:	10 mg/L
Design Organic Loading Rate:	0.5 lbs. BOD ₅ /lb. MLSS @ 4,500 mg/L MLSS
Design Oxygen Requirements:	1.5 lbs. O ₂ /lb. BOD ₅ and 4.6 lbs. O ₂ /lb. TKN
4. Filter Process	
Process:	Cloth-Media Disk Filter
Peak Process Capacity:	2.4 MGD
No. of Filter Disks:	Four
Design Effluent TSS Concentration:	5.0 mg/L
5. Disinfection Process	
Disinfectant:	Sodium Hypochlorite Solution
Maximum Feed Rate:	170 gpd
6. Biosolids Handling	
Stabilization Process:	Aerobic Digesters
No. of Digester Basins:	Three
Design Avg. Retention Time:	60 Days
Drying Process:	AC-Lined Sludge Drying Beds @ NWWTP
Biosolids Disposal:	Land Application of Class B Biosolids
7. Storage Ponds	
Lower Pond Volume:	25 MG (Onsite SBR Effluent Storage)
Upper Pond Volume:	88 MG (Remote Site)
Design Storage Capacity:	Five Months @ 0.54 MGD Avg. Flow
8. Effluent Pumps	
No. of Pumps:	Two
Design Pump Capacity:	1,050 gpm each

2.3 CONDITIONS ASSESSMENTS

2.3.1. *Criteria for Conditions Assessments*

General. The main factors to consider when assessing the condition of each WW facilities component are listed below:

- how well the component functions;
- the actual physical state of the materials (or age, if component cannot be checked visually);
- whether the item meets current industry standards or codes;
- whether the capacity of the component is sufficient to handle existing flow conditions; and
- the criticality of a component in the overall purpose of safely conveying, treating and disposing of the WW and byproducts.

These factors must be weighed both when deciding whether the City should plan to improve a component within the next 20 years and in prioritizing the recommended improvements. If an improvement is recommended, the condition assessment can also help determine, where practical, whether the City should plan to repair, modify or replace a component. However, further evaluation may be needed to determine if an item can still be repaired.

Since a visual inspection of all facilities is not practical for this level of study, particularly in the case of buried components, many of the condition assessments are based on age. If the age of an item is a key driver in recommending an improvement, further investigation may be warranted in the future to help focus and prioritize the City's efforts.

The issues of required future capacity and identification of future capacity deficiencies for WW facilities are addressed separately in Chapters 4 and 5 of this WWMP.

Gravity Sewers and Force Mains. The AC and PVC pipes used for the sewers and most force mains in Madras are both stable materials that can provide a reliable service life of 50 years or more in WW service. AC pipe is vulnerable to attack from hydrogen sulfide in the sewage and aggressive soils that can leach the calcium from cement-based pipe. But these conditions do not appear to pose serious problems in Madras and the AC pipe could potentially last 60 to 70 years.

PVC resists chemical attack, but can degrade over time and become more likely to fail. Plastic pipe is also susceptible to eventual damage from concentrated loads, if the pipe bedding and backfill materials were not installed properly.

The ductile iron pipe that was used for the force main between the "B" St. PS and the NWWTP would have been cement-mortar lined on the interior and asphalt coated on the exterior. These coatings slow the rate of corrosion and extend the life of the pipe. Ductile iron pipe also typically has a thicker wall for pressure service than PVC and is better suited for withstanding pressure surges from pumps cycle on and off over extended periods.

Pipe joints for both the gravity and pressure sewers are often the first part of the assemblies to degrade and begin to leak. When this occurs, gravity sewers experience increases in infiltration and inflow (I/I) from rainfall events and snow melt. Blockages also become more likely due to root intrusion and the accumulation of grease, rags and other solid materials.

Sewer service laterals and manholes are other system components that often experience degradation and can cause problems of high I/I. Proper connection of the laterals to the collector sewers and proper pipe connections to MHs are both critical in preventing I/I or clogging problems.

For planning purposes, we recommend using a conservative service life of 60 years for the AC sewers and ductile iron force main in Madras. We also recommend using a conservative 50-year service life for PVC sewers and force mains. These estimated service lives also extend to the associated service laterals and MHs.

Pump Stations and WWTPs. We have separated PS and WWTP components into the following main categories to assess their conditions:

- structures;
- mechanical, electrical and controls components;
- piping, process valves and appurtenances.

The recommended service lives for these categories are summarized in Table 2-5 and briefly discussed in the following paragraphs.

Table 2-5 Recommended Component Service Lives for Pump Stations and WWTPs	
Component Category	Estimated Service Life
CIP Concrete & CMU Structures	50 Years
Precast Concrete Vaults & Wet Wells	50 Years
Metal & Wood Structural Components	30 Years
Roof Decking or Shingles	30 Years
Mechanical, Electrical & Controls Components	25 Years
Exposed Piping & process Valves	30 Years
Buried Piping & Lagoon Liners	50 Years

Cast-in-place concrete and cement masonry unit (CMU) structures can be anticipated to last over 50 years if surfaces receive protective coatings. Since aggressive soil conditions do not appear to be present in Madras, buried concrete should have average to above average service lives. However, deterioration from sewage, biosolids and associated gases can shorten the life of process structures. As a conservative estimate, we recommend planning for a service life of 50 years for cast-in-place concrete and CMU structural components.

The structural framing for the roofs should generally last as long as the building structure. But the City needs to plan to replace shingles or decking on building roofs every 25 to 30 years. Similarly, a service life of 30 years is recommended for metal structural components such as stairs and grating.

Precast concrete vaults and wet wells are generally not as robust as CIP concrete, with thinner wall construction. Additionally, embedded aluminum hatches can experience considerable wear and deterioration over time. Therefore, we recommend using a service life of 40 years for precast concrete vaults and wet well.

Equipment, electrical components, and controls components should last 25 to 30 years, and can last longer. For planning purposes, it is recommended that a 25-year service life be used. Exposed piping, valves and appurtenances should typically last 30 years. Therefore, a 30-year service life is appropriate for these components.

We recommend a 50-year service life for buried yard piping since it should generally last as long as the sewers and force mains. The plastic liners used for lagoons and effluent ponds should also have a similar life as buried plastic piping.

2.3.2. *Conditions of Existing Gravity Sewers and Force Mains*

General Assessment. Indications are the sewers, force mains and appurtenances in the two collection systems are generally in adequate to good condition. The City reports that no sewer failures have occurred in the collection systems and backups are rare. Similarly, flow data indicate the infiltration and inflow (I/I) rates entering the sewers are not excessive, meaning it would not be cost effective to rehabilitate or replace sewers to reduce I/I rates. Also, as reported in Chapter Four, our models of the collection systems under current flows show the sewers and force mains have enough capacity to serve the existing population and a fully-occupied DRCI.

These findings are consistent with what we would anticipate based on local conditions. The oldest sewers and force mains have been in service just over 40 years, groundwater depths in the area are typically below the sewers, and stormwater drainage in the City is generally adequate. Based on our assessment, the City does not need to plan for major replacements of existing sewers and force mains strictly based on condition.

Age Considerations. Since the original AC sewers and the force main to the NWWTP were installed in 1975, these pipes are projected to be at or near the end of their projected 60-year service life at the end of the planning period. Some of the oldest existing PVC pipes may also approach the end of their expected 50-year service life within 20 years.

Based on the age criteria presented in Subsection 2.3.1, it is recommended the City plan to begin a program of inspecting the existing AC pipes and associated MHs in 10 to 15 years, or earlier if clogging or surcharging problems begin to occur. Inspection efforts should primarily be focused on areas where problems are reported. The City should also adopt a schedule for regularly cleaning the sewer pipes in each collection system. Given the lack of reported clogging problems, a cycle of 3 to 5 years may be appropriate.

It is preferable to avoid making repairs to AC pipe due to health concerns associated with exposure to asbestos fibers. Therefore, old AC pipe is typically either lined with cured-in-place pipe (CIPP), if the existing pipe is not too deteriorated, or it is replaced by a separate pipe. After replacement, the AC pipe is then abandoned in place. Since CIPP liner is less costly than full replacement, the City can potentially reduce future costs by proactively cleaning, inspecting and lining AC pipe before it becomes too deteriorated.

Hydrogen Sulfide and Odors. Currently, the AC sewers and concrete MHs do not show signs of accelerated deterioration from sewer gases. But there is the potential for concrete deterioration over time from hydrogen sulfide exposure at key locations. These include the discharge MHs at the ends of the PS force mains and the MHs adjacent to the inverted siphon in the Ashwood Rd. interceptor. The MH in McTaggart Road at the end of the force main from the B South PS is of particular concern since odors have been an issue there.

The City adds calcium nitrate at the B South PS to control hydrogen sulfide and prevent odors at the force-main discharge MH. As long as the chemical addition equipment is working properly and enough chemical is added, this system generally controls odors. Also, the discharge MH does not show signs of accelerated deterioration.

Odor problems have also been a periodic problem at the inverted siphon along Ashwood Rd. that are probably caused by low flows and long detention times in this sewer line. The City and the DRCI have added odor-control products that so far have been unsuccessful at solving the problem.

One maintenance measure the City should take to reduce odors is hire a Contractor to flush and clean the inverted siphon annually or at least every two years. This effort will reduce solids accumulation and potentially reduce hydrogen sulfide general.

Culver Highway Sewer. The sewer line along the Culver Highway has been identified as one area of concern regarding pipe condition and capacity. Observations and electronic monitoring performed by City staff in 2008 revealed periods of unusually high flow depths and brief periods of surcharging in the Culver Highway sewer southwest of 1st Street.

The velocity and depth data collected by the electronic monitoring device indicated these were transient conditions most likely caused by sewer backups due to the accumulation of solids. Some of the monitoring data also appeared to be erroneous. The data do not suggest that the high depths and surcharging were steady-state conditions caused by high peak flows. Records show the Culver Highway sewer was designed with minimum allowable slopes which could promote periodic sediment buildup.

Due to concerns about operating conditions in the Culver Highway sewer, we recommend the City plan to clean this line regularly and complete a video inspection of the pipe. Chapter Four includes additional discussion regarding the capacity of this sewer.

2.3.3. *Conditions of Existing Collection-System Pump Stations*

General Assessment. Table 2-6 provides overall summaries for the conditions of the five collection system pump stations. Condition assessments of the various PS components are provided in Appendix D.

Table 2-6 Summary of PS Condition Assessments			
Facility Name	Year Built	Overall Condition	Current Areas of Concern
"B" Street North	1974-75	Poor to Good	Problems with pump clogging and condition of original components from the 1970s.
"B" Street South	2001	Adequate to Good	Corrosion of metal components in valve vault and need for recoating.
"B" St. Genset	1999	Good	Need for recoating exterior wood structural components.
South 97	Not Available	Adequate to Good	Condition of valves and long detention time of sewage in wet well.
Demers	1989-90	Poor to adequate	Condition of valves and piping in valve vault.
Golf Course	1996	Poor to adequate	Age of existing facility, no SCADA unit, and condition of pumps and valves.

The pump stations have the firm capacity to handle peak flows and can reliably serve the tributary sewer systems. Therefore, the main issue is how old some components currently are or will become during the planning period. Some older components are in poor condition and need replacement.

Also, some metal components are corroding and the City will benefit from the adoption of a regular program of coating metal surfaces other than stainless steel. The metal components located in the below-grade wet wells and valve vaults are of primary concern.

“B” Street Pump Stations. The North PS was renovated in 1999 and new pumps were installed in 2014 and 2015. But age remains a concern for the remaining components installed with the original facility. These items include the metal roof, heating and ventilation equipment, three of the accumulators, some valves, and various piping. The condition of the North PS structure is generally good. However, this CMU and concrete structure is not designed to current code and age could become a concern within the next 20 years. The North PS pumps are also susceptible to clogging from common “flushable” items and the City wants to add a screen at the North PS.

Additionally, the South PS is 15 years old and the genset is over 15 years old, therefore age will become a consideration for equipment at these facilities within about 10 years.

Demers PS. The valves and other components in the valve vault are heavily corroded. The vault leaks and provides a damp, corrosive environment. The City should plan to replace components in the valve vault within about 3 to 5 years. The pumps at Demers PS were recently replaced with previously used pumps and are in good shape. However, corrosion is a concern for metal components in the wet well and age is a concern for electrical components.

South 97 PS. The pumps at the South 97 PS only operate infrequently due to low tributary flow and are in good condition. The City should be able to have these pumps rebuilt at least once in the future before planning for their replacement. The valves show signs of corrosion and City should also plan to replace the valves within 10 years.

Golf Course PS. The replacement of the existing Golf Course PS is planned in conjunction with the Willowbrook residential development. However, the timing of this development is unknown. As a result, the City needs to budget for rebuilding the pumps and performing other maintenance work in the next two years. Given its age and condition, full replacement of the PS should be planned in about 10 years in the event the Willowbrook development does not proceed by then.

Portable Generator. The portable power generator the City owns for use at the pump stations does not have the capacity to power either pump at the Demers facility. Currently, the City borrows a portable generator from the DRCI to use at the Demers PS. When the Demers PS is either retrofitted or expanded to accommodate industrial development, the City should plan to either purchase a larger portable generator or have a fixed genset installed at the Demers site.

2.3.4. *Conditions of Existing WWTPs*

General. Summary descriptions of existing deficiencies at the two WWTPs are presented in the following paragraphs. Assessments for the specific components at each plant are provided in Appendix E. Flow records show that treatment capacity is not an issue for Madras at existing average and peak flows. Therefore, WWTP expansions only need to be addressed with regard to future population growth. Issues relating to the future expansion and associated renovation of the WWTPs are addressed in Chapters 5 and 6.

NWWTP. The overall condition of the NWWTP is adequate-to-poor. Some equipment is in good condition due to recent replacements or repairs; but other equipment is over 20 years old. The original components still remaining at the plant are close to 40 years old.

This facility will require renovation to remain in service at its current design capacity for another 20 years. Some improvements would likely be needed within about 5 years and a major renovation would probably be necessary with about 15 years. Chapters 5 and 6 discuss plans for upgrading the facility and coordinating the upgrade with potential future expansions. Specific issues with regard to the condition of the existing NWWTP are summarized below.

- **No Influent Screening.** The absence of any raw screening allows debris in the WW to accumulate in the Primary Cells and potentially damage aeration equipment.
- **Solids Accumulation in Lagoons.** Over time grit in the influent WW and non-biodegradable solids accumulate in lagoon systems. Typically, solids should be pumped or dredged from lagoons about every 20 years. The accumulation of grit and other solids in the Primary Cells reduces effective volume and treatment capacity.
- **Aerators.** Inadequate aeration can reduce biological treatment capacity and increase odors. The City recently replaced the existing aerators in the two Primary Lagoons with used units that were obtained from Metolius. The replacement of these aerators and their motor starters should be planned in about 5 years.
- **Lagoon Liner.** There is no evidence the liner under the lagoons is leaking. However, the liner will be 50 years old in 10 years. Because of age considerations, the City should plan a complete renovation or replacement of the lagoons in 10-15 years.
- **Lagoon Transfer/Outlet Structures.** The wood walkways to these structures are deteriorating and in need of replacement within 5 years. The concrete structures and the metal components are generally adequate, but the City should plan to replace them in 10-15 years due to their age.
- **DAF Clarifier.** The air compressor, the variable frequency drives for the clarifier, and some controls components have recently been replaced. However, most process components are over 20 years old. The steel structural supports are showing signs of coating failure and corrosion. Overall the process is adequate, but age of the entire system is a concern and ongoing corrective work would be needed to keep it in service.
- **Sandfloat (DAF) Filtration.** The City no longer uses the DAF filter because Class A effluent does not need to be produced for recycling. The filter is in poor condition due to age, equipment condition, and lack of recent operation.
- **Chemical Feed Pump.** This metering pump that feeds coagulant (polyaluminum chlorohydrate) is old and in need of replacement. The City now prefers to use peristaltic pumps for chemical feed applications.
- **Chlorine Residual Analyzer.** The analyzer is adequate, but should be replaced within 5 years. The model at the NWWTP is no longer available.
- **Electrical and Controls Components.** The main electrical service components are nearly 20 years old and should be replaced within 5-10 years. Currently, the controls for the DAF Clarifier are in the process of being upgraded.

- **Coating on Equipment and Structures.** Some coatings on ferrous metal and wood structural components are failing and in need of recoating. Some metal equipment components are also in need of recoating.
- **Seasonal Operation of Clarifier and Disinfection System.** A major limitation of the DAF clarifier and effluent pumping equipment is that their outdoor installations make them subject to freezing. This prevents the City from reliably operating the equipment during cold weather. Lagoon effluent is only sent through the clarifier during the growing season when effluent can be used for irrigation. Outside the growing season, lagoon effluent must be sent to the storage pond. This approach makes irrigation practices dependent on the operation of the clarifier and there is no redundancy in the process equipment.

Continuous operation of the clarifier and disinfection system all year is a priority for the City because that would allow recycled effluent to be sent to the storage pond when cold and/or wet weather prevents irrigation. The recycled effluent could then be drawn from the storage pond as needed for irrigation without reliance on further treatment.

- **Sludge Drying Bed Capacity.** Although the drying beds are in good condition, they are not enclosed. As a result, cold temperatures and precipitation during the winter reduce evaporation and drainage from the solids. In recent years, the beds have either been nearly or completely filled to capacity. As flows increase, drying bed capacity will need to be expanded and/or the amount of liquid will need to be reduced in the biosolids hauled from the SWWTP.

SWWTP. The overall condition of the SWWTP is good. Since the plant is 15 years old, structural components and yard piping should generally be adequate for the next 20 years and beyond. The age of most equipment components will not become a concern for about 10 years. However, there are specific condition issues with some components that should be addressed within the next 3 years. These components and condition issues are summarized in the following paragraphs.

- **SCADA System.** The control system at the SWWTP provides the required functions. But much of the system hardware and software components are either obsolete or soon becoming obsolete. The system integrator has proposed the phased replacement of most SCADA hardware and software components.
- **SBR Process Building.** Cracking has occurred in the CMU walls of the bldg. and in the concrete wall between the bldg. and the adjacent digester tank. The crack in the concrete would likely require injection grouting to seal. The CMU cracks have been sealed and recoated, but their condition should be monitored to check for any changes.
- **Coating on Equipment.** Some metal equipment components are also in need of recoating.
- **SBR Diffusers.** The fine-bubble, diffuser tubes in the SBR basins are generally expected to last about 10-12 years and become more likely to fail at that point. Also the diffuser tubes lose flexibility over time and become less efficient at diffusing the air into the basins.
- **SBR Effluent Control (Decant) Valves.** The effluent control valves on the decant lines for the two SBR basins do not close tightly and leak. Cost information obtained by the City indicates replacing the valve seals would cost more than half as much as new valves. Therefore, the City should plan to replace the entire valves.
- **Disk Filter Backwash (BW) Pump and Control Valve Actuators.** The BW pump and control valves are located in a below-grade chamber that provides a damp environment during cool and cold weather. The BW pump and skid are heavily corroded. The valves are stainless steel, but the internal parts on the motorized actuators may be subject to corrosion.

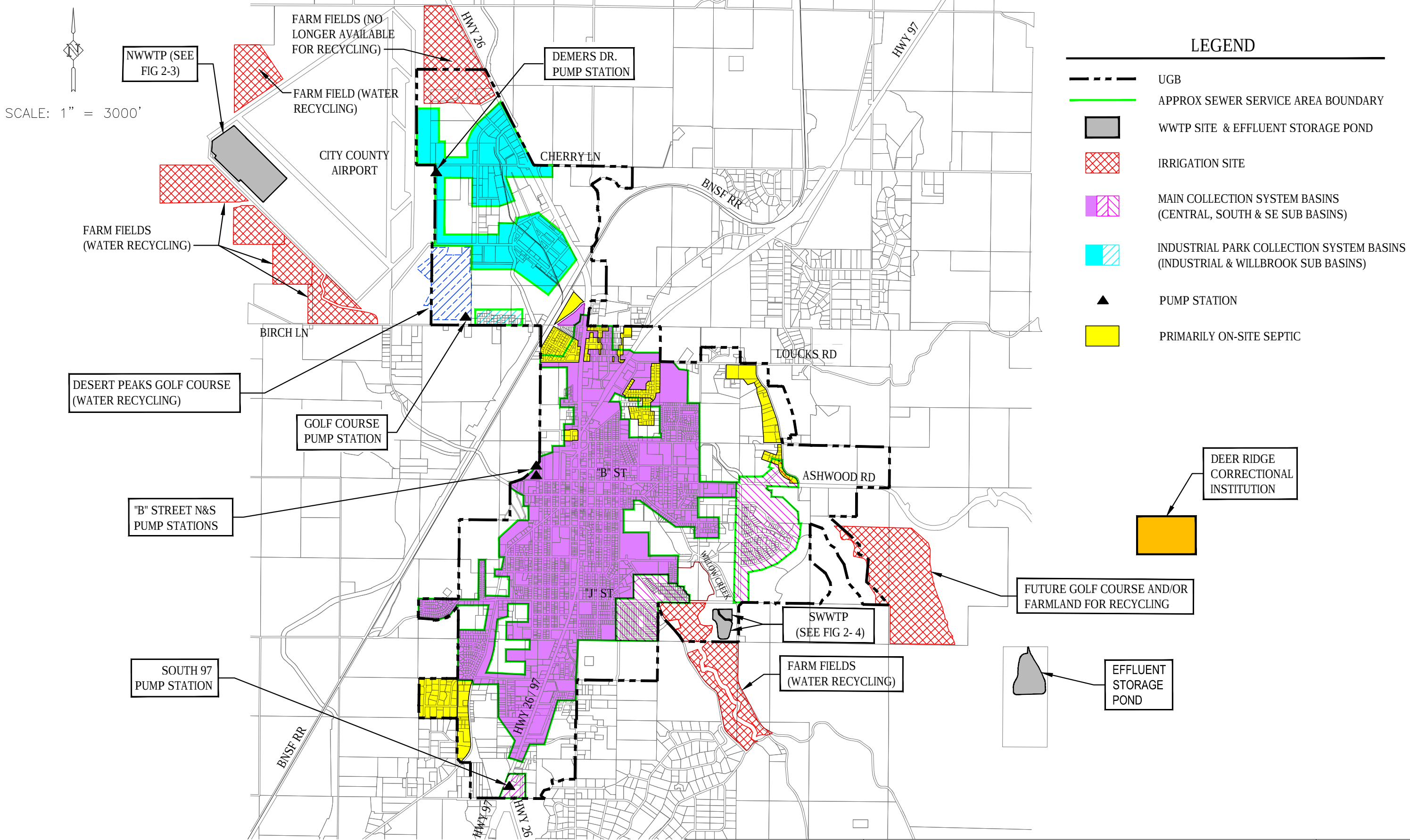
Another area of concern for the SWWTP is the operation of the aeration system for the aerobic digesters. The City plans to improve process performance and energy efficiency by installing dissolved oxygen (DO) probes in the digesters and adding a new DO control algorithm to the SCADA System programming.

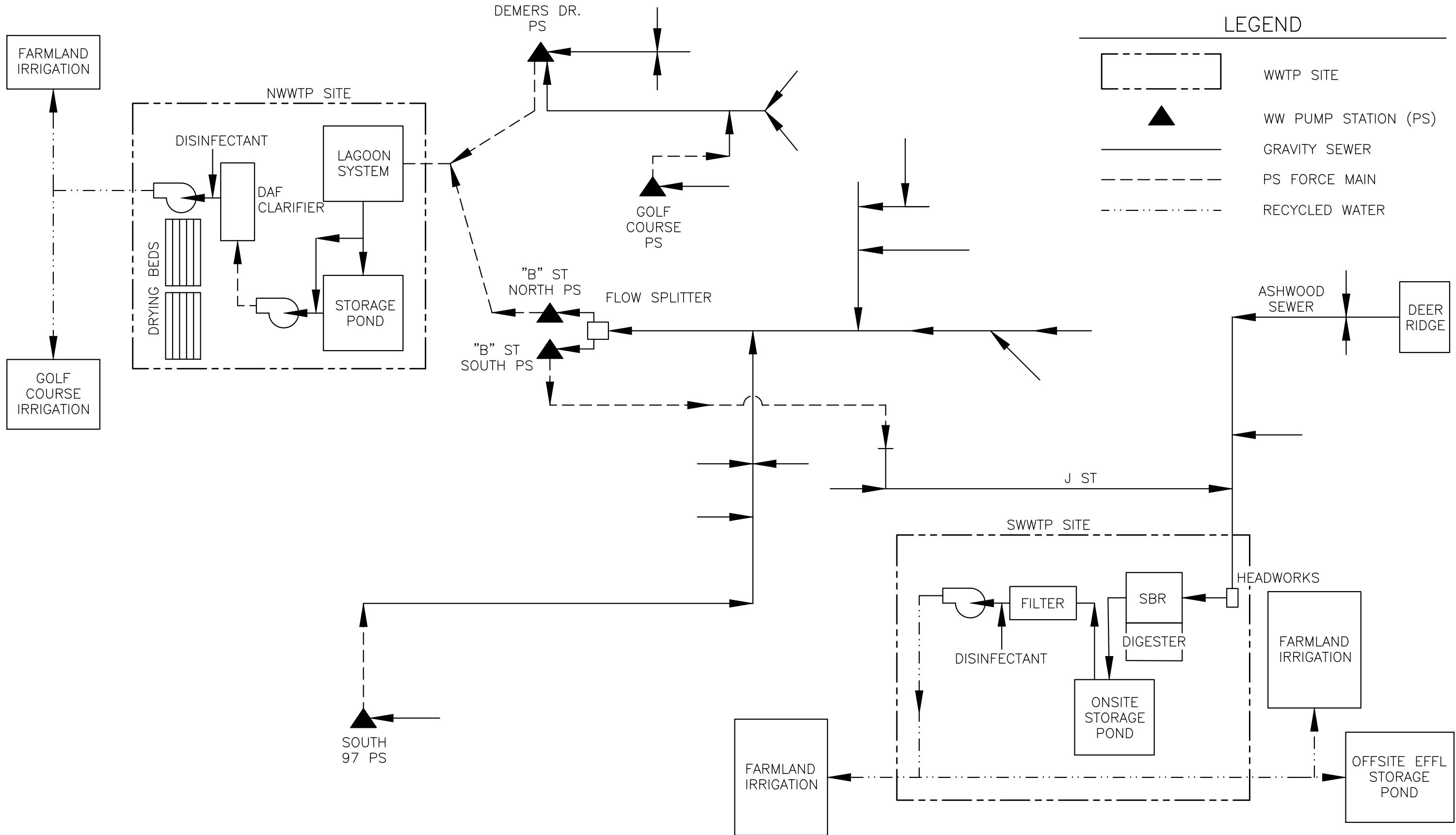
The SWWTP was designed to accommodate the addition of one more SBR treatment module and the plant may be expanded within the 20-year planning period. If a plant expansion occurs within about 10 years, it may be cost effective to plan a major renovation of existing equipment in conjunction with this future expansion.

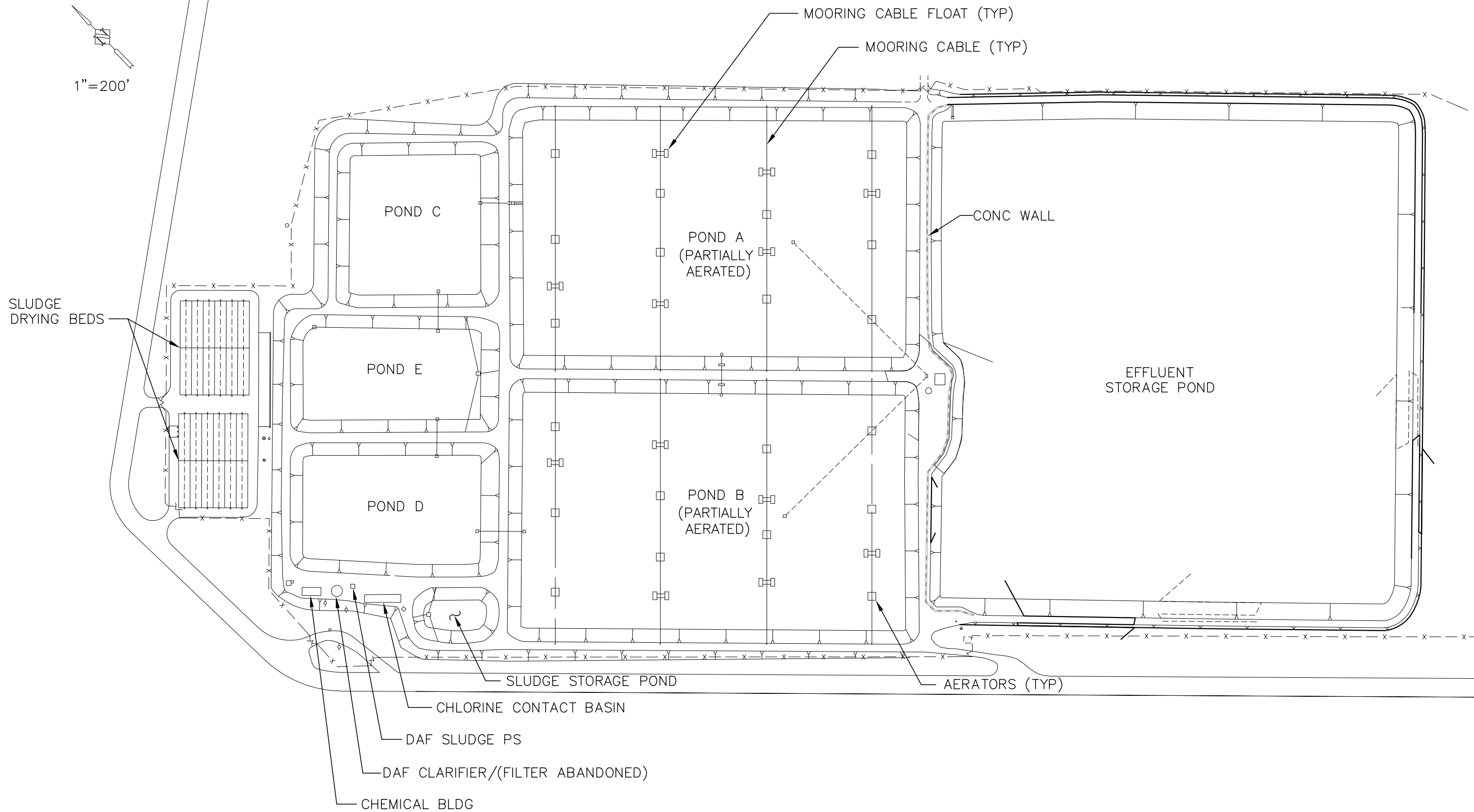
2.3.5. *Maintenance and Corrective Work Plan*

Recommendations for completing major maintenance tasks and corrective work were developed in cooperation with the City. These plans are outlined in Appendix F. The items covered by the recommended plans are non-routine tasks that typically must be performed by outside contractors or repair shops. Routine preventive maintenance tasks are not included in the recommendations.

Although the recommended tasks have been prioritized and incorporated into a schedule, the timing of the work will depend on how well the affected components continue to function and on the availability of funds. Some corrective work noted in the plans will likely be completed as part of capital improvements either in conjunction with a WWTP expansion or as a major facility overhaul.







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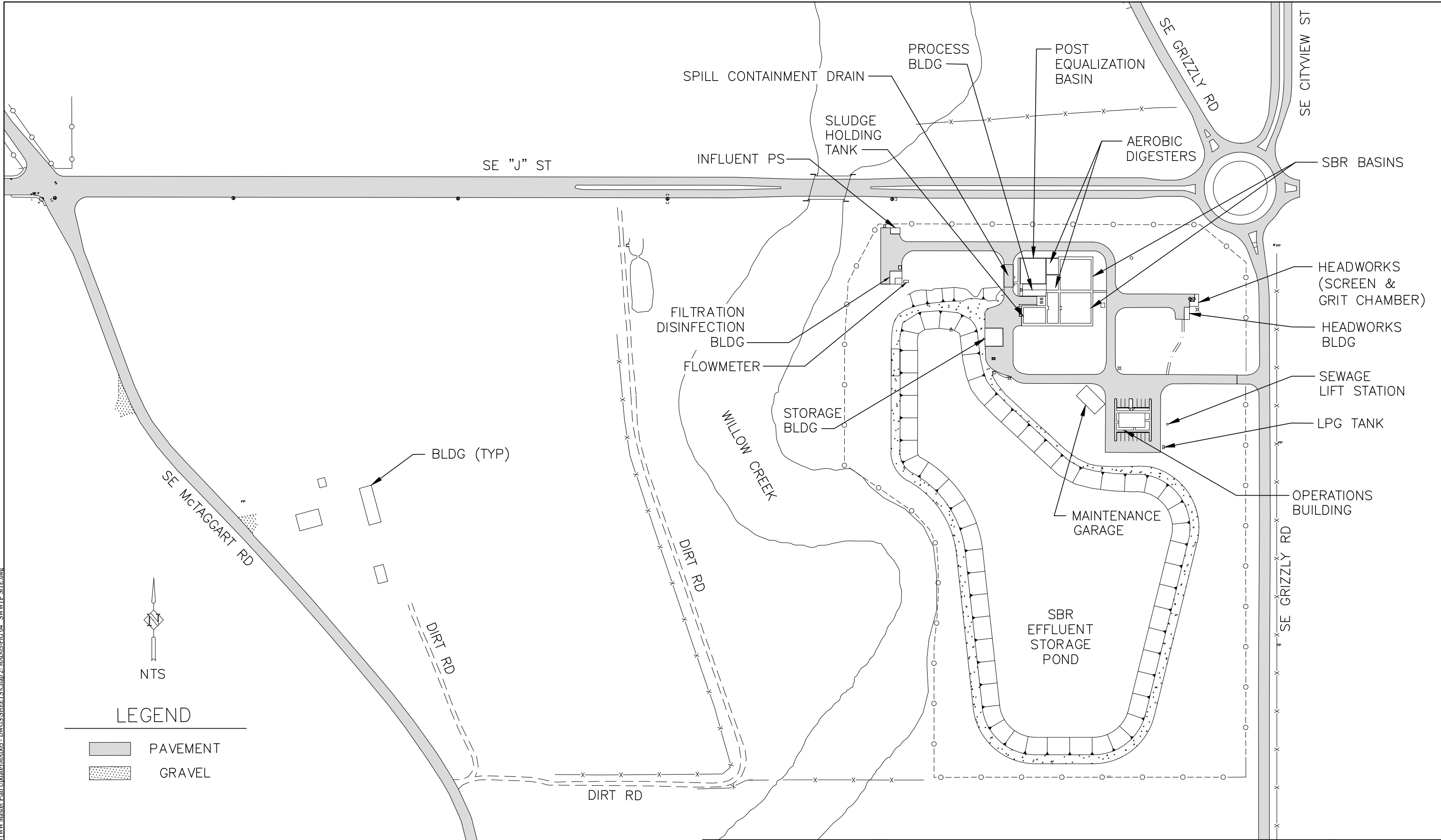
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CHAPTER 3 BASIS OF PLANNING

3.1 POPULATION AND DEVELOPMENT

3.1.1 *Recent Population Trends*

Population estimates for the City of Madras over the past 15 years are listed in Table 3-1. Estimates are also shown for the total population within the Madras UGB in the Years 2000, 2010 and 2015. These estimates are prepared annually by the Portland State University Population Research Center (PSU).

Table 3-1 Recent City of Madras Population Estimates				
Year	Population Estimates		Avg. Annual Growth Rate	
	Inside City	Within UGB	Inside City	Within UGB
2000	5,130	6,475	---	---
2001	5,200	---	---	---
2002	5,290	---	---	---
2003	5,370	---	---	---
2004	5,430	---	---	---
2005	5,600	---	---	---
2006	6,070	---	---	---
2007	6,585	---	---	---
2008	6,640	---	---	---
2009	6,650	---	---	---
2010	6,050	6,987	1.66% ⁽¹⁾	0.76% ⁽¹⁾
2011	6,225	---	---	---
2012	6,260	---	---	---
2013	6,255	---	---	---
2014	6,260	---	---	---
2015	6,265	7,484	1.34% ⁽¹⁾	0.97% ⁽¹⁾

(1) 10-year AAGR from 2000 to 2010 and 15-year AAGR from 2000 to 2015.

The City's population has increased at an average annual growth rate (AAGR) of 1.34% since the Year 2000, even though growth has fluctuated significantly from year to year depending on economic conditions. By comparison, the 1990 and 2000 U.S. Census data for Madras show an AAGR of about 4.0% over that 10-year period. Also, the PSU population estimates from 2000 to 2008, when the economy was performing better, show an AAGR of 3.3%.

3.1.2 *Current Land Use and Dwelling Units*

Figure 3-1 shows a current zoning map for Madras. According to the 2007 *Madras Urbanization Report* prepared by ECONorthwest, tax lots made up 86% of the land, or about 3,308 acres, within the UGB in 2005. The remaining 14% of the land, about 540 acres, consists of rights of way.

The Urbanization Report estimated 56% of the land in tax lots was developed in 2005 and about 35% of this land remained available for development. The other 9% was reported to be “constrained” and not developable.

The 2010 U.S. Census reported a total of 2,569 dwelling units (DUs) in the City and 2,970 DUs inside the UGB. The American Community Survey data provided by the Census for 2010-2014 show 2,363 of the units inside the City were occupied households and lists an average unit occupancy of 2.67 people per household (PPH). The PSU 2015 Coordinated Population Forecast for Jefferson County reported an average of about 2.7 PPH in 2010 for all occupied DUs within the Madras UGB. These figures are close to the average of 2.6 PPH used in the 1996 WWMP.

According to the 2007 Urbanization Report the average PPH is expected to decline in the future. Given this projection and the recent Census data, the continued use of 2.6 PPH for average unit occupancy is reasonable.

3.1.3 Population and Dwelling Unit Forecasts

Table 3-2 lists projected populations and the resulting number of occupied DUs within the Madras UGB at 5-year intervals over the next 20 years. The population forecasts were reported in the PSU 2015 Coordinated Population Forecast for Jefferson County and prepared under the Oregon Population Forecast Program.

Forecasts PSU prepares under this program are to be used for land-use planning according to Oregon state law and Department of Land Conservation and Development policy. Therefore, these data have been used to project WW flows within the existing UGB over the next 20 years. Since the 2015 PSU forecasts also include a 50-year projection, that figure is listed in Table 3-2 as well.

Table 3-2			
Madras UGB Forecasts for Population and DUs			
Year	Population ⁽¹⁾	Occupied DUs ⁽²⁾	AAGR ⁽³⁾
2015	7,484	2,880	---
2020	8,070	3,105	1.52%
2025	8,700	3,345	1.51%
2030	9,268	3,565	1.27%
2035	9,815	3,775	1.15%
2065	12,749	4,905	0.88%

(1) PSU 2015 estimate for entire area within UGB.

(2) Forecasts for occupied DUs based on an average unit occupancy of 2.6 PPH.

(3) AAGR is calculated for each incremental 5-year period to 2035 and for the 30-year period from 2035 to 2065.

The forecasts show the incremental 5-year AAGR inside the UGB steadily declining over the next 50 years. The overall 20-year AAGR from 2015 to 2035 works out to be close to 1.4% and the AAGR over the 30-year period from 2035 to 2065 is forecast to be just under 0.9%.

The controlling parameter for WW flow projections is the population forecast since flow projections are based on average per capita water use and the resulting per capita WW production. At this time there is no basis for forecasting any change in per capita water use. Therefore, a higher or lower average occupancy would not impact projected WW flows because the estimated flow per DU would be raised or lowered proportionally to reflect a consistent per capita WW production.

The projections for DUs are used for sewer modeling since WW flows are input to the model based on tributary equivalent dwelling units (EDUs). The DU forecast provides the basis for projecting a WW flow per EDU described in Subsection 3.2 and also for calibrating the distribution of EDUs in the sewer system model.

3.1.4 *Nonresidential Development*

Nonresidential Growth. The *2007 Urbanization Report* forecast that employment in Madras would grow in proportion to population growth. To be consistent with that projection, the WW flow projections in this study assume nonresidential flows will increase in proportion to growth in residential flows.

Industrial Development. Projections in the *Urbanization Report* showed more than enough industrial land was available in the City than would be needed over the next 20 years. This forecast should still be current given that no significant development has occurred since 2007 and the PSU population-growth forecast is lower than the projection used in the *Urbanization Report*.

Because the City has available land and industrial development is important to the area's economy, it is essential the City address infrastructure requirements to serve future industrial customers. To support such planning efforts, this study evaluated the WW facilities needed to serve major industrial customers at potential sites in the Industrial Park.

It is not feasible to forecast what type(s) of major industrial users might seek to locate in Madras and how much WW might be generated. Therefore, the evaluations we performed focused on two considerations. First is the question of how much capacity can be made available at the existing NWWTP, without an expansion. Second, is the matter of what improvements would be needed and what it would cost to expand the Industrial Park Collection System and the NWWTP to handle an assumed flow contribution from a major industrial user.

For planning purposes, we assumed a maximum daily flow contribution of 1.0 MGD above the flows from the forecast residential and nonresidential growth. The evaluations of WW service needs for industrial development are included in Chapters 4 and 5.

3.2 **WASTEWATER FLOWS**

3.2.1 *Analysis of Recent Wastewater Flows*

Combined monthly WW flow rates to the two WWTPs were analyzed for the period from 2010 through 2014 to establish benchmark WW flows for master planning. These flow rates are reported on the City's monthly Discharge Monitoring Reports (DMRs) that must be submitted to the DEQ as part of their WPCF permit requirements.

Records of the wastewater flow contribution from the prison were also obtained from the City for 2013 and 2014 to identify the net flows contributed by City users without the prison. Monthly average prison flows ranged between 9% and 12.5% of the total monthly wastewater flows. Therefore, to estimate the City's contribution, the total flows reported in the DMRs were reduced by 10% to exclude the prison.

Table 3-3 (following page) summarizes overall average annual WW flows and the resulting average flow per DU for all users other than the prison from 2010 through 2014. The calculation of average flow per DU excludes an estimated 106 homes within the City that have onsite septic systems.

Table 3-3 Total Wastewater Flows from City Users			
Year	Average Annual Flow (MGD)	Estimated No. of DUs on Sewers	Average Flow Per DU (gpd) **
2010	0.440	2,220	198.0
2011	0.432	2,288	188.8
2012	0.445	2,302	193.3
2013	0.452	2,300	196.5
2014	0.455	2,302	197.7

** Calculation excludes 106 DUs not connected to sewer (on septic systems).

The flow data over the last five years indicate that 200 gpd/DU is a reasonable approximation of the total, average unit flow from all users. For a conservative estimate of the peak-monthly unit flow, we applied a 20% allowance (or a factor of 1.20) to the average unit flow. The resulting peak-monthly unit flow is 240 gpd/DU. Records show the highest peak monthly flow from 2010 to 2014 (August 2012) was actually about 10% above the average flow for the corresponding year.

To estimate the average WW flow strictly from residential users, we reviewed 2015 water use records. Assuming 82% of the water used in residences becomes WW, the same factor used in the 1996 WWMP, the data indicate residential WW flow to be about 160 gpd/DU. This translates to 61.5 gpd/capita (gpcd) for a residential density of 2.6 people/DU and is consistent with master planning information from other Central Oregon cities.

A peak-monthly residential flow of 192 gpd/DU is estimated based on the peaking factor of 1.2 described in the previous subsection. When this residential portion is subtracted from the total unit flow of 240 gpd, the peak monthly contribution from all other users (commercial, institutional and industrial) besides the prison is 48 gpd/DU or 20% of the total non-prison flow.

Currently, WW flows from the Industrial Park and airport average about 40,000 gpd or about 9% of the total average WW flow. Other nonresidential and non-prison flows are generated by commercial, institutional, and small-scale industrial customers located in the remainder of the City.

3.2.2 Wastewater Flow Projections

Table 3-4 summarizes the projected WW flow rates generated from the unit flows described above and the population forecasts shown in Table 3-2. These flow projections have been used as the basis for planning for this study.

Table 3-4 Wastewater Flow Projections for Madras UGB					
Year	Flows (MGD) **		Prison Flows (MGD) ***	Total Flows (MGD)	
	Avg. Annual	Peak Month		Avg. Annual	Peak Month
2020	0.55	0.66	0.23	0.78	0.89
2025	0.67	0.80	0.23	0.90	1.03
2030	0.71	0.86	0.23	0.94	1.09
2035	0.76	0.91	0.23	0.99	1.14
2065	0.98	1.18	0.23	1.21	1.41

** Average annual and peak monthly flows based on 200 and 240 gpd/DU, respectively.

*** Prison flow based on dedicated capacity allotted to prison by agreement with ODOC.

To be conservative, the flow projections for the Year 2020 assume sewer service will be extended to 25% of the population outside the City, but inside the UGB. The forecasts for 2025 and beyond assume the entire population within the UGB will be served by sewers. In reality, some existing homes in outlying areas within the UGB probably will not receive sewer service within 20 years. However, that number is projected to be insignificant compared to the total number of DUs.

3.3 **WASTEWATER STRENGTHS**

3.3.1 *Analysis of Recent Wastewater Strength*

The strength of the influent WW to a WWTP is characterized by the concentrations of five-day biochemical oxygen demand (BOD) and total suspended solids (TSS). This influent strength is used in conjunction with flow capacity to establish the basis of planning for biological treatment processes and to estimate biosolids production.

Data from 2010 through 2014 for both WWTPs and the prison were used to analyze the WW strengths. Influent BOD and TSS concentrations are measured at each WWTP from composite samples taken once every two weeks. The results are reported in the monthly DMRs according to the WPCF permit. Weekly samples of prison flows are tested separately by ODOC for BOD and TSS concentrations.

The monthly data for BOD and TSS show significant fluctuations in the strength of the WW flowing to each WWTP and being discharged by the prison. Such fluctuations are understandable given the various WW sources, including the prison, the Industrial Park and commercial customers.

Due to infrequent sampling and the variability in the data, our analysis focused on the annual averages of BOD and TSS levels to characterize WW strengths for master planning purposes. These annual averages for the WWTP influents and the prison are summarized in Table 3-5 (following page).

Table 3-5 Annual Average Influent BOD and TSS Concentrations (mg/L)						
Year	NWWTP		SWWTP		Prison	
	BOD	TSS	BOD	TSS	BOD	TSS
2010	292	298	324	303	293	246
2011	336	301	290	304	344	384
2012	300	310	264	284	310	325
2013	324	260	258	268	294	273
2014	322	256	318	285	332	250
5-Yr. Avg.	315	286	291	289	315	296

At this time most of the flow to both WWTPs is pumped from the two pump stations at “B” Street and, in general, the strength of the WW pumped from these two pump stations will be equal. Also, since the flow to “B” Street is primarily domestic WW from residences, the BOD and TSS concentrations will typically be less than those found in the WW from the prison and Industrial Park. Our analysis bears this out, although at this time the prison is not producing WW at the strengths originally anticipated during design of the SWWTP expansion for the prison.

3.3.2 SWWTP Wastewater Strength

Data on average flows from 2013 and 2014 indicate about 27% of the SWWTP influent comes from the prison, with the rest coming from City users. Most of the City flows are pumped from the “B” Street South PS. The current SWWTP design is based on BOD and TSS concentrations of 750 mg/L in the prison flows and 250 mg/L in the flows from City users.

Although prison WW strength is thus far lower than anticipated, waste concentrations will likely increase once the facility is used at or near capacity due to improved operating efficiencies. Therefore, at this time the same design BOD and TSS concentration of 750 mg/L should be used along with the design flow of 0.23 MGD to plan for prison WW contributions.

Based on the average BOD and TSS concentrations reported for the SWWTP and the prison, the average concentrations from City users are calculated to be about 280 mg/L for BOD and 285 mg/L for TSS. These values are 12-15% higher than the 250 mg/L BOD and TSS concentration previously used for SWWTP design.

It is reasonable to expect that WW strength has become more concentrated over the last 15 to 20 years, given the increased use of low-water use plumbing fixtures and appliances, particularly in commercial enterprises. However, at this time we recommend the continued use of 250 mg/L for average BOD and TSS levels in City WW as the basis of planning. Our reasons for this approach are described below.

- Our projections for flows per EDU are conservative enough to account for WW flows that are only 12 to 15% higher in waste strength than previous designs.
- An under-loaded WWTP is less efficient to operate and prison flows may not become as concentrated as previously thought.
- Residential flows from the east side of the City are expected to make up a higher proportion of future City flows to the SWWTP. That domestic WW without significant commercial flows may tend to be less concentrated.

Table 3-6 summarizes the recommended basis of planning for influent BOD and TSS concentrations to the SWWTP. These WW strengths should be reevaluated during the preliminary design of any future SWWTP expansions.

3.3.3 NWWTP Wastewater Strength

Based on the data from 2010 through 2014 and the reasons described below, we recommend the use of 300 mg/L for the average influent BOD and TSS at the NWWTP (see Table 3-6).

Table 3-6	
Average Influent BOD and TSS Concentrations	
Description	Avg. Design Concentration (mg/L)
A. South WWTP Influent BOD & TSS	
Municipal Flows	250
Prison Flows (0.23 MGD)	750
B. North WWTP Influent BOD & TSS	
Total Combined Flows	300

All WW from the two “B” Street pump stations is expected have the same strength. On the other hand, the Demers PS serves the Industrial Park and currently receives very little residential flow. Our analysis indicates existing flows pumped from the Demers PS have highly variable BOD and TSS concentrations, with the overall average concentrations close to 550 mg/L.

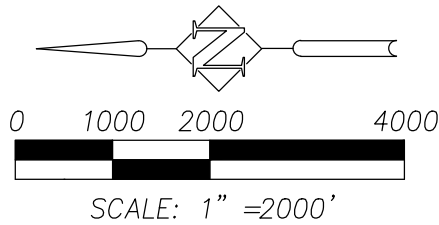
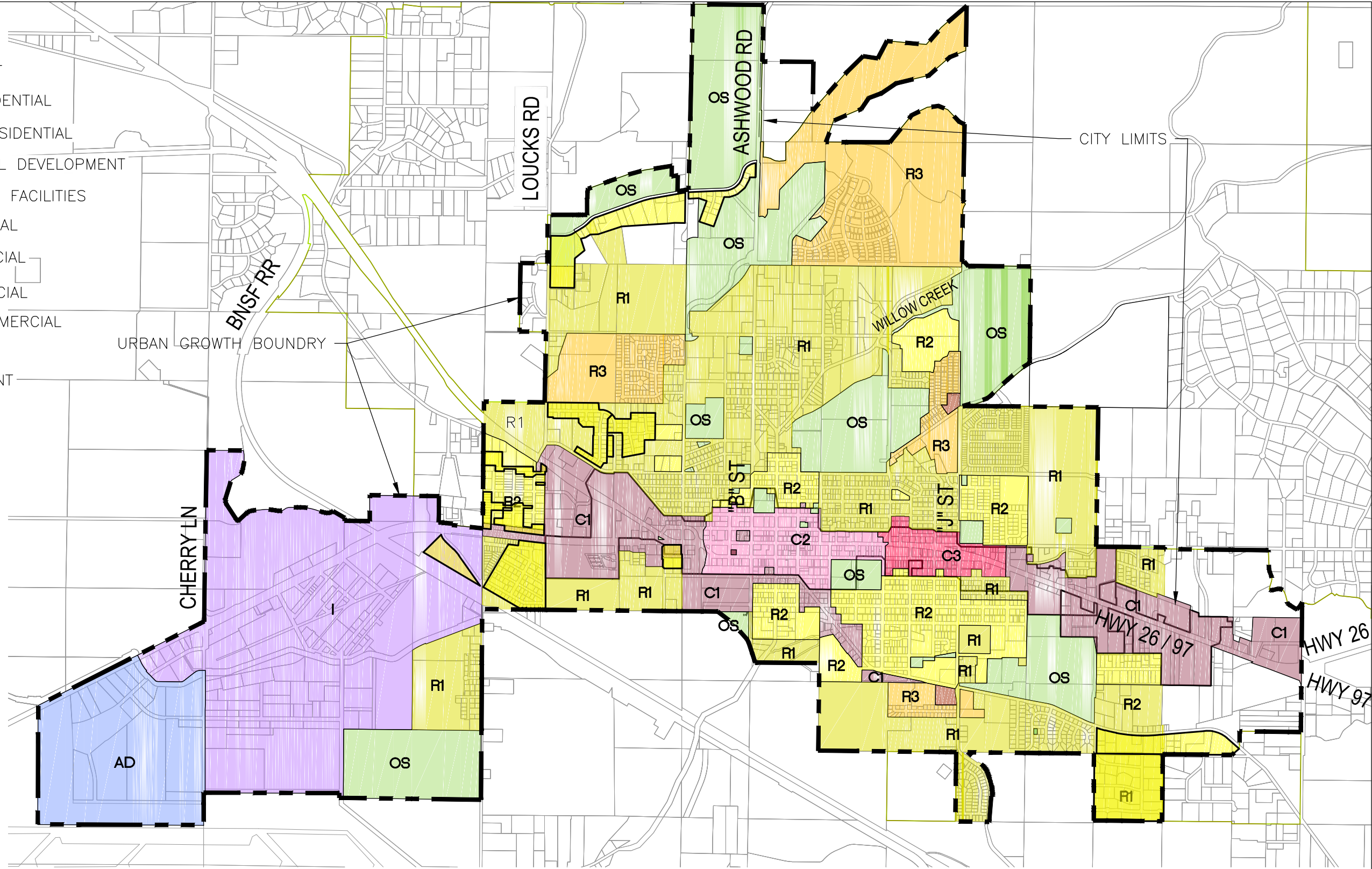
Data on average flows from 2013 and 2014 indicate 85% to 90% of the NWWTP influent comes from the “B” Street North PS, with the rest coming from the Demers PS. Although development in the Industrial Park will result in more flow from Demers relative to “B” Street, we anticipate WW strength from the Industrial Park will decline as overall flows increase.

The development of business parks and other commercial users in particular will tend to produce less concentrated WW than industrial users. Also, large industrial users will have a strong incentive to limit WW strength in order to avoid surcharges on their sewer bill from high waste loads.

The strength of the WW from Demers will need to be evaluated in more detail during the preliminary design of an expansion or upgrade to the NWWTP. Further evaluation may be warranted sooner, if development in the North Area is sufficient to force a significant reduction in flows pumped from “B” Street.

MADRAS ZONING LEGEND

- R1 SINGLE FAMILY RESIDENTIAL
- R2 MULTIPLE-FAMILY RESIDENTIAL
- R3 PLANNED RESIDENTIAL DEVELOPMENT
- OS OPEN SPACE/PUBLIC FACILITIES
- C1 CORRIDOR COMMERCIAL
- C2 DOWNTOWN COMMERCIAL
- C3 COMMUNITY COMMERCIAL
- NC NEIGHBORHOOD COMMERCIAL
- I INDUSTRIAL
- AD AIRPORT DEVELOPMENT



CHAPTER 4

COLLECTION SYSTEM EVALUATION

4.1 EVALUATION OF SEWER MODELING SOFTWARE

4.1.1 *Background*

The hydraulic analysis we performed for the 1995 Wastewater Master Plan (WWMP) used Eagle Point software to model the existing sewer system and proposed improvements. Eagle Point stopped supporting this sewer-modeling software by 2010. Their modeling software allowed users to export data on gravity sewers to spreadsheets. The software also provided both a stand-alone graphical user interface (GUI) and a GUI through AutoCAD.

As part of this project, we have evaluated new sewer modeling software to replace the software that we previously used in the 1996 WWMP. To complete this evaluation, we took the following steps.

- Identified the key considerations in evaluating and selecting the software;
- Developed a list of suitable software and evaluated the software characteristics; and
- Selected the recommended software for use on the project.

4.1.2 *Key Considerations for Evaluating/Selecting Sewer Modeling Software*

The key factors we addressed in our software evaluation are described in the following paragraphs.

Longevity/Stability. One objective is to select modeling software that we believe should have a lasting presence in the industry. We identified software that had a sufficient market share of users to demonstrate stability. Although it is difficult to project what developments may occur, a company whose product has a large number of users is more likely to have the resources to adapt and stay ahead of software changes in the field. Any software in widespread use for various modeling applications also provides a wider pool of experienced users.

Adaptability. The software should be capable of efficiently importing and exporting data in a format that both allows the use of data from previous studies and supports continued use of the data in the future. The software also needs to have the flexibility to model scenarios for changes that occur in projected patterns of development and flow.

Functionality. The software needs to provide the functions needed for modeling the sewer system. Some software is better suited for modeling stormwater rather than sewer systems. Stormwater and sewer systems experience differing hydraulic conditions and modelers tend to use different units.

Graphical User Interface (GUI). There are four common methods used to display sewer systems in conjunction with other features such as streets and utilities. The three most common methods use AutoCAD, Microstation, or ArcMap.

AutoCAD and Microstation are widely used and provide the ability to print plans, profiles and other graphical displays. ArcMap is the main software for Geographical Information Systems (GIS) and is used by many government agencies to record and map data for infrastructure and developments. The fourth GUI method is the use of a stand-alone GUI that is typically part of a proprietary software and may not offer the ability to import and export data from other GUI systems.

Simulation Options. It is useful to identify software that meets the City’s needs and offers adequate flexibility, but is not overly complex. Different models can be executed in different ways and various features are available. Different assumptions, equations, and time-based calculations are used. Alternative methods are also used to calculate infiltration and inflow (I/I) and changes from open-channel to surcharged pipe conditions. Another factor in simulation is the ability to calibrate flows and hydraulic grade lines (flow depths) so they reflect actual sewer conditions.

Education and Support. Sewer modeling and GUI software are complex and require training for efficient use. The level of support for each software is specific to the manufacturer, their distributors, and the character of the market they have developed. Some software suppliers sell expensive training or a support package for assistance. Other software is so widely used that online training is available through the software supplier’s website, user groups, You Tube videos, and other educational websites.

Cost. We compared initial costs to verify competitiveness in the marketplace.

4.1.3 *Listing and Initial Screening of Sewer Modeling Software*

The following is a list of the software we reviewed for the City of Madras Wastewater Master Plan:

<u>Software Name</u>	<u>Vendor</u>
Autodesk Storm and Sanitary Analysis	Autodesk
Eagle Point	Eagle Point
EPA-SWMM	USEPA
H2Omap SWMM/ InfoSWMM	Innovyze
Mike Urban	DHI
SewerCAD/SewerGEMS	Bentley

We eliminated the following products through an initial screening process.

- Eagle Point Software was ruled out because they no longer provide support for the product.
- EPA-SWMM was ruled out because the GUI interface is standalone and the software is not user friendly.
- H2Omap SWMM was ruled out due to its limited functionality relative to InfoSWMM.
- Mike Urban was ruled out because the software did not support the latest form of the GUI.

4.1.4 *Shortlist and Evaluation of Sewer Modeling Software*

The following paragraphs summarize our evaluations of the products that remained after screening. Appendix G provides a comparison of the features of all the modeling software.

Autodesk Storm and Sanitary Analysis. This program is a separate Autodesk module that allows users to develop a Pipe Network in Autodesk Civil 3D (C3D) software and export it to the module. The module uses a standalone GUI to represent the model in the software.

Distributors for Autodesk software told us a software consultant would be required to convert the 1995 WWMP data into a format we could import into either C3D or Storm and Sanitary Analysis. Alternatively, we could import the 1995 data by redrawing the entire system as a C3D Pipe Network.

The drawings from the 1995 WWMP were in Civil 2D and had only the locations of the sewers and manholes. The data on manhole rim elevations and pipe inverts was in a separate spreadsheet that would have required a time-consuming process to import into C3D.

As part of our evaluation of the Autodesk software, we put together a small C3D Pipe Network with pipes, manholes and an outlet. We did import the data into the Storm and Sanitary Sewer Analysis module; but we had difficulties exporting data back to C3D when changes were made. We found it would require changes to the way the programs identified the various components during importing and exporting to allow the software to properly model the components.

Based on all of the above considerations, Autodesk Storm and Sanitary Analysis software were considered too inefficient and cost prohibitive for the Madras WWMP model.

InfoSWMM. This program is a commonly-used, commercially-available variation of the EPA SWMM program and HHPR has a version available for use on the WWMP project. Table 4-1 provides a partial list of Oregon Cities and Agencies that have used InfoSWMM.

Table 4-1 List of Cities and Agencies Using InfoSWMM in the State of Oregon	
Ashland	Klamath Falls
Bend	Medford
Clean Water Services	Milwaukee
Corvallis	Newberg
Dallas	Oregon City
Eugene Water & Electric Board	Portland
Grants Pass	Prineville
Gresham	The Dalles
Hermiston	Tualatin
Hood River	Wilsonville

InfoSWMM has more functionality than H2OMap SWMM and is capable of doing stormwater systems alone or with combined sewer systems. InfoSWMM also provides simulations that allow modeling of pipe surcharging where the conditions are known to occur.

InfoSWMM uses the ArcMap GUI, which is also used by Jefferson County. The City has said they can work with the County to map the City's sewer system. The data from the 1995 WWMP can be imported and drawn up by InfoSWMM software in ArcMap format. The additional data for changes to the collection system since 1995 can be easily added to the model and its maps.

Data can be exported from the InfoSWMM software into a Microsoft Excel spreadsheet that can be used by the City. The data can also be imported and exported easily into C3D in a 2D format. This feature is useful for providing a sewer map the City can view using AutoCAD Lite along with maps of zoning and future developments. The software also allows data to be exported as a "kml" file that can be viewed on free software like Google Earth. Reports are output in HTML and so can be viewed with any web browser.

SewerCAD and SewerGEMS. Bentley, the maker of Microstation and InRoads, supplies these software products. Bentley supplies different versions of these programs with varying costs and functionality.

In its simplest form, the academic SewerCAD is offered with a book on collection system modeling for \$195. This version of SewerCAD functions with a standalone GUI. More advanced forms of SewerCAD can operate with AutoCAD, Civil 3D, Microstation, and Inroads.

SewerCAD has a proprietary hydraulics simulation engine acquired from Haestad Methods that can perform steady state and extended period simulations. It is also capable of capacity, backwater and automatic constraint-based analyses. Other features of SewerCAD include:

- the ability to do polyline to pipe conversion from “dxf” and “dwg” files;
- the ability to import and export data from spreadsheets, databases, shapefiles and OLE DB; and
- the ability to extract elevations from contours, point, shape files, CAD drawings and surfaces.

SewerGEMS is a superset of SewerCAD, including all its functionality, plus additional features. This program has the highest functionality and interoperability of any of the software we reviewed, but also has the highest price.

4.1.5 *Selected Sewer Modeling Software*

InfoSWMM has been selected as the sewer modeling software for this Master Plan Update based on the following reasons.

- The software is widely used for sewer modeling which suggests the potential for longevity in the wastewater industry.
- The software has a high level of functionality for modeling sewer systems.
- The data from the 1995 Wastewater Master Plan can be imported to InfoSWMM for system modeling and mapping.
- The additional data for the system changes since 1995 could be readily input to the model.
- The ArcMap GUI that InfoSWMM uses is also used by the County for GIS data. This allows the City to work in conjunction with the County to update the sewer system data and mapping.
- Data can be exported from the software into Microsoft Excel spreadsheets, which is the format used by the City.
- The data could be imported and exported readily into Civil 3D in a 2D format. This allows the City to view maps with AutoCAD Lite.

4.2 **MODEL OF EXISTING CONDITIONS**

4.2.1 *Sewer Data Input*

The sewer model was developed in two segments to analyze the gravity sewers in the two existing collection systems, the main system and the Industrial Park system. Chapter 2 includes general descriptions of these two collection systems.

The starting point for the new model was the sewer data from the 1995 WWMP model. These data were imported directly from data spreadsheets. We then input data from available records for sewer installations that occurred since the 1995 modeling effort. Table 4-2 (following page) lists the sewer projects and developments constructed from 1995 through 2015 with the number of new dwelling units (DUs) and/or commercial equivalent dwelling units (EDUs) associated with each.

Any gaps or inconsistencies in available records and in the 1995 model were filled through field investigations and surveys performed by either City staff or HHPR surveyors. The result is a backbone sewer model that represents to the extent feasible actual characteristics (sizes, lengths, lines and grades) of the sewer systems.

Figures 4-1A to 4-1D provide an overall map showing the two existing collection systems that were modeled. Appendix C includes a tabulation of the pipe and MH data for the existing sewer segments in the two collection system models.

Table 4-2 Previous Sewer Projects and Developments from 1995-2015			
File No.	Project Name	Residential Units	Commercial EDUs
1	Berg Drive & Related Improvements		6
3	10th Street		
4	Buff St		
5	Lincoln Heights	12	
6	Sunnyside Estates Phase 1	29	
7	Ashwood Rd FM & San Sewer		
8	Cherry Lane		2
9-10	Chestnut Sewer Phases 1 - 2	9	3
16	Jefferson Park Business Center Phase 1A		2
17	Jefferson St. Sewer	14	1
18	Madras Hotel Crosskeys		
19-21	Morning Crest Phases 1 - 6	87	
23	Fourth, Oak to Poplar		
24-25	Oak St, 4th to Hwy 26 & 6th to 7th		
26	Poplar and Hwy 26		
28	Park Place Phase 1 (1996/1999)	37	
30	Polk Street		
31	Lee Street to Tops and B St PS to North Wye Sewer		
32	Sunrise Estates	87	1
34	South 97 Sewer		2
35	Strawberry Heights Sub Phase IV	127	1
36-36A	The Pines Phases 1 - 2	83	
37A	City View Street Intersections (Yarrow)		
37B	Yarrow City View RAB	15	2
37-38	Yarrow Phases 1 - 2		
41	Bi Mart Sewer June 1998		4
42	Buff Estates [18 Lots 1999]	21	2
43	H & Marshall Apt Sewer Extension		
44	Kinkade Crossing (50 Lots 2006)	8	
46	Juniper Crest	101	
47	Sage Hill Addition		
48	Wagon Wheel	28	
49	J Street - City View Improvements		
50	Madras 1B		
51	Madras Eff Pump Main and Pond		
52	Oak Terrace	3	
53	Bean Drive 2nd Addition	1	2
54	S&P Estates (sewer built but no development)		
55	Madras Aquatic Center		
56	Fairway Estates	41	2
Total EDUs		703	30

4.2.2 Existing Wastewater Flow Inputs

Main Collection System. Existing WW flows were input to the model by developing estimates of the current DUs tributary to each sewer segment. We then applied the average daily flow per DU previously reported in Chapter 3 to estimate the average flow contributions throughout the collection systems. The model produced a total average flow of 0.50 MGD in the main collection system, which compares well to recent flow data for the system (see Chart 4-1).

A diurnal curve is used to model peak flows throughout the system and address their impacts on the sewers. This curve represents the flow variations that occur over the course of a typical day. Sewage flows tend to drop overnight and then rise during the daytime with flow spikes occurring in the morning and evening. The highest peak flows in Madras generally occur in late morning around 10:00 AM.

The ratio of the peak flow to the average daily flow is termed the peaking factor. The 1995 study applied a peaking factor of 2.6 to establish a peak flow for the model. Our analysis of recent flow data found that a peaking factor near 3.1 better characterized the current ratio of peak to average flows. The diurnal curve we developed for the model was based on this higher peaking factor.

Industrial Park Collection System. Existing WW flows in the Industrial Park Collection System consist of contributions from businesses and a few residences in the Industrial Park plus the output from the Golf Course PS. Since the WW reaching the Golf Course PS is all residential flow, the same average daily flow per DU and peaking factor described above were used to estimate flows the tributary flows.

Water use records were analyzed to estimate flows from users in the Industrial Park. A factor was applied to the metered water use to estimate the proportion of the water that becomes wastewater. This factor is based on overall water use and wastewater treatment plant (WWTP) flows in the City.

4.2.3 Calibration of Wastewater Flow Inputs

Main Collection System. The model for the Main Collection System was calibrated by comparing the depth predicted from the execution of the model to the actual flow depth measured by Madras Public Works. The actual depth was measured in the 18-inch interceptor near the “B” Street PS (MH I-1). A comparison of the depth predicted by the model simulation to the actual depth measured in the field found the model was sufficiently accurate in its representation of flow conditions.

We also calibrated our model of the Culver Highway sewer using velocity and depth data collected along that line in 2008 by a temporary monitoring device. Our analysis of the 2008 data indicates peak flows in that sewer may be higher than our initial estimates. Therefore, we used a higher peaking factor of 4.0 for the Culver Highway sewer in the model. The data also suggest the condition of this sewer warrants the use of a higher Roughness Coefficient than we used for the other sewers. As a result we used a coefficient of 0.015 for the line along Culver Highway, instead of the 0.013 coefficient we input for the other sewers.

Industrial Park Collection System. We used estimated WW outputs from the Demers PS to check the accuracy of the modeled flows for the Industrial Park. Estimates of PS output were generated from data on Demers pump run times and the design pump output. Since the pumps at the Demers PS were recently rebuilt, the pump output was assumed to be equal to the rated capacity of 290 gpm at the design total dynamic head (discharge pressure). The outputs estimated from pump run times were consistent with the WW flow rates developed in the model from factored water uses.

4.2.4 *Modeling Results for Existing Conditions*

The model found no surcharging conditions under existing peak flows for any gravity sewer pipes in both the Main and Industrial Park Collection Systems. These results indicate the existing gravity sewers should have adequate capacity for existing flows. Additionally, our analysis shows the existing pump stations have adequate capacities to pump the estimated peak flows.

4.3 **MODEL OF PROJECTED FLOW CONDITIONS**

4.3.1 *Modeling Future Developments*

To estimate the locations and amounts of future WW flows entering existing sewers, projections were generated of a probable distribution of future developments in the City. For modeling purposes, the projected DUs from these developments were translated into flow contributions entering existing sewer MHs.

City records were used to identify all proposed development projects along with the proposed numbers of DUs and EDUs associated with each. Records show the total number of DUs in proposed developments, plus build-out of existing developments, is more than twice the number of DUs forecast to be built over the next 20 years by Portland State University (PSU). In fact, the proposed developments are close to equaling the 50-year growth forecast from PSU.

To generate a development distribution more consistent with growth forecasts, proposed developments that were most likely to proceed within the next 20 years had to be identified. In support of this effort, the City performed an analysis that estimated relative development timing based on infrastructure needs and divided proposed developments into separate timing categories. The City generated these estimates based on an analysis of the relative street and sewer improvements required for each project. Other infrastructure elements did not appear to be significant factors in development timing. A planning memo that describes the City's analysis is included in Appendix H.

The City's development-timing estimates were based only on infrastructure considerations and did not address the socio-economic issues PSU uses to generate population forecasts. The infrastructure requirements for the proposed developments are estimated to have less of an impact on development than economic and demographic factors. The result is the PSU forecasts show less development occurring over the next 20 years than the City's timing estimates.

Our sewer analysis under future conditions is based on a more gradual development rate than shown in the City's analysis to correlate more closely with the PSU forecasts used as the basis of planning. To be conservative, our analysis includes approximately 5% more DUs than are forecast by PSU. We also focused on areas where developments would have the greatest impact on existing sewers.

Table 4-3 (following page) lists the proposed development projects identified for this study with associated DUs and EDUs. The developments are divided into four development-timing categories to address incremental impacts on the sewers in the near term, mid term and long term. The DUs and EDUs in developments listed under Categories 1 through 3, plus EDUs resulting from full build-out of ongoing developments, were included in the model of projected conditions in 20 years.

Table 4-3 also identifies the existing sewer MH that would receive the WW flows from each proposed development. All these developments, except the planned Willowbrook Subdivision, will be tributary to the Main Collection System and are shown in Figures 4-2A through 4-2C.

Table 4-3 Estimated Timing for Proposed Developments				
File No.	Project Name	DUs	EDUs	Existing MH Receiving Flow
Category 1 – Near-Term Planning Forecast (0-5 years)				
6	Kinkade Heights	5		A.17-1.1
12	Parkside Manor	8		A.14-5
14	Sonrise View Estates	8		E12
20 & 21	Yarrow Phases 1 & 2	150		CV-14 & CV-08
26	Lincoln Heights	5		M3.1
29	Morning Crest	33		SP-11
30	Kinkade Crossing	15		A.15-3
Subtotal – Near Term		224		
Category 2 – Mid-Term Planning Forecast (5-10 years)				
7	Meridian Park (Partial)	79		MH-02
9	Mule Deer Estates (Partial)	27		MH-06
11	Park Place (Partial)	21		N.2
15	Sunridge	75		B-10
40	Jefferson Street Area	38		JS-8A
Subtotal – Mid Term		240		
Category 3 – Long-Term Planning Forecast (10-20 years)				
1	Ashwood Heights	30		CV-26
7	Meridian Park (Partial)	79		MH-02
9	Mule Deer Estates (Partial)	27		MH-06
11	Park Place (Partial)	20		N.2
13	Roadrunner	23		MH-01
15	Sunridge	75		B-10
16	Sunset View	30		MH-01
19	Willowbrook (Partial)	77		IA-12
22 & 23	Yarrow Phase 4	65		CV-03
25	Strawberry Heights	48		SH-3
Subtotal – Long Term		474		
Category 4 – Beyond Planning Period (20-50 years)				
1	Ashwood Heights	30		CV-26
2	Cascade View Estates	230		SP.12-3
3	Eldorado Estates	133		SH-17
4A	Fairgrounds Estates	48		M13
5	Grizzly Estates	41		MH-01
7	Meridian Park (Partial)	80		MH-02
8	Miller Crossing	50		M5-5
10	Belmont Basin	17		M5-3
17	The Ridge	149		SH-17
18	Willow Heights	39		MH-01
19	Willowbrook (Partial)	76		IA-12
22 & 23	Yarrow Phase 3	65		CV-03
24	S&P Estates	147		SP-5
25	Strawberry Heights	48		SH-3
27	Sunnyside Estates	30		M5-12
32	South Town Center	90	160	M13
Subtotal – Category 4		1,273		
Total Proposed DUs		2,211		---

The development projects show a future growth pattern that differs significantly from the projections used in the 1996 WWMP. The proposed projects more closely coincide with the layout of the existing Main Collection System than the 1996 WWMP showed. The result is no new pump stations have been identified to meet projected needs. Instead future flows are projected to either be tributary to existing collection system pump stations or, in the case of the SE sewer basin, be conveyed by gravity to the South WWTP site.

4.3.2 Service Extensions to Existing Residences With Onsite Disposal Systems

Areas inside the UGB with existing homes not served by sewers were identified from available records. Based on input from the City, the unsewered areas projected to receive sewer service within 20 years were also identified and incorporated into the sewer model for future conditions. Table 4-4 (Page 4-9) lists these unsewered areas that were added to the model along with the receiving MH and interceptor in the existing sewer system that would handle the flows from each.

Table 4-4				
Projected Sewer Service Timing for Currently Unsewered Areas				
File No.	Unsewered Area	Projected DUs	Projected Timing (Yrs.)	Existing MH Receiving Flow
34	Bean 2nd Addition	21	0-5	CV-26
35	Bel Air	30	0-5	A.4-16
36	Cedar Heights	4	0-5	A.4-18
37	Herzberg Heights	32	0-5	A.4-18
38	Manathiola Heights	30	6-10	G-10
39	Mountain View	74	6-10	LS-11
40	North Unit	55	6-10	JS-8A
41	Sky Ridge	24	6-10	LS-14
42	Watts Addition	4	11-20	A.4.5-1
N/A	Juniper Heights (Burns Dr.)	30	16-20	M13
Total DUs		304		

Other unsewered areas within the UGB are not considered a high-priority for sewer extensions due to their distance from existing sewers and the lack of reported problem areas. It is anticipated these other areas will only be brought into the City limits and receive sewer service if plans for redevelopment are proposed. Given the current 20-year forecast for growth and the availability of developable land in the City, such redevelopment is not expected to occur within 20 years. Therefore, sewer extensions to these area were not evaluated or modeled in this study.

4.3.3 Modeling Results for Main Collection System

General. The model of the Main Collection System was executed with projected flows from developments and unsewered areas as described above to identify future deficiencies. Appendix I includes an example printout of the modeling results for a maximum-flow, buildout scenario incorporating all the projected developments identified in Tables 4-3 and 4-4.

Projected Sewer Deficiencies. Modeling of projected conditions in the Main Collection System shows two existing sewer sections lack the capacity needed to handle projected peak flows. These sewer deficiencies are described in the following paragraphs.

1. **North Wye Sewer.** Our model shows projected sewer connections in the north part of the Main Collection System will cause surcharging in the 8-inch sewer from the intersection of U.S. 97 and Cedar Street down to the intersection of 4th and Maple Streets. At that point the pipe size increases to 18-inch.

The area tributary to this North Wye sewer is projected to have 294 additional DUs connected within 20 years and a total of 671 DUs after full development. These consist of an estimated 223 existing houses in unsewered areas and 448 DUs that are proposed as future developments.

The unsewered areas tributary to the North Wye sewer consist of the following:

- East of U.S. 97 – Belair, Cedar Heights and Herzberg Heights (66 DUs)
- West of U.S. 26 – Mountain View, Sky Ridge and Watts Addition (102)
- Between U.S. 26 and U.S. 97 – North Unit (55 DUs)

The future developments that will contribute to this total include the following:

- Morning Crest and Jefferson Street Area (71 DUs within 10 years)
- Cascade View Estates and S&P Estates (377 DUs beyond 20 years)

2. **Culver Highway Sewer.** Modeling results following the calibration described in Section 4.2.3 show projected flow increases in the Culver Highway Sewer would exceed the pipe capacity. The capacity shortfall is projected to occur in the 8-inch sewer along Marshall Street, Culver Hwy. and 1st Street, between “G” and “B” Streets. This sewer connects to an 18-inch interceptor at 1st and “B” Streets.

The following developments proposed for the south and west sides of the City are forecast to add 591 tributary DUs/EDUs.

- Lincoln Heights, Park Place, and Sunridge (196 DUs projected within 20 years).
- Fairgrounds Estates, Miller Crossing, Belmont Basin, Sunnyside Estates and Madras South Town Center (235 DUs and 160 EDUs projected beyond 20 years).

We also estimate there are 30 houses in the unsewered, Juniper Heights area that would be tributary to this sewer.

Table 4-5 summarizes our estimates of the current and projected conditions for the North Wye and Culver Highway sewer sections where our model shows capacity deficiencies. The pipe capacities are listed for the sewer sections in each interceptor with the mildest slopes. We have listed available capacities and projected tributary growth both in terms of peak flow rates and EDUs.

Table 4-5						
Existing Sewer Capacities – North Wye & Culver Hwy. Interceptors						
	Model Results for Existing Conditions		Estimated Available Capacities		Projected Tributary Developments	
Existing Sewer	Pipe Capacity (MGD)	Peak Flow (MGD)	Peak Flow (MGD)	No. of EDUs	Peak Flow (MGD)	Total No. of EDUs
8-inch North Wye	0.47	0.17	0.30	400	0.67	677
8-inch Culver Hwy	0.43	0.17	0.26	265	0.64	591

Our analysis shows projected, 20-year growth tributary to each interceptor may not exceed available capacity. However, we project that available capacity would be exceeded well before proposed developments are fully constructed. Since it is impossible to accurately predict the pattern and pace of developments in each tributary area, the City should plan to address both sewer deficiencies within the planning period.

Projected Sewer Extensions. New public sewers would need to be extended in a few locations to connect proposed developments to the existing system. Each location that would require a sewer extension is described in the following paragraphs and the recommended improvements to address these projected needs are described in Subsection 4-4.

1. **Madras South Town Center Sewer Extension.** The development of Madras South Town Center and Fairgrounds Estates will add a proposed 250 DUs and EDUs. A sewer extension will be required to connect these developments to the existing Main Collection System.
2. **Eldorado Estates and The Ridge Extension Sewer.** These two developments propose the construction of 282 DUs and will require a sewer extension to connect to the existing system.
3. **Willow Creek North Sewer.** The area on the west side of SE City View Street between “B” and “J” Streets slopes down toward Willow Creek and gravity service cannot be provided by the sewer in City View. The following four developments with a total of 133 DUs are proposed for this area.
 - Roadrunner and Sunset View (53 DUs projected within 20 years)
 - Grizzly Estates and Willow Heights (80 DUs projected beyond 20 years)

4.3.4 *Modeling Results for Industrial Park Collection System*

General. The model of future conditions in the Industrial Park Collection System was first executed with projected flows from the planned Willowbrook development to identify deficiencies. Additional modeling runs were then executed to address potential future need from major industrial users. The modeling results of future conditions are discussed below. Appendix I includes an example printout of the modeling results for buildout conditions at Willowbrook.

Willowbrook Subdivision. The only planned residential development that will be tributary to the Industrial Park sewers is the Willowbrook Subdivision (153 DUs projected). The capacity of the existing Golf Course PS must be expanded to pump the projected peak flows from Willowbrook to the Industrial Park and a replacement PS is planned for the development.

Results show no surcharging will result in the existing Industrial-Park sewers and flows to the Demers PS will be below the existing pumping capacity when Willowbrook reaches buildout.

Industrial Development. Currently, there are no major industrial developments proposed for the Industrial Park that would provide a basis for estimating future WW contributions. Given this limitation, it is typical practice to model future industrial flows by identifying a per-acre flow factor and multiplying this factor by the approximate area tributary of each MH. The per-acre flow is commonly based on current flows from existing industrial users.

The model shows the existing sewers and Demers PS are adequate for the development of the Industrial Park with future users that contribute similar WW flows as the existing users. However, it is a priority for the City to prepare for major industrial users that could potentially produce high peak WW flows not represented by a general flow factor.

To address sewer needs for major commercial/industrial developments, we estimated available capacities in each existing interceptor in the Industrial Park. These estimates were identified by subtracting the estimated, existing peak flow from the calculated sewer capacity in each segment with the mildest slope. Table 4-6 (following page) shows the results we calculated for each of the Industrial Park interceptors.

Since flows in the Industrial Park are currently quite low, most of the interceptor capacity remains available for developments. Each of the three interceptors can accept additional peak flows of 0.73 to 0.77 MGD at their upstream end without surcharging. This translates to average design flows of about 0.25 MGD, assuming a design peaking factor of 3.1.

Table 4-6 Existing Sewer Capacities – Industrial Park Interceptors					
Existing Interceptor	Pipe Size (inches)	Min. Slope (%)	Design Depth (d/D) *	Pipe Capacity (MGD)	Available Peak Flow Capacity (MGD) **
Berg Drive	10	0.25	0.90	0.75	0.73
Cherry Lane	8	0.40	0.90	0.79	0.77
Demers Drive	10	0.29	0.90	0.80	0.74

* d/D = allowed ratio of flow depth (d) to pipe diameter (D) per City Standards.

** Available capacity = calculated capacity minus estimated existing peak flow.

Major Industrial Flow Contributions. An additional analysis was performed to address the impacts of major industrial developments on the collection system. This analysis applied a conservative flow at upstream manholes in the system to model flows from large-scale contributors and adjacent developments. For planning purposes, we applied a 1.0 MGD average flow with a peaking factor of 3.5 to model the combined future flows from major users and adjacent developments.

The locations chosen for applying the WW flow were selected where adjacent areas have large parcels of land available for major developments. These sites, labeled as the North, South and East Industrial Areas, are shown in Figure 4-2A. The North Area would be tributary to the Berg Drive sewer and the South Area would be tributary to the Demers Drive sewer, or directly to the Demers PS. We investigated alternatives alignments for serving the East Area, across U.S. 26, either along Cherry Lane or along Hesse Road, Adler Street and Demers Drive.

As Table 4-6 indicates, the existing sewers do not have available capacities sufficient to convey the future peak industrial flow applied to model major users. The sewer improvements required to handle the flows are discussed in Subsection 4-4.

Land along the northwest side of the airport is identified in the 2010 Airport Master Plan Update as a future redevelopment area. Because of the topography of the airport property, potential developments in this area could not be served by gravity sewers tributary to the Industrial Park Collection System. The area would require a PS and a force main, most-likely extending directly to the North WWTP. Therefore, development of this area would not impact the existing sewers and Demers PS.

4.4 SEWER IMPROVEMENTS

4.4.1 *Alternative Installation Technologies for Sewers*

Capacity problems in existing sewers can be addressed either by installing a second, parallel sewer to increase capacity or a replacement sewer with larger diameter. A parallel sewer is more cost effective if the existing sewer can be anticipated to have at least 20 years left in its service life and there is adequate space in the right of way for a second pipe. The interceptor sewers in the Industrial Park are less than 30 years old and parallel sewers are likely to be more cost effective for capacity upgrades proposed within the next 20 years.

It is often more cost effective to augment the capacity of an older sewer through replacement. A replacement sewer can either be installed using open-trench or trenchless methods. Trenchless technologies for sewer installations generally include pipe lining, pipe bursting, horizontal directional drilling (HDD), and pipe boring and jacking.

The trenchless options that have the most potential of being cost effective for replacement applications in Madras are pipe lining and pipe bursting. Boring and jacking or HDD would typically be considered for crossings under railroads or highways when pipe lining or bursting is not feasible.

Lining an existing pipe is considerably less costly than open-trench installation of a new pipe. But lining is suitable only when no increase in pipe capacity is required and the existing pipe is in adequate condition to support the lining. Alternatively, pipe bursting is a practice that allows an increase in sewer size by bursting the existing pipe and pulling a larger pipe through the opening.

Pipe bursting can be less costly than open-trench installation in congested, high-traffic areas and in other circumstances. Key considerations in evaluating pipe bursting are described below.

- Pipe size increase – the incremental increase in pipe diameter that pipe bursting can accomplish is limited.
- Existing pipe material – certain materials, such as asbestos cement, are better suited to bursting.
- Soil conditions – pipe bursting can be limited by soil type or the presence of rock adjacent to the pipe trench.
- Pipe depth – adequate depth is required to prevent surface heaving.
- Proximity of existing utilities and structures – depending on the method used and soil conditions, pipe bursting can impact nearby facilities.

4.4.2 *Sewer Improvements in Main Collection System*

General. The sewer improvements required for the Main Collection System to convey projected flows and extend service to unsewered areas are summarized in the following paragraphs. The locations of these improvements are shown in Figure 4-2B or Figure 4-2C. The timing of the installations will depend on the actual pace of development.

North Wye Sewer. The developer for Cascade View Estates previously designed a pipe-size increase to 12 inches for the North Wye Sewer by pipe bursting. Our model confirms a 12-inch pipe will have the capacity to carry the projected tributary flows from the north-northeast portion of the Main Collection System.

Current forecasts suggest the North Wye sewer improvement might not be necessary within the next 20 years. However, if developments in the tributary area occur sooner than anticipated, the sewer improvement would need to be implemented within the planning period.

The feasibility of pipe bursting for the North Wye Sewer should be confirmed by a detailed analysis of existing subsurface conditions and pipe material. The designer of the sewer replacement should document in-situ conditions during project development.

Culver Highway Sewer. The model shows a parallel, 10-inch sewer in Marshall Street, Culver Hwy. and 1st Street will be needed to allow all projected flows to be conveyed to the interceptor in “B” Street. A portion of this parallel sewer has previously been designed by a developer.

Since the existing sewer between “G” and “B” Streets is over 40 years old and the timing of future developments is uncertain, the City should give consideration to the replacement of the existing pipe instead of a parallel line. A replacement pipe would need to have a 15-inch diameter to handle all projected flows from existing and future developments.

We recommend the City complete a video inspection of the existing pipe prior to project development to assess its condition and help determine if replacement would be more cost effective than a parallel pipe. If the existing line is in adequate condition, future lining of this pipe may be feasible. Therefore, a parallel sewer and pipe lining at a later date may be the preferred option.

Madras South Town Center Sewer Extension. Due to existing topography, most of the sewer connections at the Madras South Town Center site would flow to the northwest. Therefore, development of the Town Center and the Fairgrounds Estates will require an extension of the sewer system. The model shows an 8-inch pipe that will connect to the Main Collection System at the intersection of Culver Hwy. and Fairgrounds Road (MH M13) will be adequate.

The model assumes WW from the east side of the site will discharge to existing sewers along Highway 26/97. Further investigation will be needed during project development to determine the actual layout of sewer extensions and distribution of WW flows. Current forecasts indicate these projects and the associated sewer extension may occur beyond the 20-year planning period.

Eldorado Estates and The Ridge Extension Sewer. The development of these properties will require a gravity sewer that would extend north and connect to the existing system at MH SH-17. The model shows an 8-inch pipe will be adequate for the proposed developments and the developers have included the sewer extension in their proposed plans. Current forecasts indicate these projects and the associated sewer extension may occur beyond the 20-year planning period.

Willow Creek North Sewer. Proposed developments between Willow Creek and City View Street will require a sewer service extension. Given the sizes and locations of the proposed developments, an 8-inch gravity sewer extending along Willow Creek south to “J” Street would be adequate. Proposed plans submitted by the developers have followed this concept. The alignment of the extension and the need for easements will require coordination between development plans.

Service Extensions to Unsewered. The City plans to extend sewer service to most unsewered areas that have existing residences within the next 20 years (see Table 4-4). At full development, these unsewered areas can be served by 8-inch gravity sewers. The timing of these sewer extensions will depend on the availability of funds.

4.4.3 *Sewer Improvements in Industrial Park Collection System*

North and South Industrial Areas. Modeling results show an 18-inch parallel sewer would be needed to supplement the existing capacity of either the Berg Drive or Demers Drive sewer to handle an average-flow increase of 1.0 MGD. The proximity of the South Area to Demers PS might allow a parallel sewer along Demers Drive to be shorter than a parallel line in Berg Drive.

However, to be conservative, our analysis assumed the Demers Drive parallel sewer would extend south to Adler Street.

East Industrial Area. Modeling results show an 18-inch parallel sewer would be needed to supplement available capacity if the East Area were served by the Demers Drive sewer. Alternatively, a 15-inch parallel sewer would be needed to supplement available capacity in the Cherry Lane sewer. Also, the parallel line along Cherry Lane would be shorter than the Demers Drive alignment. Therefore, it is recommended the City plan to continue serving most of the East Area from Cherry Lane.

Service Extensions to Unsewered Areas. Two 8-inch sewer extensions are planned in the Industrial Park Collection System. One line would extend along Mill Street and the second would extend along Hesse Street and across U.S. Highway 26. These extensions would provide service to a small number of current developments that are currently using onsite disposal systems and support infill development within these parts of the Industrial Park.

Undeveloped Airport Property. Due to topography, potential developments along the northwest side of the airport would require a separate WW collection system with a separate PS and force main. Figure 4-2A illustrates a potential system layout. The actual alignments and sizing of the gravity sewers serving the area would depend on the patterns and service needs of the developments that occur. These sewers would generally be constructed as part of individual developments.

It may be feasible to serve the land immediately northeast of the airport with a gravity sewer extending west along Dogwood Lane. However, if gravity service to the northwest side of the airport is not feasible, another PS would be needed for the northeast area.

4.5 COLLECTION SYSTEM PUMP STATION IMPROVEMENTS

4.5.1 *Pump Stations in Main Collection System*

“B” Street Pump Stations. Based on conversations with operating staff, the addition of influent screening is warranted at the “B” St. North PS due to an increase in plastic fabrics such as flushable wipes that tend to clog pumps. Because this problem is a priority, our estimate of probable costs includes a separate project for the addition of a mechanically-cleaned screen in a new concrete channel. Future NWWTP upgrades and expansions should be coordinated with the installation of screening equipment at the PS.

The condition assessment in Chapter 2 also recommended the City renovate the “B” St. North PS in 11 to 15 years. This renovation would include electrical, instrumentation and controls components that serve both the North and South pumping systems.

The “B” St. South PS is generally in adequate to good condition and should not require a major overhaul. But the PS will need to be expanded when the South WWTP is expanded (see Chapter 5). Due to age and wear, the City will need to replace some other components during the planning period. Consequently, a PS expansion and partial renovation is listed as a capital improvement.

The generator set and associated equipment will be in need of a major overhaul or replacement in 11 to 15 years. Some renovation work on the Generator Building would also likely be due at that time. To be conservative, we have included the replacement of the genset and related work as a capital improvement.

South 97 PS. The City should plan a renovation of this PS in 6 to 10 years. Because of the small size and limited amount of equipment at this PS, we have considered this work to be part of ongoing maintenance and have not included it as a capital improvement.

4.5.2 *Pump Stations in Industrial Park Collection System*

Golf Course PS. The Golf Course PS will need to be replaced when the Willowbrook subdivision development proceeds. But the timing of that development cannot be predicted. Therefore, as noted in Chapter 2 (Subsection 2.3.3), the City should plan to replace the existing PS within 10 years due to its age and condition.

Demers PS and Force Main. The existing capacity of the Demers PS is 0.42 MGD and the net available capacity for future developments is estimated to be 0.14 MGD after full development of Willowbrook. Given the condition of this PS, the City should plan to renovate much of the facility in 3 to 5 years. Due to the age of the overall facility, the City should plan to fully replace the entire PS in 16 to 20 years, or sooner if a capacity expansion is needed to serve industrial development. The replacement PS would be designed with space for an additional pump, and also for larger pumps, to accommodate phased expansions as development necessitates.

Future expansions of the Demers PS will require an increase in the size of the PS force main to accommodate the higher pumping rate. We have included a future replacement of the Demers force-main with an 8-inch pipe as a recommended PS improvement project. A second 8-inch pipe could be installed parallel to this replacement pipe when a further PS expansion is required.

The timing of the future PS expansions, force main replacement and second, parallel force main would depend on the pace of industrial development and their actual WW contributions.

Northwest Airport Area PS and Force Main. As mentioned in Section 4.4.3 and illustrated in Figure 4-2A, a separate PS and force main would be needed to extend service to land along the northwest side of the airport. The actual sizing and location of the PS would depend on the service needs of the future users who locate to the area. For industrial site readiness planning we have assumed the PS would have the same capacity as the future Demers PS replacement.

A more detailed study into the most cost effective way to extend sewer service to undeveloped airport property will be required during the early stages of site development. A key consideration in siting the PS will be the shallow depth to rock in the airport area.

It may be feasible to serve the land immediately northeast of the airport with a gravity sewer extending to the potential PS site shown in Figure 4-2A. However, if gravity service to the northwest PS site is not feasible, another PS would be needed for the northeast area.

4.6 ESTIMATES OF PROBABLE PROJECT COSTS

Tables 4-7 and 4-8 present planning-level estimates of probable costs for the sewer system and PS improvements, respectively. These estimates are current to December 2016 (Engineering News-Record Construction Cost Index = 10,530). Chapter 8 provides additional information on the basis of these estimates. *The capital improvements plan presented in Chapter 8 lists updated probable costs that are current to December 2017.*

Table 4-7 Estimates of Probable Costs for Sewer Improvements (December 2016 Dollars)				
Project Description	Pipe Size (inches)	Approx. Length (ft.)	Probable Construction Cost ⁽¹⁾	Probable Project Cost ⁽²⁾
A. Industrial Park Collection System Sewer Extensions				
1. Hess Street Sewer	8	1,670	\$250,000	\$338,000
2. Mill Street Sewer	8	1,430	\$215,000	\$290,000
Subtotal for Industrial Park Sewer Extensions			\$465,000	\$628,000
B. Potential Alternative Sewers for Major Users ⁽³⁾				
1. East Area Parallel Sewer – Cherry Ln.	15	4,800	\$1,200,000	\$1,620,000
2. South Area Parallel Sewer – Demers Dr.	18	2,400	\$720,000	\$972,000
3. North Area Parallel Sewer – Berg Drive	18	2,240	\$673,000	\$909,000
C. Main Collection System Sewer Improvements				
1. North Wye Sewer Replacement (By Pipe Bursting)	12	2,060	\$631,000	\$852,000
2. Culver Hwy. Parallel Sewer	10	3,100	\$543,000	\$733,000
3. Bel Air/Herzberg Heights ⁽⁴⁾	8	5,100	\$829,000	\$1,120,000
4. Jefferson Street/North Unit ⁽⁴⁾	8	2,900	\$399,000	\$539,000
5. Mountain View/Sky Ridge Sewer Extensions ⁽⁴⁾	8	4,500	\$844,000	\$1,139,000
6. Juniper Heights Sewer Extension ⁽⁴⁾	8	5,350	\$1,003,000	\$1,354,000
7. Willow Creek North Sewer Extension	8	2,500	\$406,000	\$548,000
8. Madras South Town Center Sewer Extension ⁽⁵⁾	8	2,420	\$454,000	\$613,000
9. Eldorado Estates/The Ridge Sewer Extension ⁽⁵⁾	8	1,800	\$338,000	\$456,000
Subtotal for Main Collection System Sewers			\$5,447,000	\$7,354,000

(1) Construction cost includes a 25% construction contingency.

(2) Project cost includes a 35% allowance for nonconstruction costs (engineering and administrative).

(3) Potential improvements shown for planning industrial site readiness. Timing dependent on developments for major users. Required sewer projects for major user(s) must be reevaluated based on actual development proposal(s).

(4) Projects for unsewered areas with current development.

(5) Projects would not be required within 20 years based on current population projections and anticipated development timing. Listed for information in case developments are proposed within planning period.

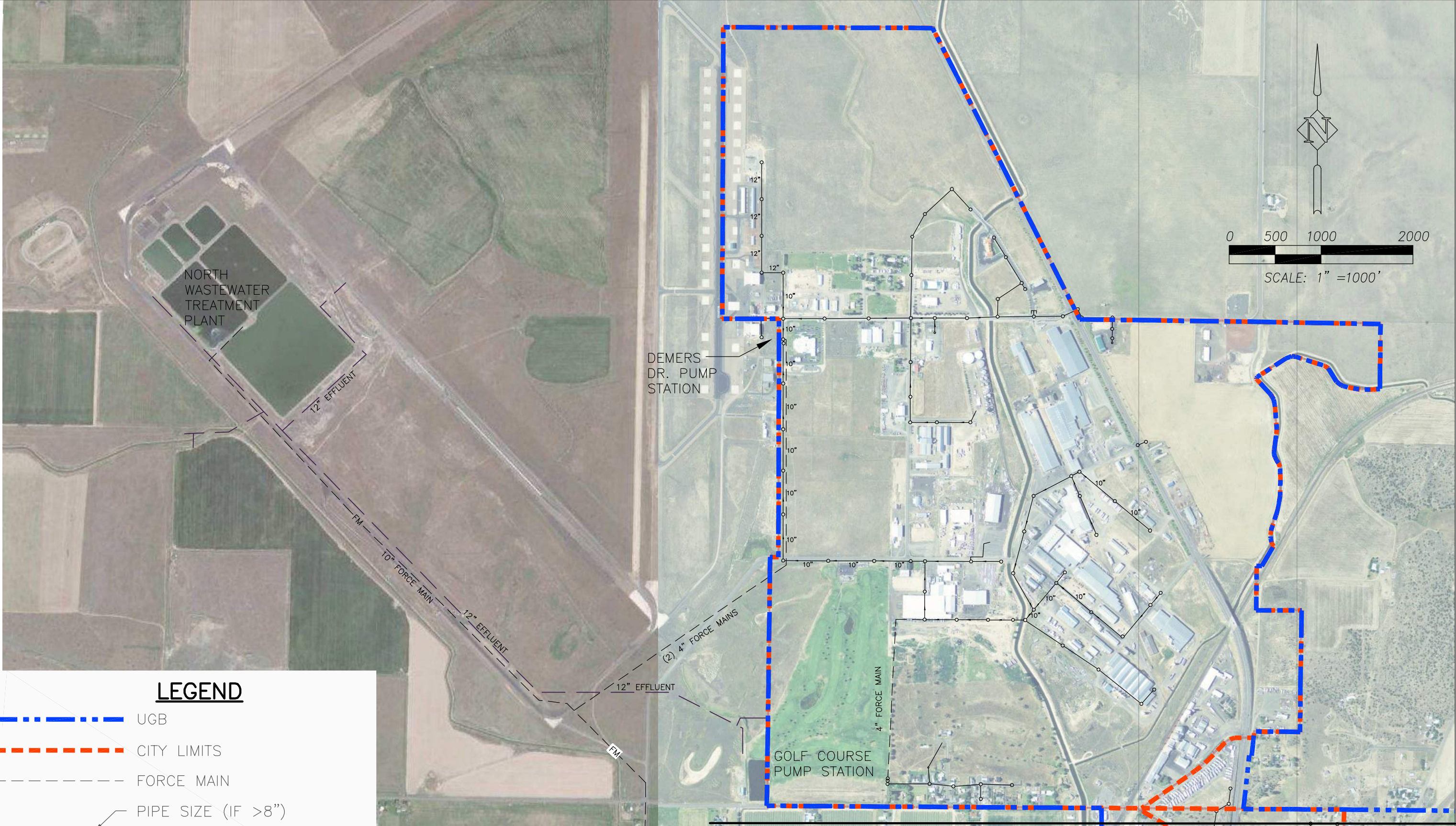
Table 4-8 Estimates of Probable Costs for Pump Station Improvements (December 2016 Dollars)				
Project Description	Current Capacity (MGD)	Planned Capacity (MGD)	Probable Construction Cost ⁽¹⁾	Probable Project Cost ⁽²⁾
A. Industrial Park Collection System (North Area)				
1. Golf Course PS Replacement – Expanded Capacity	N/A	0.21	\$350,000	\$475,000
2. Demers PS Renovation – Major maintenance and repair	0.42	--	-----	\$200,000
3. Demers PS Replacement – Expanded Capacity	0.42	1.33	\$1,285,000	\$1,735,000
4. Demers Force Main Replacement – 8” Pipe ⁽³⁾	--	1.33	\$1,137,000	\$1,535,000
5. Potential PS & Force Main for NW Airport Area ⁽³⁾	--	1.33	\$1,680,000	\$2,270,000
Subtotal – Pump Stations in North Area				\$6,215,000
B. Main Collection System Pump Stations				
1. Influent Screen at “B” Street North PS – No expansion	1.11	1.11	\$334,000	\$450,000
2. Renovation of “B” Street North PS – No expansion	1.11	1.11	\$438,000	\$590,000
3. “B” Street South PS Expansion & Renovation	1.33	2.66	\$525,000	\$710,000
4. “B” Street North & South Generator Set Replacement	N/A	N/A	\$175,000	\$235,000
5. South U.S. 97 PS Renovation – No expansion	0.06	0.06	\$44,000	\$60,000
Subtotal – Pump Stations in Main Collection System				\$2,045,000
Total – Pump Stations & Force Mains				\$8,260,000

(1) Construction cost includes a 25% construction contingency.

(2) Project cost includes a 35% allowance for nonconstruction costs (engineering and administrative).

(3) Potential improvement shown for planning industrial site readiness. Timing dependent on developments for major users. Required pump station and force main projects for major user(s) must be reevaluated based on actual development proposal(s).

P:\MAD (City of Madras)\MAD-05 (WW Master Plan Update)\MAD-05 - DWGS\SHEETS\Chap. 4\Figure with Existing Collection System.dwg



LEGEND

- UGB
- CITY LIMITS
- FORCE MAIN
- 10" PIPE SIZE (IF >8")
- 10" GRAVITY SEWER
- MANHOLE
- SIPHON (NO LATERALS)
- EFFLUENT IRRIGATION PIPE

MATCHLINE – SEE FIGURE 4-1B



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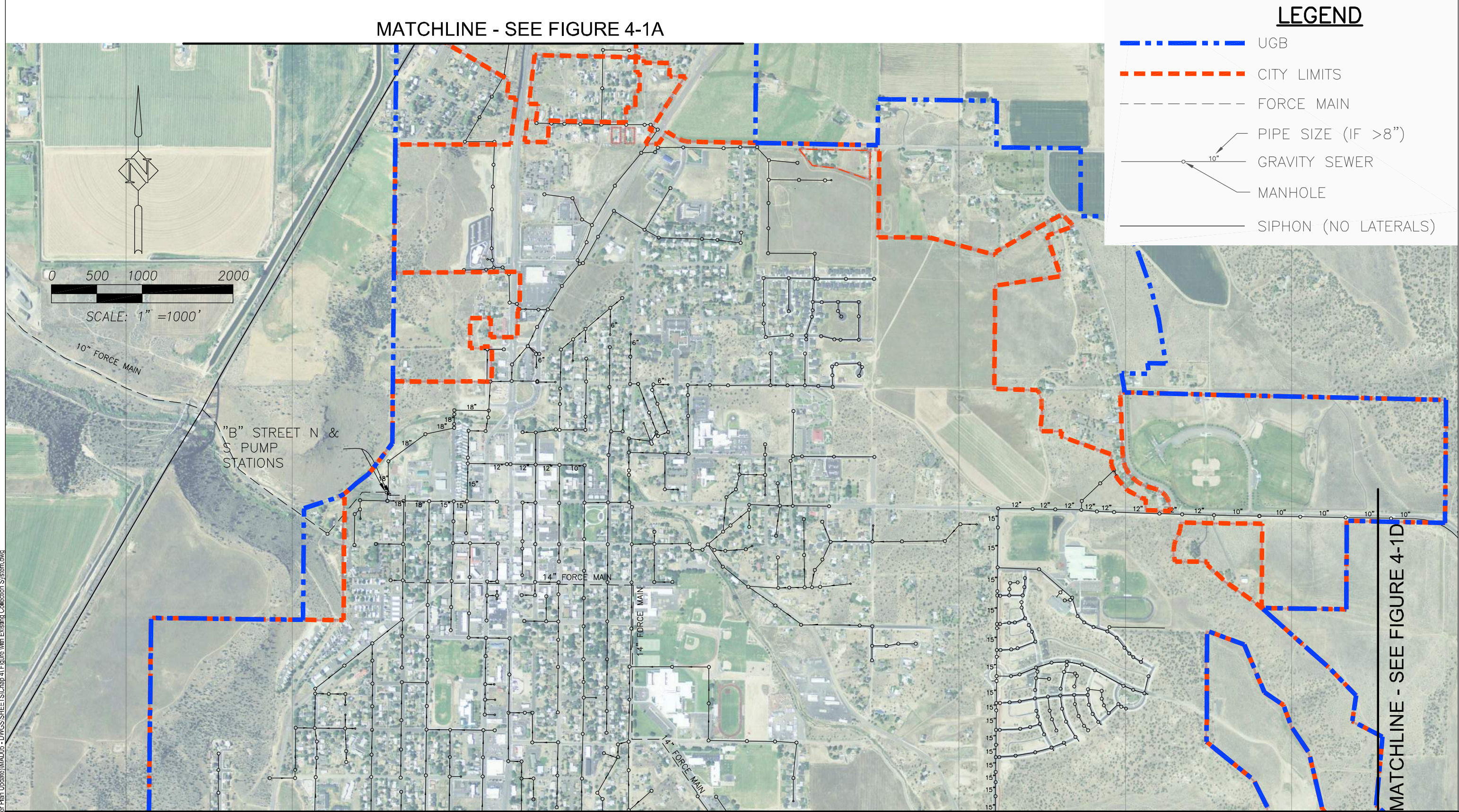
EXISTING WW COLLECTION SYSTEMS - NORTH AREA
WASTEWATER MASTER PLAN UPDATE
CITY OF MADRAS, OR

SHEET NO.

FIG 4-1A

JOB NO.

MAD-05



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EXISTING WW COLLECTION SYSTEMS - CENTRAL AREA
WASTE WATER MASTER PLAN UPDATE
CITY OF MADRAS, OR

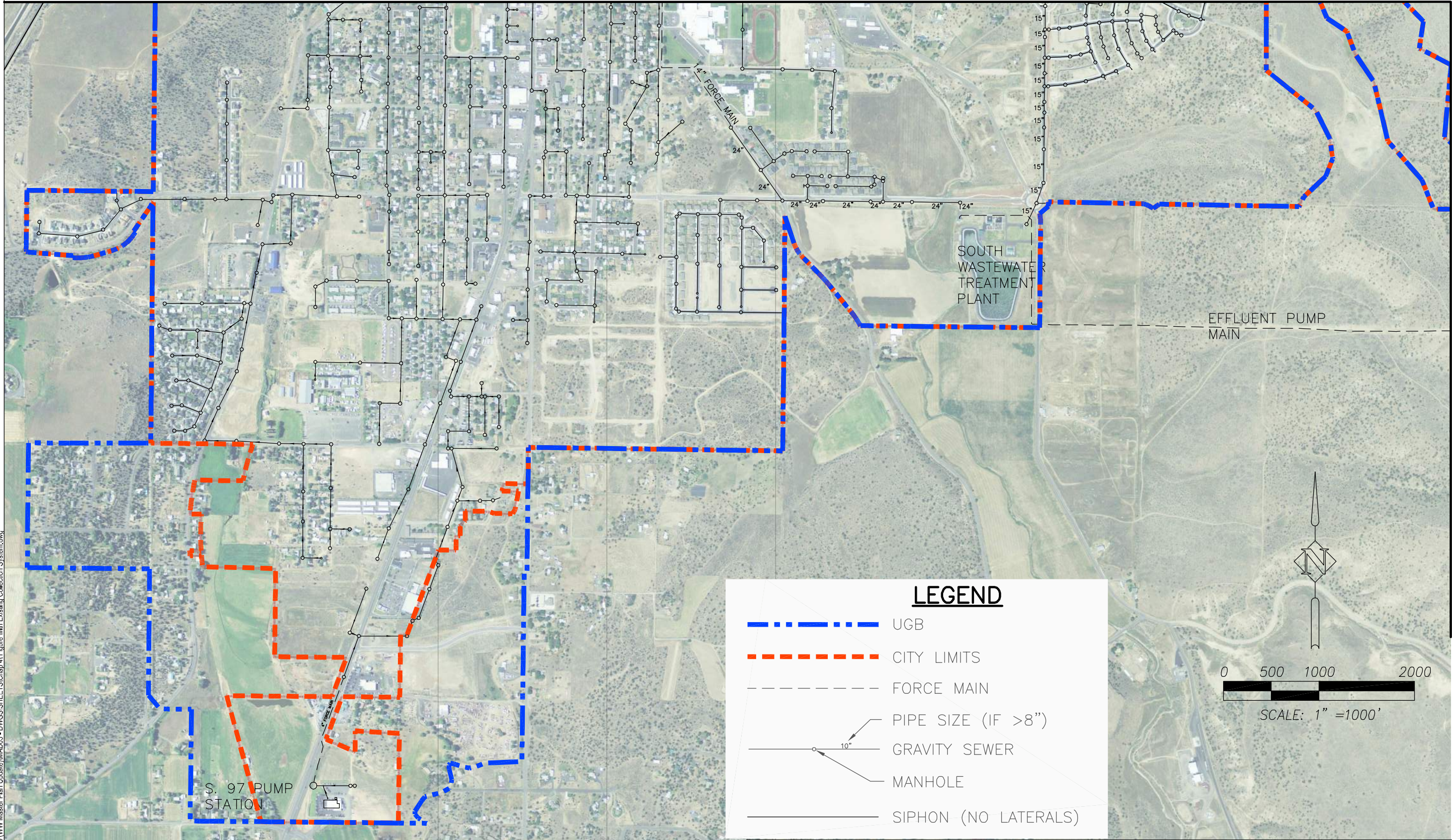
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FIG 4-1B

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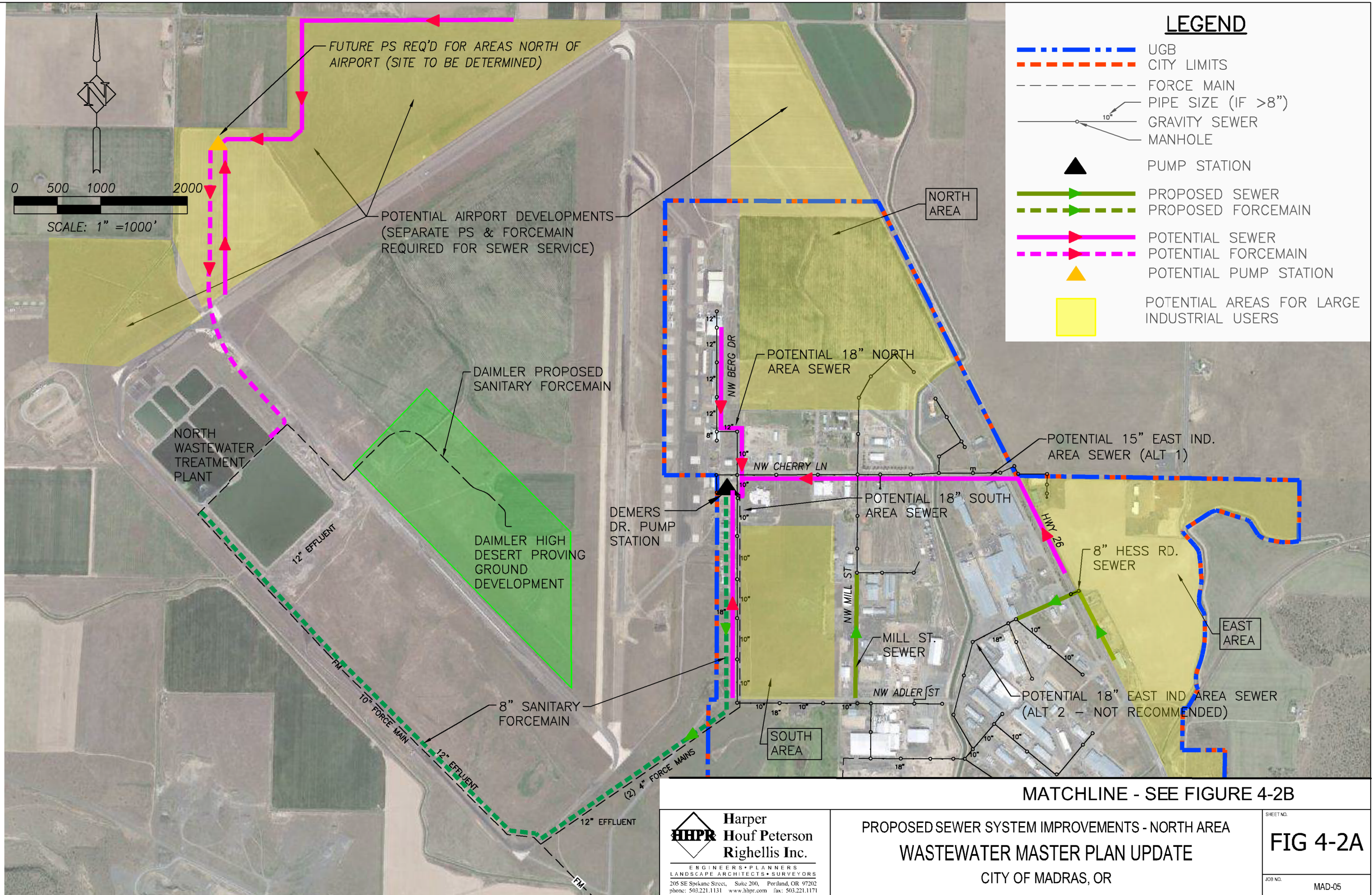
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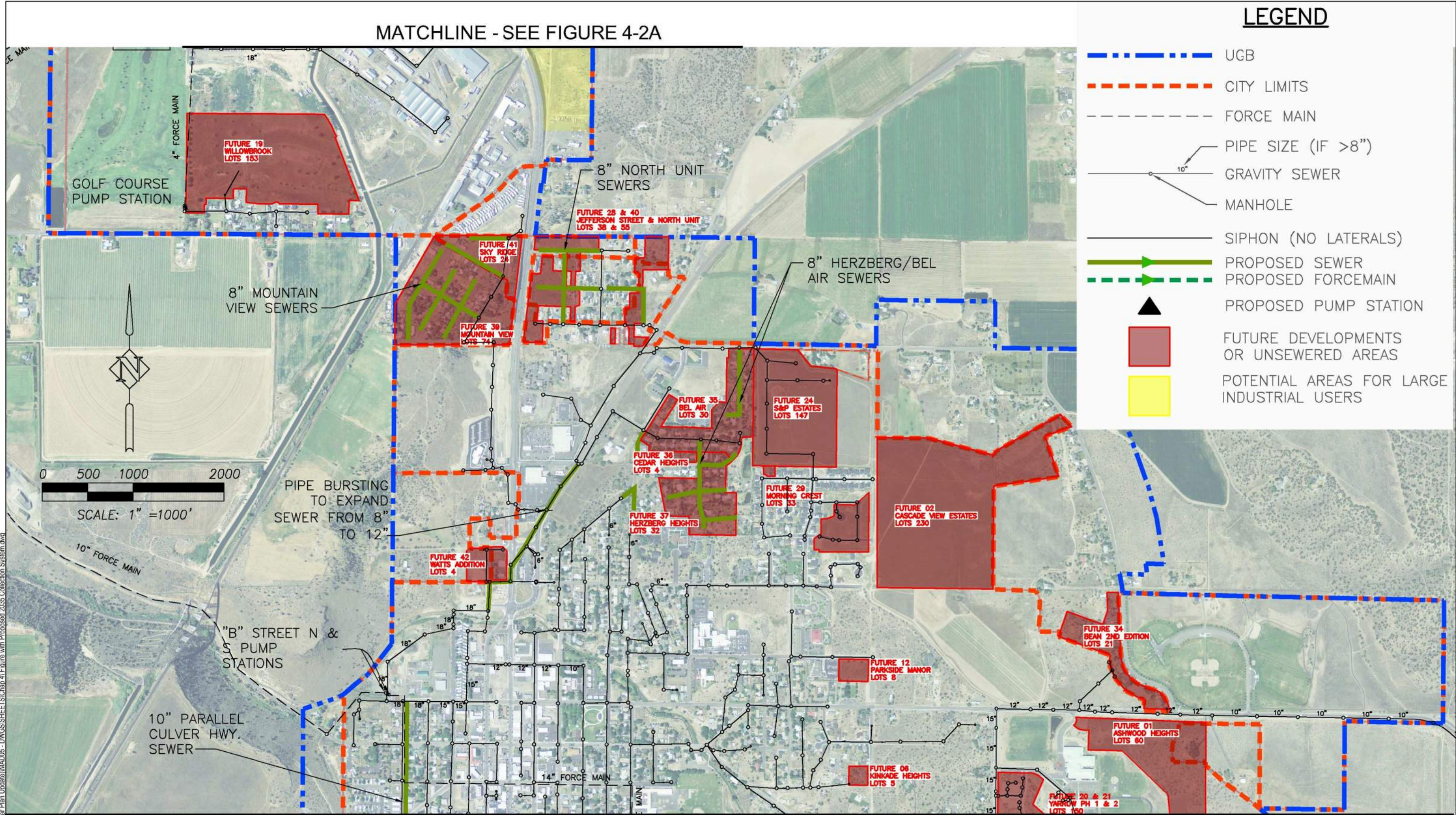
MATCHLINE - SEE FIGURE 4-1B



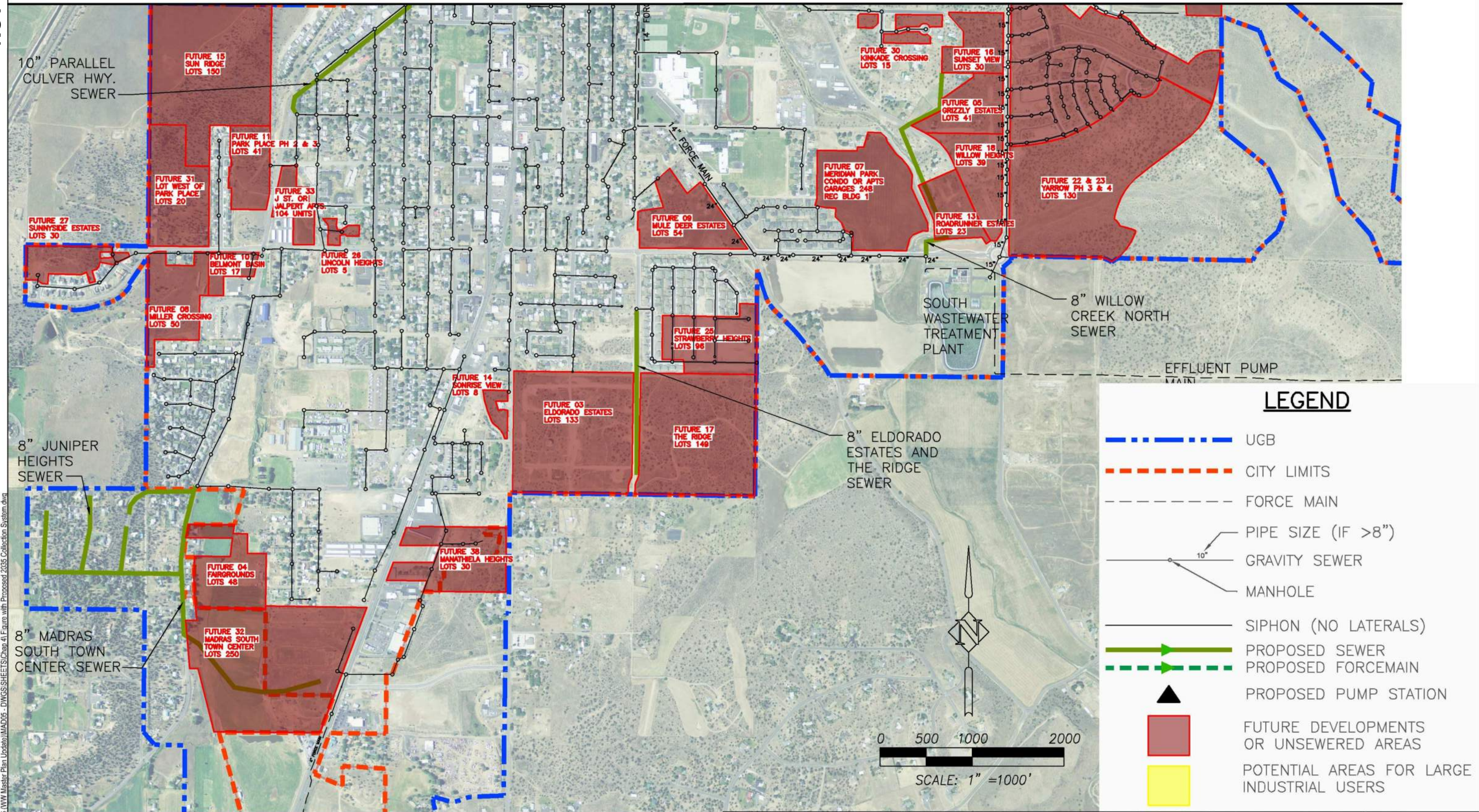
MATCHLINE - SEE FIGURE 4-1D







MATCHLINE - SEE FIGURE 4-2B



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CHAPTER 5

TREATMENT ALTERNATIVES DEVELOPMENT AND SCREENING

5.1 INTRODUCTION

This chapter discusses wastewater (WW) treatment alternatives and identifies those alternatives that are recommended for further consideration. The main elements of this chapter are listed below.

1. Background Information – establishes the framework for developing treatment alternatives.
2. Basis of Alternatives Development – identifies the required Wastewater Treatment Plant (WWTP) capacity and treatment level.
3. Overall Treatment Scenarios – identifies and screens overall treatment scenarios based on projected service needs.
4. Treatment Process Alternatives – identify liquid-stream and solids-stream treatment process alternatives that have the potential to cost effectively meet the City's needs.
5. Identify alternatives screening criteria and present the results of the screening effort.

5.2 BACKGROUND FOR ALTERNATIVES DEVELOPMENT

5.2.1 *Existing Liquid-Stream Treatment Facilities*

NWWTP. A focus of this WWMP is to support industrial site readiness by evaluating alternatives for serving Industrial Park developments, including alternatives for upgrading and expanding the existing NWWTP. The facility offers the following key advantages.

- The lagoon system provides an inexpensive biological treatment process.
- The facility is located close to the existing Industrial Park as well as irrigable land for effluent recycling.

Other key considerations in developing alternatives for the NWWTP are described below.

- The NWWTP requires significant corrective work to keep the facility operating reliably for another 20 years and some of this work should be completed within 5 years.
- Land-use guidance from the Federal Aviation Administration (FAA) identifies WWTPs as potential wildlife attractants. This guidance recommends 10,000 feet of separation between wildlife attractants and airports serving turbine-powered planes. The NWWTP is well within this separation distance.

Wildlife biologists from the U.S. Department of Agriculture (USDA) have conducted site visits to the NWWTP in the past on behalf of the FAA. Correspondence from the USDA has recommended the airport implement mitigation measures to remove wildlife hazards at the site.

- The shallow depth to underlying rock at the NWWTP site impacts the cost of excavations. To limit rock excavation, lagoons and basins must be shallow and/or built mostly above grade.

SWWTP. The 1996 WWMP established the cost effectiveness of the SWWTP to provide service for most of the City. The facility offers the following main advantages.

- The plant has significant useful service life remaining and should provide reliable service for 20 years with routine maintenance, scheduled replacements, and other planned corrective work.
- The facilities are designed to accommodate another expansion with common-wall construction.
- The site accommodates gravity flow from sewers in the east and southeast parts of the City.

The only disadvantage to keeping the SWWTP in operation is the relatively higher operations and maintenance (O&M) cost of a mechanical plant relative to a plant using a lagoon system such as the existing NWWTP. However, the offset to the higher O&M cost is that far less land is required for the mechanical plant than for a lagoon system.

5.2.2 WWTP Effluent Uses

Recycling. Chapter 1 outlines the regulations that govern effluent recycling and Chapter 2 briefly describes the City's current recycling practices. As described in Chapter 1, the Department of Environmental Quality (DEQ) generally encourages effluent recycling as a beneficial use. The DEQ considers the treated effluent as a resource because it supports water conservation and/or a reduction in fertilizer use.

The City began recycling treated effluent as irrigation water after the NWWTP was constructed and never obtained an operating permit for a surface-water discharge. The 1996 WWMP evaluated a discharge to Willow Creek as an alternative for a new WWTP. However, that report recommended effluent recycling through irrigation for the proposed SWWTP, as well as the continuation of this practice at the NWWTP.

Surface-Water Discharge. At the time we prepared the WWMP scope, a key assumption was that the City would continue effluent recycling throughout the planning period. Therefore, the WWMP scope only included an evaluation of treatment alternatives for effluent recycling and no detailed study was conducted of alternatives for other effluent disposal methods.

However, the cost of pond(s) to store effluent between growing seasons and the costs of irrigation systems for recycling provide an impetus to consider other options. DEQ's recent approval of a year-round discharge from constructed wetlands in the City of Prineville has also raised questions regarding the potential application of a similar approach in Madras.

Because of these cost considerations, we briefly examined the main issues relating to a surface-water discharge as part of this study. Our examination included a review of current WQS, consideration of discharge options, a brief analysis of WW treatment requirements, and initial communications with DEQ (see Appendix J). The following paragraphs summarize key issues our preliminary investigation identified regarding the pursuit of a permit for a surface-water discharge.

Regulation of Discharges. As described in Chapter 1, DEQ regulates surface water discharges in Oregon to maintain compliance with published water quality standards (WQS) for receiving water bodies. Since Willow Creek and the Deschutes River are the only potential receiving streams near Madras, the WQS that pertain to the Deschutes Basin would be used to regulate a discharge. Chapter 1 lists the key WQS governing treatment requirements for such a discharge.

Key Water Quality Considerations.

1. DEQ lists Willow Creek, Simtustus Lake and the lower Deschutes as water-quality impaired because they do not meet WQS for the following parameters.
 - Willow Creek is listed as being water quality impaired for water temperature.
 - The Deschutes River, above and below Pelton Dam is listed as being water quality impaired for dissolved oxygen and pH level.
 - Lake Simtustus is listed as water quality impaired for dissolved oxygen and chlorophyll-a (phytoplankton growth).

Because these water bodies are water-quality impaired, the DEQ must issue Total Maximum Daily Loads (TMDLs) for the parameters that do not meet WQS. The TMDLs would be used to determine discharge limitations from a permitted facility. To date DEQ has not yet published proposed TMDLs for the Deschutes Basin and there is no schedule for their development. Input from DEQ suggests issuing these TMDLs is not a priority.

2. In general, Lake Simtustus is characterized as eutrophic and nutrient loads (nitrogen and phosphorus) are a concern. Nitrogen in the form of nitrates is also a concern in the Willow Creek basin. These issues could impact permit limitations for nitrogen and phosphorus on any Willow Creek discharge.
3. Willow Creek commonly runs dry from July to December and stream flows can be very low during winter and spring.
 - The WQS include a criterion for dilution of organic material that effectively requires some background flow for the approval of a discharge. The dilution criterion requires the concentration of biochemical oxygen demand (BOD) in a discharge to be no greater than a dilution factor.
 - With no background flow present, the dilution factor becomes zero and the discharge of measurable BOD levels is effectively prohibited. Therefore, the absence of background flows in Willow Creek might serve as the basis for an outright denial of a discharge permit, at least for most of the year.
 - Even during winter, low flows are common in Willow Creek which means there would be very little mixing and dilution of any WWTP discharge. This lack of dilution would result in stricter discharge limitations and higher treatment costs.
4. The published WQS include an antidegradation policy (OAR 240-041-0004) that is intended to prevent the further degradation of water quality from new or increased pollution sources. This policy could be used as the basis for denying a discharge permit, unless the City could provide an analysis showing a discharge to Willow Creek would not degrade water quality. The City might need to request an exemption from the antidegradation WQS and submit the analysis as supporting data.

Discharge Options. Approved (permitted) discharges to a surface water body can be either seasonal or for the entire year and also either direct or indirect. These options are briefly described below.

1. Seasonal Discharge. Discharges from November 1 through March 31 are sometimes permitted in Oregon because stream flows are higher during these months and WQS are less stringent. Higher stream flows provide more dilution for the effluent and, when combined with less stringent WQS, this can result in less-costly treatment requirements. If Madras were issued a seasonal discharge permit, all effluent produced during spring, summer and part of autumn would need to be recycled. Also, some effluent storage may be required.
2. Year Round Discharge. A discharge all year typically requires a receiving stream that has flow all the time for mixing and dilution. Even with year-round flow, treatment requirements are commonly more stringent in summer due to lower stream flows. An indirect discharge may be regulated differently than a direct discharge, as discussed below.
3. Direct Willow Creek Discharge. A direct discharge to Willow Creek would be subject to strict WQS and might be denied. If permitted, a direct discharge would probably require advanced treatment that would increase costs relative to current treatment requirements.

4. Indirect Willow Creek Discharge

- This approach involves releasing treated WW into intermediate facilities such as constructed wetlands. The effluent then seeps into the ground and enters a water body indirectly. The intermediate facilities are designed to provide additional treatment.
- DEQ has stated that their guidance for indirect discharges is currently in flux. As a result, there is uncertainty as to the regulatory requirements for this type of discharge.
- Due to the complexity of treatment mechanisms in wetlands, DEQ would probably require detailed analyses into the fate of pollutants through these pathways.

5. Outfall Pipeline for Direct Deschutes River Discharge

- The relatively-high flows of the Deschutes River would provide good dilution and mixing for a direct discharge to that receiving stream. As a result, such a discharge may be more feasible in terms of treatment requirements for meeting WQS, particularly downstream of Pelton Dam.
- A discharge to the Deschutes River from the NWWTP would require a gravity pipeline of more than 4 miles extending to Lake Simtustus or further south to the river just below the Pelton Dam. A discharge from the SWWTP to the Deschutes River would likely require effluent pumping across town, through a force main, and then gravity flow through a pipeline to Lake Simtustus or the river.
- Construction of an outfall pipe to the river would face major technical, environmental, financial and political obstacles. A discharge to the Deschutes River would also probably not qualify as a beneficial use.

Summary of Options for Effluent Uses. Incentives for the City to continue effluent recycling are as follows:

- a successful track record with the practice;
- the local availability of irrigable farmland;
- the recommendations of the 1996 WWMP;
- a favorable regulatory climate; and
- less uncertainty regarding future rules affecting effluent recycling.

Regulations for effluent recycling are generally less strict and subject to fewer revisions than the regulations that govern surface-water discharges. Chapter 7 provides projections for the effluent storage capacity and farmland needed to continue recycling all effluent for the next 20 years.

The City may want to further investigate the feasibility of a discharge to Willow Creek and pursue a permit from the DEQ before undertaking any major WWTP upgrades or expansions. However, the City would face significant regulatory hurdles in pursuing a discharge permit. A direct discharge may not gain approval or be feasible due to water quality issues and technological constraints. The DEQ may not permit an indirect discharge for similar reasons or only permit an indirect discharge during the winter season, which means effluent recycling during the spring and summer would still be necessary.

The main benefit to a creek discharge in winter would be to reduce the amount of effluent storage capacity needed when recycling cannot occur. To evaluate the cost effectiveness of this approach, the City would need to compare the savings of reduced storage capacity to the added costs associated with more stringent treatment requirements.

Feasibility Study for Surface-Water Discharge. To investigate a surface-water discharge, the City would need to conduct a feasibility study that would generally include the following elements.

1. Perform an initial evaluation of the impacts that WQS and related issues would have on a discharge-permit application.
2. Develop concept-level discharge options and screen out options that do not appear feasible based on input from DEQ and site-specific factors.
3. Investigate the permit limitations that would be established for remaining discharge options and the resulting treatment requirements.
4. Consider the potential differences in permit limitations, treatment requirements for seasonal and year-round discharges.
5. Evaluate the potential benefits of an indirect discharge through constructed wetlands.
6. Identify the probable scope of the water quality modeling and study that would be required to address the fate of potential pollutants and show no adverse water quality impacts would result.
7. Provide concept-level estimates of probable costs for discharge options that appear feasible.

5.2.3 *Biosolids Handling*

The 1996 WWMP evaluated alternatives for biosolids handling and recommended the construction of aerobic digesters at the SWWTP and sludge drying beds at the NWWTP. Both these facilities were constructed and subsequently expanded. The aerobic digesters stabilize the waste activated sludge (WAS) produced in the sequencing batch reactor (SBR) process and the City then hauls the stabilized biosolids by tank truck to the drying beds. Solids removed in the dissolved air flotation (DAF) clarifier at the NWWTP are pumped to a sludge storage pond where the material dries during warm-weather months.

The City spreads the dried solids as Class B biosolids on City-owned farmland according to the City's Biosolids Management Plan. The aerobic digesters at the SWWTP are designed to accommodate future expansion along with the SBR process.

The existing digesters and drying beds have adequate capacity to handle the biosolids currently being produced and these facilities are generally in good condition. Continued operation of these facilities and land application of dried Class B biosolids should remain cost effective as long as the existing WWTPs remain in service and regulations do not change. However, the capacity of the drying beds is barely adequate to handle current volumes of biosolids being hauled from the SWWTP. Therefore, either the drying beds will need to be expanded to accommodate flow increases, or the water content of the biosolids will need to be reduced before hauling.

5.3 **BASIS OF ALTERNATIVES DEVELOPMENT**

5.3.1 *Required Treatment Capacity*

Table 5-1 (following page) lists the maximum-monthly, influent flow projections identified from the basis of planning presented in Chapter 3. This maximum-monthly flow is typically used to establish the total ADF capacity required for secondary WW treatment. The net available ADF capacity listed in Table 5-1 is the difference between the projected maximum-monthly flows and the existing combined ADF flow capacity of 1.04 MGD. Maximum-month flows are projected to reach existing WWTP capacity in 10 to 12 years. Table 5-1 also shows the projected peak-hourly flow used to establish the required overall WWTP hydraulic capacity. The peak-hourly flows are based on an estimated peaking factor of 2.67.

Table 5-1 Wastewater Treatment Capacity Requirements			
Year	Projected Total Flows (MGD) ⁽¹⁾		Net Available ADF Capacity ⁽²⁾
	Maximum Monthly	Peak Hourly	
2020	0.89	2.43	0.15
2025	1.03	2.75	0.01
2030	1.09	2.91	– 0.05
2035	1.14	3.04	– 0.10
2065 (build-out)	1.41	3.76	– 0.37

(1) Flow projections assume full occupancy of the DRCI and nonresidential flows will increase in proportion with projected growth in residential flows.

(2) Based on the existing total ADF capacity of 1.04 MGD.

5.3.2 Required Treatment Level and Storage

Liquid Stream. As long as the City continues the practice of effluent recycling, the required treatment level depends on how the treated effluent is recycled. These minimum treatment requirements are established by the DEQ regulations for recycling and are divided into effluent Classes A through D (refer to Table 1-1). Current opportunities for irrigation readily accommodate recycling with Class B effluent and the additional treatment required to produce Class A effluent is not necessary. Therefore, this study evaluates treatment alternatives for continuing the practice of recycling Class B effluent.

Both WWTPs readily produce Class B effluent through the operation of a biological process, solids removal equipment and a disinfection system. This approach provides good effluent quality, which limits chlorine demand for disinfection and lowers the potential for fouling of irrigation components. Class B effluent also carries less-strict conditions of use for irrigation practices than Class C and Class D effluent.

As discussed in Subsection 5.2.2, there is uncertainty regarding the treatment requirements for a surface-water discharge and further study is required to address options for a discharge. Such a study is beyond the scope of this WWMP Update.

Effluent Storage. Water recycling for irrigation purposes in Madras can occur only during the growing season, usually from early-to-mid April until early-to-mid October. As a result, without a seasonal discharge to Willow Creek, effluent must be stored for 6 to 7 months annually. The effluent storage capacity needed for each 0.5-MGD of treatment capacity would require a gross area of about 30 acres, assuming one pond with a water depth of 10 feet when full. Chapter 7 provides further information on the requirements for effluent storage.

The alternatives developed herein would all require effectively the same effluent storage capacity to address this seasonal limitation for recycling. Therefore, each alternative would include the same space requirements and costs for effluent storage capacity. Since effluent storage costs are equal for all alternatives, they are not included in the cost comparison.

Solids Handling. Given the local availability of farmland and the conditions of existing biosolids handling facilities, the continued practice of land applying Class B biosolids is the best option at this time. Therefore, biosolids handling alternatives were developed based on current treatment requirements to produce Class B biosolids.

The FAA would need to agree that expansions to the sludge drying beds would be compatible with airport operations and mitigation measures would likely be required to deter wildlife attraction.

5.3.3 Summary of Projected Treatment Requirements

Table 5-2 summarizes the influent WW characteristics that the WWTPs must be designed to treat to meet the projected 20-year needs. These characteristics were used to develop preliminary process sizing for the alternatives evaluation.

Table 5-2 Influent Wastewater Characteristics	
WW Parameter	Value
A. Flow Rates ⁽¹⁾	
Average Daily Flow	0.99 MGD
Maximum Monthly Flow	1.14 MGD
Peak Hourly Flow	3.04 MGD
B. Average BOD₅ & TSS Concentrations	
Municipal Flows (to SWWTP)	250 mg/L
DRCI Flows (to SWWTP)	750 mg/L
Combined Industrial & Municipal Flows to NWWTP	300 mg/L
C. Influent Temperature	
Average Winter Design	10 deg. C

(1) Includes 0.23 MGD design flow from DRCI

5.4 OVERALL TREATMENT SCENARIOS

5.4.1 General Descriptions

Technically-feasible WW treatment scenarios the City could potentially implement over the next 20 years are summarized in Table 5-3 and evaluated in the following paragraphs.

Table 5-3 General Treatment Scenarios	
Scenario One:	No WWTP expansions. Renovate, maintain and operate both WWTPs to provide the current 1.04 MGD capacity for the planning period.
Scenario Two:	Abandon both existing plants. Construct one or two new plants to treat all flows at an existing site or alternate sites.
Scenario Three:	Abandon one WWTP. Expand the other to treat all flows.
Scenario Four:	Maintain one WWTP at its current capacity and provide the necessary repairs. Replace or expand the other WWTP to treat the remaining flows.
Scenario Five:	Implement modifications to both WWTPs to expand the capacity of each.

5.4.2 *Evaluation and Initial Screening of Scenarios*

Scenario One – No-Expansion. According to projections, the City will begin to experience a small, capacity shortfall in about 10 years. Since the City must plan to treat all projected WW flows, the only way to avoid planning for a capacity expansion would be to implement water conservation measures and/or reduce I/I rates.

Current estimates of per capita WW production indicate there is little potential for water-use reductions through conservation. Similarly, influent flow records show I/I rates are not significant. As a result, the City should plan for a treatment capacity expansion to meet projected needs and no further evaluation of this scenario is warranted.

Scenarios Two and Three – Abandonment of One WWTP or Both WWTPs. No scenarios that involve abandoning one or both of the WWTPs have been evaluated for this study due to the following considerations.

- **NWWTP.** It is important for the City to plan for increased WW flows from Industrial-Park developments as part of maintaining industrial site readiness. Because of this factor, maintaining a WWTP near the Industrial Park provides a key benefit.

Service needs can vary widely for industrial users and planning efforts for site readiness need to incorporate flexibility by accommodating phased expansions. Since the existing NWWTP is located near the Industrial Park, this site reduces the technical complexities and costs of implementing a phased plan. Therefore, this study has only evaluated alternatives that continue providing treatment at the NWWTP site.

- **SWWTP.** The SWWTP is in fair-to-good condition and is strategically located to serve projected growth in the east and southeast parts of the City. As a result, all alternatives developed for this study include maintaining this plant in service for the next 20 years.

Scenarios Four and Five – Expand Either or Both WWTPs. Expansions to either or both plants are technically feasible and potentially cost effective. Therefore, we have developed alternatives incorporating these scenarios. Key factors in developing the treatment alternatives include:

- the small size of the projected capacity shortfall;
- the need for the SWWTP to treat all gravity flows entering the plant from the east and southeast parts of the City, including the DRCI;
- the need for near-term renovations at the existing NWWTP to maintain current capacity;
- the amount of land needed for seasonal effluent storage and effluent recycling by irrigation;
- the importance of addressing Industrial Park site readiness; and
- impacts on the “B” Street pump stations.

All scenarios we evaluated include an expansion of the SWWTP because of the flexibility this would provide. Expanding the SWWTP capacity would free up capacity at the NWWTP to treat flows from North-Area developments and also offer greater flexibility for providing service to the entire Main Collection System, including gravity service to east-side developments.

5.4.3 Summary of Overall Scenario Screening Results

The remaining overall scenarios used to develop treatment alternatives selected for further evaluation are described in Table 5-4. The two scenarios maintain both WWTPs in service and allow for future expansions to the NWWTP as needed to serve developments in the North Area.

Table 5-4 Remaining Overall Treatment Scenarios	
Option A – Expand SWWTP to Treat Projected Flows; Dedicate NWWTP to Serve North Area	
1. Phased improvements to the SWWTP (see Figure 5-1). • Phase 1. Modify the plant to provide a small increase in capacity to 0.65-MGD and renovate the existing facilities as needed to maintain reliable service. • Phase 2. If necessary, construct the additional 0.5-MGD treatment module for which the SWWTP is designed to provide a total 1.14-MGD ADF capacity. This second phase would free up capacity at the NWWTP to serve industrial developments. 2. Renovate, retrofit, or replace the existing NWWTP to maintain the 0.5-MGD capacity. After SWWTP expansions, the NWWTP would be dedicated to serving the North Area and would be expanded as necessary to serve Industrial Park developments.	
Option B – Expand SWWTP; Use Both WWTPs to Treat Projected 20-Year Flows	
1. Construct the additional 0.5-MGD treatment module for which the SWWTP is designed to provide a 1.04-MGD ADF capacity at that plant. 2. Renovate/retrofit/replace the NWWTP to maintain the 0.5-MGD capacity. Operate the plant to treat projected WW flows above the SWWTP capacity and additional flows from the North Area (expand the plant as needed to serve Industrial Park developments).	

We propose Option A as the primary option because the Phase 1 SWWTP expansion would provide an overall treatment capacity that meets the 20-year flow projections. Under current projections, the Phase 2 SWWTP expansion would free-up the entire capacity of the NWWTP to serve the North Area. This second expansion would only be recommended if additional capacity is needed above current projections and expansions to the NWWTP are heavily constrained by the need to mitigate impacts on the airport. Otherwise, Phase 2 for the SWWTP would not be justified.

Option B would be a fallback plan if development accelerates above the forecast growth rate. The smaller Phase 1 SWWTP expansion would not make sense under such conditions.

5.4.4 Improvements to Existing NWWTP

An upgrade of the NWWTP is part of the overall treatment scenario because improvements must be made within the planning period to maintain reliable service. The plant could be renovated to keep existing treatment processes in service, retrofitted to modify existing treatment processes, or replaced with a new facility.

Potential approaches for upgrading the existing NWWTP lagoon system are summarized in the following paragraphs. Upgrades and/or replacements of the clarification and disinfection systems would be part of each upgrade described below.

1. **Full Renovation.** Renovate the existing lagoons to maintain the current capacity in the existing footprint. A future expansion would require the construction of a treatment module and an effluent storage pond on adjacent land.
2. **Partial Renovation.** Renovate half the lagoon system to provide a 0.25-MGD and construct adjacent treatment facilities to maintain or expand current capacity. The other half of the lagoon system could be replaced with an effluent storage pond as part of a future expansion.
3. **Retrofit to Maintain Capacity and Reduce Footprint.** Retrofit half the existing lagoon system with a new, higher-rate lagoon system to maintain a 0.5-MGD capacity within half the existing lagoon footprint. The other half of the existing lagoons could then be replaced with a second-phase lagoon retrofit or an effluent storage pond as part of a future expansion.
4. **Complete Replacement.** The City could construct a new 0.5-MGD WWTP adjacent to the existing facility and abandon all existing components. When an expansion of the replacement plant is needed, the existing lagoon footprint could then be converted to effluent storage.

Chapter 6 evaluates each of these alternative approaches to upgrading the NWWTP in coordination with alternatives for future expansions intended to provide industrial site readiness. This coordinated evaluation also considers potential impacts the alternative NWWTP upgrades and expansions might have on the airport and adjacent farmland used for effluent recycling.

Modifications to the NWWTP must address FAA constraints on the siting of wildlife attractants near airports. Plans for any modifications would need to be submitted to FAA for review and, if costly mitigation measures are mandated, the City may need to investigate alternative sites.

5.5 ALTERNATIVE TREATMENT PROCESSES

5.5.1 *General*

A review of treatment processes was conducted to identify technically-feasible alternatives that should be considered in the evaluation of improvements to the NWWTP. This review identified alternative liquid-stream and solids-stream processes as a first step in determining the most cost-effective approach to upgrading and expanding the plant.

Our review effort considered the guiding principles described in Table 5-5. The technologies selected for evaluation have generally been found to be suitable for the plant size and treatment level required in Madras.

Table 5-5 Principles Used for Identifying Suitable Treatment Processes	
Regulations:	The processes produce Class B effluent for water recycling and Class B biosolids for land application.
Affordability:	The process costs (capital, operating and maintenance) have the potential to be within the City's financial reach.
Operability:	The process complexity and O&M requirements are appropriate for locally-available labor and support.
Reliability:	The process uses technology proven to be reliable.
Compatibility:	The process is compatible with local conditions and constraints.

5.5.2 Liquid-Stream Treatment

The following paragraphs describe the main treatment categories commonly provided at WWTPs and Table 5-6 lists by category the liquid-stream processes we included in the screening effort.

1. **Preliminary Treatment.** Preliminary treatment is provided at the headworks of a WWTP to screen out coarse materials and settle out grit particles. The City should plan to provide screening and grit removal processes upstream of all secondary treatment alternatives to protect the downstream equipment from damage and prevent grit accumulation in tanks or lagoons.
2. **Primary Treatment.** Primary clarifiers are used to reduce the size and cost of the downstream secondary treatment process. But for smaller plants the incremental cost of constructing and operating primary clarifiers is generally more than the potential savings in the secondary process. Therefore, primary clarifiers were not evaluated for this study.
3. **Secondary Treatment.** Secondary treatment consists of a biological process that oxidizes and stabilizes the WW. This level of treatment is required to produce Class B effluent for recycling. Biological processes promote the growth of microorganisms that consume organic matter and, as a byproduct, generate biosolids. These processes require some type of solids separation as part of the overall process to clarify secondary effluent prior to further treatment.
4. **Advanced Treatment.** Advanced treatment processes are provided for further removal of suspended solids, nitrogen and phosphorus to meet strict treatment requirements. Common advanced treatment processes include filtration, polishing lagoons, and constructed wetlands. Since the City does not currently need to meet such strict requirements, our evaluation does not address advanced treatment. The City could add advanced treatment should future conditions warrant. The WWTP layouts we developed allot space for the addition of filters, if necessary.
5. **Disinfection.** Disinfection destroys or inactivates a high proportion of the microorganisms present in the treated WW, particularly pathogens (disease-causing organisms). The disinfection requirements for Class B effluent are established by limitations on total-coliform levels published in the DEQ regulations for water recycling (see Table 1-1).

Table 5-6 Liquid-Stream Treatment Alternatives Included in Screening Effort		
Preliminary Treatment ⁽¹⁾	Secondary Treatment ⁽²⁾	Disinfection
1. Influent Screening • Manually-cleaned bar racks • Mechanically-Cleaned screens 2. Grit Removal • Vortex Grit Chamber • Aerated Grit Chambers • Horizontal Grit Channels	1. Lagoons Systems • Facultative • Partially-Aerated/ Facultative • Aerated/Partially-Mixed 2. Extended Aeration • Earthen Basins • Oxidation Ditch • Compact Configuration • SBR Process	1. Sodium Hypochlorite Bulk Storage & Feed ⁽³⁾ 2. Tablet Chlorinator ⁽³⁾ 3. Ultraviolet Light w/Low-Pressure Lamps 4. Onsite Hypochlorite Generation and Feed ⁽³⁾ 5. Onsite Chlorine Dioxide Generation and Feed ⁽³⁾

(1) New NWWTP screening and grit removal included for all secondary treatment alternatives.

(2) Clarification included with all biological treatment alternatives for solids removal.

(3) Requires chlorine contact tank.

Appendix K provides brief descriptions and summarizes key advantages and disadvantages of the liquid-stream alternatives we screened. Appendix K also identifies other liquid-stream processes we reviewed but discarded prior to screening because they are not suitable for this application.

5.5.3 *Biosolids Handling*

The following paragraphs describe the main categories of biosolids handling processes considered for this study.

1. **Stabilization.** Biosolids from a biological process must be stabilized to meet current requirements for Class B land application. Therefore, a stabilization process must be included with each secondary treatment alternative that produces waste activated sludge not already stabilized in a lagoon system. The existing aerobic digesters at the SWWTP will be maintained in service and expanded along with that plant's future 0.5-MGD expansion.
2. **Thickening.** Thickening equipment separates water from biosolids to reduce the volume that must be processed and, therefore, reduce the size and cost of other biosolids processes. A thickener can produce biosolids with 5.0% solids content by weight and would reduce the volume of biosolids that must be hauled from the SWWTP and emptied onto the drying beds at the NWWTP.

The City has space in an existing storage building on the SWWTP site to house the thickening equipment for freeze protection. Therefore, the key issue is whether the life-cycle costs associated with thickening equipment would be offset by savings in hauling costs and capital costs for drying bed expansions.

Similar savings would not be available at the NWWTP because liquid biosolids can be pumped directly to the drying beds and a new building would be needed to house the equipment. Additionally, if a lagoon system is recommended as the secondary process for NWWTP expansions, biosolids production would be lower than at the SWWTP. Therefore, it is less likely adding a mechanical thickener as part of NWWTP expansions would be cost effective.

3. **Dewatering.** Dewatering biosolids would be another alternative to either continuing the practice of hauling dilute biosolids or adding only a thickener. A dewatering process further reduces the volume of material that must be handled and produces a cake that is handled as a solid. Dewatered biosolids help mitigate potential vector and wildlife-attraction issues associated with handling and land applying liquid biosolids.

Dewatering equipment could be installed at the SWWTP to further reduce the volume of biosolids that must be hauled and emptied onto the drying beds. The drying beds would still be necessary to remove additional moisture for land application and to store biosolids when fields are not accessible.

It is unlikely dewatering equipment would be cost effective as part of NWWTP expansions for the same reasons described above for a thickener. Liquid biosolids can be pumped directly to the drying beds and a new building would be needed to house the equipment.

4. **Drying.** The existing drying beds at the NWWTP are in good condition, and have proven effective. As a result, there is no need to consider their abandonment. However, the drying beds are currently used to capacity when the winter is colder and wetter than average. Therefore, drying bed expansions must be addressed in the evaluations of biosolids handling alternatives. Space is available for drying-bed expansions at the NWWTP site.

The drying beds serve a dual purpose. In addition to drying the solids to facilitate spreading on farmland, they store the solids when fields are not accessible. It is this latter purpose that impacts the relative cost effectiveness of thickening and dewatering processes. Less water in the biosolids hauled to the beds results in the need for less storage volume during the winter.

We screened out the construction of drying beds at the SWWTP due to concerns over land use compatibility. The City prefers not to locate open-air drying beds at the SWWTP because of potential adverse impacts on residential developments planned for the immediate vicinity.

Table 5-7 lists by category the biosolids handling processes identified as suitable alternatives for evaluation in this study.

Table 5-7 Summary of Solids-Stream Process Alternatives			
Biosolids Stabilization	Thickening Process	Dewatering Process	Drying Process
1. Aerobic Digestion 2. Sludge Lagoon 3. Composting 4. Lime Stabilization	Mechanical thickener	Mechanical Screw Press	Sludge Drying Beds

Appendix K provides brief descriptions and summarizes key advantages and disadvantages of the solids-stream alternatives we screened. Appendix K also identifies other solids-stream processes we reviewed but discarded prior to screening because they are not suitable for this application. Evaluation of specific thickening or dewatering equipment can be performed early in design, if such equipment is recommended as part of a WWTP improvements project. (see Chapter 6).

5.6 SCREENING OF ALTERNATIVE PROCESSES

5.6.1 *Criteria for Alternatives Screening*

Initial screening of each alternative listed in Tables 5-6 and 5-7 was performed by evaluating and comparing the processes with regard to the criteria summarized in Table 5-8. These criteria were used to eliminate processes that do not fit this application well based on key management and site-specific considerations. The remaining alternatives were evaluated using more specific criteria as part of the cost-effectiveness analysis described in Chapter 6.

Table 5-8 Summary of Alternatives Screening Criteria	
O&M Considerations:	Operational complexity and equipment O&M requirements affecting long-term maintenance management.
Treatment Reliability:	Effluent quality produced and consistency of effluent quality.
Safety Considerations:	Potential hazards to staff and public.
Siting Constraints:	System is compatible with local land uses and has potential for fitting within framework of existing WW facilities.

5.6.2 Alternatives Screening

Preliminary Treatment Alternatives. Based on initial screening, all alternatives for the NWWTP will include a headworks structure with a mechanically-cleaned screen and low-energy, vortex grit chamber. Manually-cleaned screens (bar racks) were eliminated from further evaluation as the primary method of screening due to poor treatment reliability and safety issues.

Grit removal processes other than low-energy vortex chambers were eliminated due to site compatibility issues, lower reliability, and/or O&M concerns. The 0.5-MGD expansion to the SWWTP would include the addition of a second low-energy vortex grit chamber matching the capacity of the existing chamber. The existing influent screen at the SWWTP has adequate capacity for the potential expansions.

Secondary Treatment Alternatives. Aerated lagoons have the potential of being a cost effective alternative for expansions. But facultative lagoons and partially-aerated/facultative lagoons were eliminated as alternatives for NWWTP expansions due to land requirements and potential compatibility issues with the airport.

Renovation of the existing partially-aerated/facultative lagoon system is an option that would be cost effective only if there is no acceleration in North-Area developments. Otherwise, it would be more appropriate to retrofit the existing NWWTP with a more intensive, aerated lagoon system.

The extended aeration processes using concrete tanks (oxidation ditch, compact configuration, and SBR) are suitable for the area adjacent to the NWWTP site. But an extended aeration system using lined, earthen aeration basins was eliminated as an alternative for NWWTP expansion due to site compatibility issues relating to the shallow depth to underlying basalt.

Disinfection Alternatives. The screening effort concluded that bulk storage and feed of sodium hypochlorite, onsite sodium hypochlorite generation, and tablet chlorination (calcium hypochlorite) equipment are potentially cost effective alternatives. Ultraviolet (UV) light and onsite chlorine dioxide (ClO₂) generation were eliminated as alternatives for NWWTP expansions due to O&M considerations.

Biosolids Handling Alternatives. Alternative processes for biosolids stabilization must be evaluated in conjunction with options for expanding the NWWTP. The screening effort eliminated stabilization processes other than aerobic digesters because of site compatibility issues. Expansions to the SWWTP would include improvements or expansions to the existing aerobic digesters.

We evaluated thickening and dewatering processes for the SWWTP to reduce the biosolids volume that must be hauled and emptied onto drying beds. The evaluation included the following three alternatives:

1. The continued practice of hauling dilute, liquid biosolids from the SWWTP to the sludge drying beds at the NWWTP and the expansion of the drying beds.
2. The addition of only thickening equipment to reduce biosolids volume for liquid hauling and drying combined with a smaller expansion of the drying beds relative to the first alternative.
3. The addition of dewatering equipment to further reduce biosolids volume and hauling as solid material a dump truck.

Evaluations of thickening and dewatering equipment at the NWWTP as biosolids handling alternatives are not warranted at this time for the reasons discussed above in Subsections 5.2.3 and 5.5.3. The costs and maintenance requirements associated with these processes would not be justified.

If space constraints in the future limit opportunities for drying bed expansions, the City can revisit the potential cost effectiveness of thickening or dewatering equipment at the NWWTP.

5.6.3 Summary of Screening Results

Table 5-9 lists the treatment processes that have been identified through the initial screening as potentially cost-effective alternatives for upgrades and expansions to the NWWTP. Appendix K provides additional descriptions of the screening results. Chapter 6 describes the cost-effectiveness analysis we performed to evaluate the remaining alternatives.

Table 5-9	
NWWTP Expansions – Alternatives Recommended for Further Evaluation	
1. Preliminary Treatment at Headworks:	Mechanically-Cleaned Screen and Low-Energy, Vortex Grit Chamber
2. Secondary Treatment	a. Renovation of existing Partially-Aerated, Facultative Lagoon System (no stabilization) b. Aerated/Partially Mixed Lagoon System – retrofit and expansion (no stabilization) c. Oxidation Ditch Extended Aeration d. Compact Extended Aeration Process Configuration e. SBR Extended Aeration
3. Disinfection	a. Sodium Hypochlorite Bulk Storage and Feed b. Onsite Sodium Hypochlorite Generation and Feed c. Tablet Chlorination (Calcium Hypochlorite) Equipment
4. Biosolids Stabilization:	Aerobic Digestion
5. Biosolids Dewatering/Drying:	Sludge Drying Beds

Alternatives for expanding the SWWTP involve expansions or modifications to the existing treatment processes, plus the potential addition of biosolids thickening or dewatering equipment.

5.7 PROPOSED OVERALL SCENARIO FOR TREATMENT IMPROVEMENTS

Table 5-10 (following page) summarizes the overall improvements and potential process alternatives that could be incorporated into the proposed WWTP scenario identified in Table 5-4 (Option A).

Table 5-10
Summary of Improvements and Alternatives for WWTP Scenario

A. SWWTP Expansion, Phase 1 – Retrofit and upgrade SWWTP to increase capacity from 0.54 MGD to 0.65 MGD (Projected Timing – Complete within 10 years).

- Increase pumping capacity at “B” St. South PS and Influent PS.
- Renovate existing headworks (replace/repair preliminary treatment equipment).
- Upgrade the SBR aeration system and other SBR components to expand capacity.
- Convert lower pond to effluent storage pond.
- If determined to be cost effective, add a mechanical thickening process or a mechanical dewatering process.
- Expand sludge drying beds at NWWTP to handle biosolids from SWWTP (size of expansion depends on biosolids handling alternative selected for SWWTP).
- Replace other components as necessary to increase capacity or address deficiencies.

B. Phased NWWTP renovation, retrofit, or replacement (Complete within 15 years).

- Construct headworks with mechanical screening and grit removal.
- Renovate existing lagoons; retrofit the lagoons with aerated lagoon system; or replace the lagoon system with a new secondary treatment process (see Table 5-9 for alternatives).
- Replace DAF clarifier, disinfection system, pumps, and other components, as necessary.

C. SWWTP Expansion, Phase 2 – Construct additional 0.50-MGD treatment module. ⁽¹⁾

- Add pumping capacity at “B” St. South PS and Influent PS.
- Expand headworks to provide sufficient grit removal capacity.
- Construct a third SBR aeration system with all internal components.
- Construct a second post-equalization tank.
- Construct additional effluent storage pond to expand storage capacity.
- Expand aerobic digester capacity.
- If determined to be cost effective, expand the mechanical thickening process or dewatering process.
- Expand sludge drying beds at NWWTP.

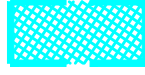
D. NWWTP Expansion(s) – Future phases to serve Industrial-Park developments. ⁽²⁾

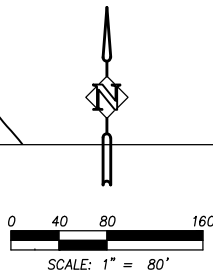
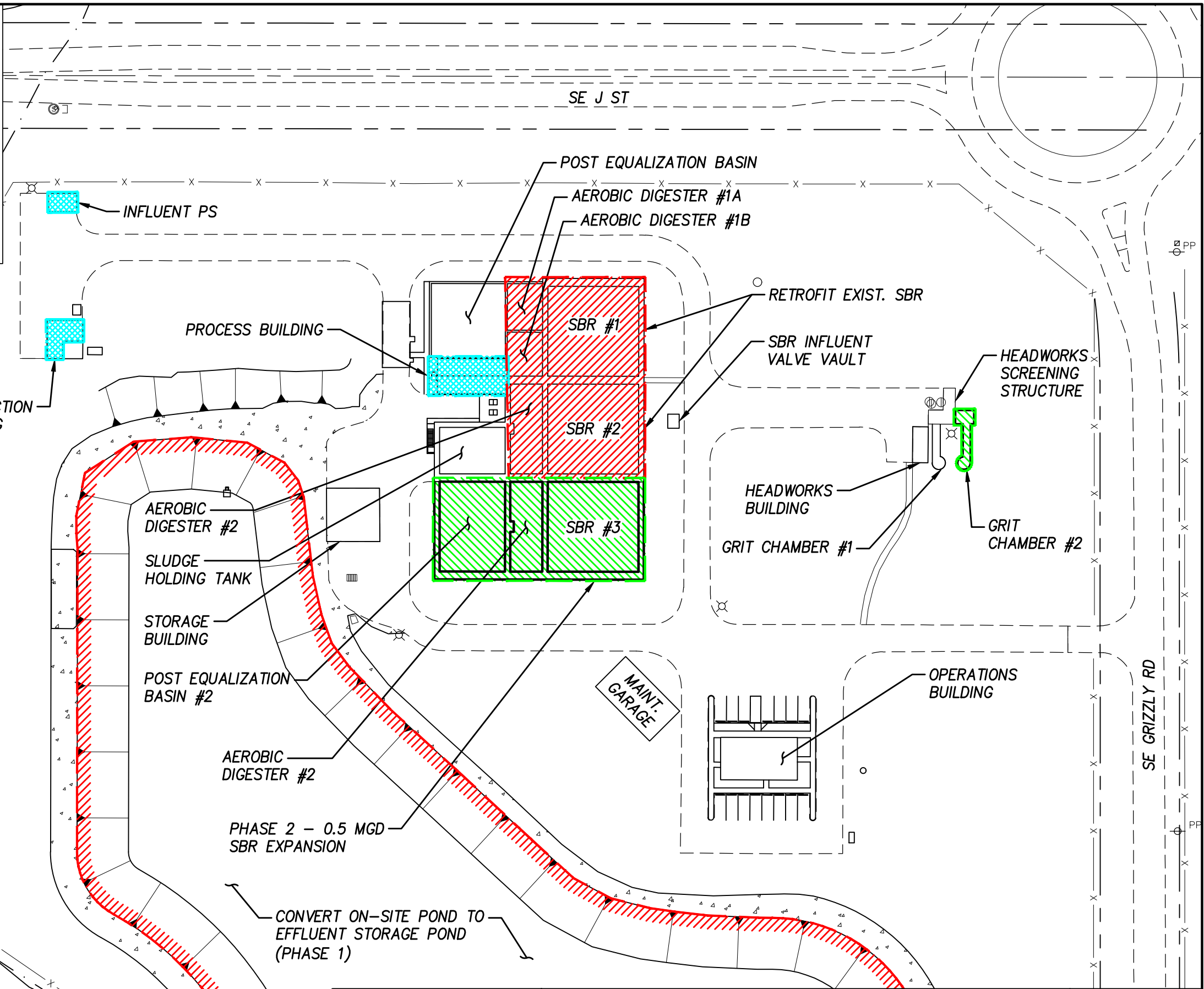
- Expand influent screening and grit removal processes at headworks.
- Construct selected secondary treatment process w/clarifier (see Table 5-9 for alternatives).
- Provide selected disinfection process (see Table 5-9 for alternatives).
- Expand effluent storage capacity.
- Provide selected biosolids stabilization process (see Table 5-9 for alternatives).
- Expand sludge drying beds.

(1) Phase 2 SWWTP expansion would be required when total City-wide flows reach the 20-year (2035) projected flow of 1.14 MGD, if the NWWTP capacity must be dedicated to serving the Industrial Park.

(2) Timing of NWWTP expansions would be driven by Industrial-Park developments and any NWWTP expansion would be contingent on acceptance from the FAA.

SWWTP PHASED RETROFIT & EXPANSIONS

-  PHASE 1 – RETROFIT EXPANSION
(0.5 MGD TO 0.65 MGD)
-  PHASE 2 – 0.5 MGD EXPANSION
(0.65 MGD TO 1.15 MGD)
-  PHASES 1 & 2 – PUMPING, AERATION
& DISINFECTION EXPANSIONS



CHAPTER 6

TREATMENT ALTERNATIVES EVALUATION

6.1 OVERVIEW OF ALTERNATIVES EVALUATION

6.1.1 *General*

This chapter presents the results of the multi-attribute, cost-effectiveness analysis we used to identify a recommended plan for wastewater (WW) treatment over the next 20 years. For the North Wastewater Treatment Plant (NWWTP), we completed a two-stage evaluation of alternatives to address condition issues and industrial site readiness. First, we evaluated alternative processes that remained after the screening effort (Chapter 5) to identify a recommended set of processes for NWWTP expansions. Then we evaluated alternatives for renovating, retrofitting or replacing the NWWTP in coordination with future expansions.

A separate alternatives evaluation was also performed for the South Wastewater Treatment Plant (SWWTP) to determine whether biosolids thickening or dewatering facilities could be cost effective additions.

6.1.2 *Criteria for Evaluating Alternatives*

Table 6-1 summarizes the attributes that were used as the criteria to rank the relative cost effectiveness of the alternatives being evaluated. These criteria were chosen as a means for fully considering the relative financial, social, and environmental impacts of each alternative.

Table 6-1 Summary of Criteria for Alternatives Ranking	
Present Worth Cost:	relative probable life-cycle costs (construction cost, operation and maintenance (O&M) cost, and salvage value).
O & M Requirements:	relative operation and maintenance requirements and process complexity.
Land Use Impacts:	relative land requirements, future expandability, and impacts on airport and other existing land uses.
Treatment Reliability:	relative process stability (sensitivity to changed conditions and potential risk of process upsets or failures).
Operating Flexibility:	relative flexibility in process control.
Sustainability:	relative energy consumption, conservation opportunities, and other environmental issues.

We generated a ranking matrix to compare the relative scores of the alternatives using the criteria presented above. The alternative with the highest ranking for a specific criterion was assigned a score of one. The second-highest ranked alternative was assigned a score of two, and so on. If our evaluation did not identify a significant difference between alternatives for a certain criterion, these alternatives were assigned the same score. Using this procedure, the alternative with the overall lowest point total is the alternative with the highest overall ranking.

The initial ranking assigned all criteria an equal level of importance, meaning we did not assign differing weights to the scoring based on a relative importance for a criterion. This approach was followed to prevent the evaluation from becoming unnecessarily complex and overly subjective.

Even though the relative present-worth cost may be viewed as the bottom line for determining the preferred alternative, other factors such as lower O&M costs and greater reliability may be sufficient to offset a higher overall cost. Once the initial rankings were established, any alternatives that scored close the top were evaluated in more detail to better characterize their relative cost effectiveness.

6.1.3 *Estimates of Probable Costs*

General. Preliminary estimates of probable, life-cycle costs over a 20-year period were generated for each alternative. These probable life-cycle costs include estimates of construction costs and the present worth of both annual O&M costs and salvage values at the end of 20 years. Although major wastewater treatment plant (WWTP) upgrades or expansions are not currently needed, our cost estimates are based on 2016 dollars to establish a consistent basis for comparisons.

The estimates are based on the alternative descriptions provided in this study and from information available at the time this evaluation was performed. The estimates exclude both construction and O&M costs for components that are effectively the same for all the alternatives. Examples of such items are main influent and effluent pipelines to and from the WWTP, effluent pumps and storage ponds, water recycling components, a site access road, and other common site work.

Construction Costs. The probable construction costs developed for this study are planning-level estimates. Chapter 8 provides additional information on the basis of these estimates. The information used to generate the cost estimates has been updated to December 2016 using the Engineering News-Record Construction Cost Index of 10,530. *Chapter 8 lists estimates of probable project costs for the recommended WWTP improvements updated to December 2017.*

Allowances for contractor overhead and profit, mobilization/demobilization, and construction contingencies are included to provide conservative estimate of the probable construction costs. However, since the estimates are provided for comparison purposes, nonconstruction costs for project planning, engineering, and administration have been excluded. At this stage, these costs would be estimated as a percentage of construction cost and would increase each alternative by a common percentage.

O&M Costs. Annual O & M costs generally result from power consumption, labor, repair/replacement parts, and biosolids hauling. Estimates of probable O&M costs were generated from information provided by equipment vendors, City of Madras operating personnel, prior HHPR studies and available literature.

Estimates of horsepower requirements and an average cost of \$0.10 per kilowatt-hour (kW-Hr) provided the basis for power costs. This unit cost includes a contingency factor for the NWWTP since bills from the Central Electric Cooperative, the utility serving that facility, showed an average cost of about \$0.07/kW-Hr in 2016. An average hourly rate of \$40 was used for labor costs to include allowances for benefits, payroll costs, and direct overhead. We used a discount rate of 1.5 percent to calculate the present worth of probable, annual O&M costs for each alternative.

6.2 DEVELOPMENT OF SECONDARY PROCESS ALTERNATIVES

6.2.1. *General*

This section describes the secondary treatment alternatives we evaluated for expansions to the NWWTP (Table 5-9). Preliminary process sizing, configurations, and site layouts were developed for these alternatives to provide a basis for estimating probable life-cycle costs and identifying space requirements.

The secondary process configurations are based on one treatment module with an average daily flow (ADF) capacity of 0.5-MGD. The influent WW characteristics and treatment requirements previously reported in Section 5.3 were used to establish the preliminary sizing criteria for key process parameters.

Secondary systems for extended aeration processes were developed assuming dual-basin configurations for each 0.5-MGD expansion module to provide operating flexibility. The lagoon systems would consist of multiple cells for the same reason. Sufficient redundancy would be provided with pumping equipment to allow operation with one unit out of service.

We assumed outdoor process tanks or basins, other than earthen lagoons, would be constructed of concrete. Tanks or basins can be constructed of steel to reduce initial capital costs and construction time. However, the life-cycle cost of steel tanks would be about the same as concrete due to the need for periodic recoating and the potential for a shorter service life due to corrosion. DAF clarifiers would be steel tanks housed in a building.

Additional facilities that would be required with each of the secondary processes are listed below. These facilities are included in the evaluation because they would differ in some ways between the alternatives. Brief descriptions of these facilities are provided at the end of this section.

- a headworks for preliminary treatment;
- secondary clarifiers for biosolids separation and removal;
- aerobic digesters for biosolids stabilization;
- building to house accessory equipment and process controls; and
- sludge drying beds for biosolids dewatering and land application of Class B biosolids.

6.2.2. Aerated/Partially-Mixed Lagoon System

Aerated lagoons would consist of lined, earthen cells with sloped sides and would be configured to operate in series. Table 6-2 lists the preliminary sizing criteria used for the aerated lagoons alternative and Figure 6-1 shows a preliminary layout of the main system components.

Table 6-2 Preliminary Sizing Criteria – Aerated/Partially-Mixed Lagoon System	
<u>Process Parameter</u>	<u>Criterion</u>
1. Hydraulic Retention Time @ ADF	30 days
2. Total System Volume	15.0 MG
3. No. of Lagoons	3 Cells
4. Design Cell Side Water Depth	8 feet
5. Design Cell Side Slope	3:1 Horizontal to Vertical
6. Design Surface-Area BOD ₅ Loading Rate	220 lbs./day/acre
7. Oxygen Feed Requirements	1.5 lbs. oxygen/lb. BOD applied

Each lagoon cell would be aerated to supply oxygen for biological activity. Aeration could be provided by mechanical aerators, air diffusers with blowers, or possibly a combination of these two. We have included a small blower building with this alternative to accommodate the blowers required with a diffused-air system.

Clarifiers would be provided either as an integral part of the last lagoon cell or as separate tanks. For this study, we have assumed separate, concrete clarifiers would be constructed to be both conservative and because this would provide greater operating flexibility. Waste activated sludge (WAS) pumps are needed to remove settled sludge from the clarifiers and pump it to aerobic digester. No return activated sludge (RAS) pumps are required with lagoons. The sizing and layout of secondary clarifiers and aerobic digesters are described later in this subsection.

6.2.3. *Oxidation Ditch Extended Aeration*

The oxidation ditch process is typified by aeration basins with “racetrack” configurations followed by clarifiers for solids separation. Table 6-3 lists the preliminary sizing criteria used for the oxidation ditch alternative and Figure 6-2 shows a preliminary layout of the process components.

Table 6-3 Preliminary Sizing Criteria Oxidation Ditch and Compact Extended Aeration	
<u>Process Parameter</u>	<u>Criterion</u>
1. Design Volumetric BOD ₅ Loading Rate	25 lbs./day/1,000 c.f.
2. Hydraulic Retention Time @ ADF	18 hours
3. Design MCRT	20 days
4. Design MLSS Concentration	4,000–4,500 mg/L
5. Design MLSS Volatile Solids Content	80 pct. volatile
6. Total Reactor Volume	0.38 MG
7. No. of Reactor Basins	2 MG
8. Design Basin Side Water Depth	14 feet

The contents of each oxidation ditch, termed mixed liquor suspended solids (MLSS), are forced around the basin and kept in suspension by mechanical aerators and/or mixers. The agitation and mixing produced by mechanical aerators also introduces the oxygen required to keep the microbes in the MLSS active.

Oxygen is sometimes added by mounting air diffusers in the oxidation ditches and providing blowers in a separate building. However, we based our analysis on mechanical aerators for this application because the basins will be at the shallow end of the typical range of depths for oxidation ditches. Air diffusers become less efficient at shallower depths. Therefore, no air blowers are included with this alternative.

RAS pumps are needed to recycle settled solids from the clarifiers back to the oxidation ditches. Since microbes multiply as they consume organic matter, WAS pumps must use provided to periodically transfer a portion of the settled solids in the clarifiers to digesters. Secondary clarifiers and aerobic digesters are described later in this subsection.

6.2.4. *Compact Extended Aeration*

The compact extended aeration alternative utilizes a concentric configuration with common-wall construction for the separate tanks. A circular clarifier is located in the center of the structure and is surrounded by a doughnut-shaped section that contains the aeration basin and an aerobic digester tank. Typically, fine-bubble air diffusers are mounted near the bottom of each aeration basin to provide the required oxygen and mixing. Air blowers are housed in a separate blower building.

RAS and WAS pumps are required similar to the oxidation ditch alternative, except that the concentric configuration often results in the pumps being mounted above the clarifier. The sizing of the clarifiers and aerobic digesters would be the same as it would be for the oxidation ditches.

The preliminary sizing criteria used for the compact extended aeration are the same as those used for the oxidation ditches and listed above in Table 6-3. Figure 6-3 shows a preliminary layout of the main process components.

6.2.5. SBR Extended Aeration

The SBR alternative is a fill-and-draw process that operates with time-based cycles. Deeper basins are needed than the aeration basins for other secondary alternatives to accommodate the fill-and-draw cycles.

Table 6-4 lists the preliminary sizing criteria used for the SBR alternative. The layout for the entire SBR system would be similar to the existing SWWTP with common wall construction for the SBR, post equalization, and aerobic digester basins (see Figure 5-1).

Table 6-4 Preliminary Sizing Criteria SBR Extended Aeration		
<u>Process Parameter</u>		<u>Criterion</u>
1. Design Volumetric BOD ₅ Loading Rate	17	lbs./day/1,000 c.f.
2. Hydraulic Retention Time @ ADF	27	hours
3. Design MCRT	16	days
4. Design MLSS Concentration	4,000–4,500	mg/L
5. Design MLSS Volatile Solids Content	80	pct. volatile
6. Total Reactor Volume (at average depth)	0.56	MG
7. No. of Reactor Basins	2	MG
8. Max. Design Basin Side Water Depth	19	feet

SBR basins function as aeration basins during aeration and mixing phases during each operating cycle and then serve as clarifiers during the settling and decanting phases of each cycle. By combining these functions in the SBR tanks, separate clarifiers and RAS pumps are eliminated. WAS pumps are required similar to the other alternatives. A post equalization basin must also be included in the SBR alternative to avoid increasing the capacity of downstream components.

The type of SBR process installed at the South WWTP (SWWTP) uses fine bubble air diffusers to introduce the required oxygen and separate mixers to provide complete mixing. Other configurations with combined aeration and mixing equipment are also available.

6.2.6. Associated Support Facilities

Headworks. We included headworks with screening and grit removal for preliminary treatment as part of each secondary alternative. The NWWTP headworks would need to be constructed with the structure almost entirely above grade to allow gravity flow to the secondary reactor basins. The height of this structure would vary for the different secondary alternatives because basin depths vary for the secondary alternatives.

Initially, the headworks structure would contain a single, mechanical screen with a bypass bar rack and a single vortex grit chamber. A second grit chamber would be added when the plant ADF capacity is expanded from 1.0 MGD to 1.5 MGD. Figure 6-4 shows a preliminary layout of the headworks with approximate dimensions.

The sizing and configuration of the proposed headworks for the NWWTP was based in part on the headworks at the existing South WWTP. Although the overall configuration was modified to better fit the planned site. Our cost estimates were based on using the same type of screening and grit removal equipment as is currently in use at the SWWTP.

Secondary Clarifiers. Clarifiers are required with all the secondary processes, except SBRs. Layouts and cost estimates are based on the construction of circular clarifiers for the oxidation ditch and compact extended-aeration alternatives. The preliminary sizing criteria used for the secondary clarifiers are listed in Table 6-5.

Table 6-5		
Preliminary Sizing Criteria – Secondary Clarifiers		
<u>Process Parameter</u>	<u>Criterion</u>	
1. Total Surface Area (both tanks)	1,400	square feet
2. Tank Side Water Depth	12	feet
3. Length of Overflow Weir (both tanks)	175	feet
4. Peak Design Surface Overflow Rate	950	gpd/sq. ft.
5. Peak Weir Loading Rate	7,600	gpd/lineal ft.

Circular tanks allow for more efficient collection and pumping of the RAS back to the aeration basins. Rectangular clarifiers are included with the aerated lagoons since no RAS system is required with this alternative and less WAS would be produced. An allowance for polymer feed to enhance solids settling was also included with the aerated lagoon alternative.

Aerobic Digesters. Each of the secondary alternative requires aerobic digesters for biosolids stabilization. The digesters would be concrete tanks with aeration provided by diffused air systems. Coarse-bubble diffusers would be installed in the digester tanks and blower equipment would be installed in the process building. The digesters would also include decanting devices to allow some thickening of the tank contents through supernatant withdrawal.

The digester tanks have been sized to provide 60 days of solids retention time assuming an average solids content of 1.0 percent. Smaller aerobic digesters would be provided for the aerated lagoons because fewer biosolids would be produced and the biosolids would be partially stabilized in the lagoons. The aerobic digester capacity would be the same for each extended aeration alternative.

Secondary Process Building. Pumps, aeration blowers, valves, polymer feed equipment, instrumentation, process controls, and electrical equipment for the secondary alternatives would be housed in a building. The building sizes vary for the different alternatives based primarily on different requirements for pumps, blowers and valves. Estimates of the probable construction costs for the buildings are based on structures with concrete floor slabs, cement masonry unit (CMU) walls, and wood-frame roofs with metal, roof decking. The layouts are based on common-wall construction with adjacent process tanks where feasible.

Process-Specific Piping and Valves. The cost estimates for each alternative include allowances for piping, valves and associated components that would be specific to each alternative.

Other Facilities. Additional facilities common to all secondary alternatives have been excluded from the cost comparisons because they would not differ significantly between alternatives. Such facilities include main pipelines to and from the WWTP, effluent pumps and storage ponds, water recycling components, potable water service, a site access road, and other common site work.

6.3 SECONDARY ALTERNATIVES COMPARISON AND RANKING

6.3.1 *Estimates of Probable Present-Worth Costs for Secondary Alternatives*

Table 6-6 summarizes the preliminary estimates of probable costs for each secondary alternative. These present worth cost estimates are based on a 20-year planning period. The scope of each estimate is based on the alternative descriptions provided in Section 6.2. Breakdowns of the preliminary estimates for secondary process alternatives are presented in Appendix L.

Table 6-6					
Estimates of Probable Costs – Secondary Alternatives					
Secondary Alternative	Present Worth Costs (in millions – Dec. 2016)				
	Capital (1)	O&M (2)	Salvage Value (2)	Total Life Cycle	Rank
1. Aerated Lagoons	\$ 4.92	\$ 1.99	\$ 1.31	\$ 5.60	1
2. Oxidation Ditches	\$ 4.90	\$ 2.61	\$ 1.17	\$ 6.34	4
3. Compact Extended Aeration	\$ 4.74	\$ 2.64	\$ 1.11	\$ 6.27	3
4. Sequencing Batch Reactors	\$ 4.56	\$ 2.54	\$ 1.14	\$ 5.96	2

(1) Capital costs are probable construction costs with allowances for contractor overhead and profit, mobilization/demobilization, and construction contingencies.

(2) Probable present worth costs were calculated using 1.5% discount over 20-year planning period.

We estimate the SBRs would be about 6% higher in life-cycle costs than aerated lagoons, whereas we estimate the other alternatives would be about 12% to 13% higher. Our estimates indicate aerated lagoons would have annual O&M costs 22% to 25% less than the other alternatives, which more than offsets the higher probable capital cost. The closeness of the probable present worth costs for the aerated lagoons and SBRs magnifies the importance of nonmonetary considerations.

6.3.2 *Evaluation of Nonmonetary Considerations*

Evaluations of the alternative secondary treatment processes were conducted based on the nonmonetary criteria presented in Subsection 6.1.2. Key advantages and disadvantages of each alternative with regard to these evaluation criteria are summarized in the following paragraphs.

1. Aerated Lagoons

a. O&M Requirements

Advantages:

- simple process to control;
- no automated control valves and no RAS system;
- process can be designed with only mechanical aerators to eliminate O&M of air blowers and fine-bubble diffusers; and
- less WAS and stabilized biosolids to handle than extended aeration systems results in less wear on equipment for handling biosolids.

Disadvantages:

- aeration components in lagoons can be difficult to reach;
- O&M requirements for clarifier equipment; and
- partial-mixing of lagoons results in some solids accumulation that would need to be dredged after 15-20 years.

b. Land Use Impacts

Advantage:

- earthen basins are shallower and less visible than concrete aeration basins.

Disadvantages:

- more land is needed than other processes (would occupy more existing farmland); and
- more difficult to mitigate wildlife attractions with lagoons due to larger surface area, less mixing and sloped sides.

c. Sustainability

Advantages:

- partial-mix process does not require aeration for mixing;
- less energy consumed for pumping influent to lagoons (through headworks) and pumping biosolids than extended aeration systems; and
- less energy is consumed to stabilize and land apply biosolids than other processes.

Disadvantage:

- need for more land with lagoons may increase energy use to pump effluent to more distant farmland for recycling.

d. Reliability

Advantages:

- aerated lagoon system is a stable process that is not sensitive to typical variations in flows and organic loading;
- limited reliance on automation and less pumping equipment reduces risk of electrical and mechanical failures; and

Disadvantage:

- process may not consistently produce the high quality effluent that extended aeration systems can produce resulting in potential for higher disinfection costs.

e. Flexibility

Advantages:

- the proposed, 3-cell system would allow each treatment cell to be separately taken out of service, leaving 67% of treatment capacity in service; and
- lagoons can be retrofitted with partitions, additional aeration and mixing equipment, and/or fixed-film modules to increase treatment capacity.

Disadvantages:

- limited process control gives operators less flexibility to adjust system for changing conditions; and
- earthen basins cannot be designed for adjacent expansions using common-wall construction.

2. Oxidation Ditches**a. O&M Requirements**

Advantages:

- oxidations ditches rely less on automated process controls and control valves than SBRs;
- process can be designed with only mechanical aerators to eliminate O&M of air blowers and fine-bubble diffusers; and
- less automation than SBRs and no automated control valves.

Disadvantages:

- requires O&M of clarifier equipment that is not included with SBRs;
- requires O&M of RAS system that is not included with aerated lagoons and SBRs;
- process control is less simple than aerated lagoons; and
- higher production of WAS and stabilized biosolids than aerated lagoons results in more O&M requirements for biosolids handling.

b. Land Use Impacts

Advantages:

- less land is needed than aerated lagoons; and
- less difficult and costly to mitigate wildlife attractions with smaller, concrete basins than with lagoons.

Disadvantages:

- concrete basins are deeper and more visible than lagoons; and
- slightly more land is needed than compact extended aeration and SBR processes;
- need for clarifiers may produce more of a wildlife attraction than SBR process.

c. Sustainability

Advantages:

- need for less land than lagoons protects more nearby farmland for recycling;

Disadvantage:

- extended aeration processes are more energy intensive than lagoons due to higher biosolids production and more mixing in aeration basins.

d. Reliability

Advantages:

- extended aeration processes may produce a higher quality effluent than aerated lagoons resulting in the potential for lower disinfection costs; and
- oxidation ditches typically produce thicker WAS than SBRs due to use of separate clarifier, resulting in less volume being handled in aerobic digesters.

Disadvantages:

- extended aeration processes with smaller reactor basins can be more sensitive than lagoons to variations in flows and organic loading.

e. Flexibility

Advantages:

- greater process control than lagoons gives operators more opportunity to adjust system for changing conditions; and
- oxidation ditches can be designed for expansions using common wall construction.

Disadvantages:

- the proposed, 2-basin system would mean greater reduction in treatment capacity than lagoons, if one basin is removed from service; and
- less room for retrofit modifications in extended aeration basins than in larger lagoons.

3. Compact Extended Aeration**a. O&M Requirements**

Advantages:

- compact extended aeration relies less on automated process controls and control valves than SBRs; and
- less automation than SBRs and no automated control valves.

Disadvantages:

- requires O&M of clarifier equipment that is not included with SBRs;
- requires O&M of RAS system that is not included with aerated lagoons and SBRs;
- process would include air blowers that require regular maintenance;
- process would include fine-bubble air diffusers that require annual cleaning to maintain efficiency;
- process control is less simple than aerated lagoons; and
- higher production of WAS and stabilized biosolids than aerated lagoons results in more O&M requirements for biosolids handling.

b. Land Use Impacts

Advantages:

- less land is needed than aerated lagoons and oxidation ditches; and
- less difficult and costly to mitigate wildlife attractions than aerated lagoons due to smaller, concrete basins.

Disadvantages:

- concrete basins are deeper and more visible than lagoons; and
- need for clarifiers may produce more of a wildlife attraction than SBR process.

c. Sustainability

Advantages:

- need for less land than lagoons protects more nearby farmland for recycling; and
- diffused air systems with fine-bubble diffusers can be more energy efficient than mechanical aeration, if diffusers are cleaned annually.

Disadvantage:

- extended aeration processes are more energy intensive than lagoons due to higher biosolids production and more mixing in aeration basins.

d. Reliability

Advantages:

- extended aeration processes may produce a higher quality effluent than aerated lagoons resulting in the potential for lower disinfection costs; and
- compact extended aeration will typically produce thicker WAS than SBRs due to use of separate clarifier, resulting in less volume being handled in aerobic digesters.

Disadvantages:

- extended aeration processes with smaller reactor basins can be more sensitive than lagoons to variations in flows and organic loading.

e. Flexibility

Advantages:

- greater process control than lagoons gives operators more opportunity to adjust system for changing conditions.

Disadvantages:

- the proposed, 2-basin system would mean greater reduction in treatment capacity than lagoons, if one basin is removed from service; and
- less room for retrofit modifications in extended aeration basins than in larger lagoons.

4. Sequencing Batch Reactors**a. O&M Requirements**

Advantages:

- no clarifier equipment or RAS system; and
- SBR options are available with only mechanical aerators that eliminate O&M of air blowers and fine-bubble diffusers.

Disadvantages:

- process control is less simple than aerated lagoons;
- SBRs rely more on automation and automated control valves than other alternatives;
- higher production of WAS and stabilized biosolids than aerated lagoons results in more O&M requirements for biosolids handling.

b. Land Use Impacts

Advantages:

- less land is needed than other alternatives; and
- less difficult and costly to mitigate wildlife attractions than all other alternatives without clarifiers and with deeper concrete basins.

Disadvantages:

- concrete basins are deeper and more visible than other alternatives.

c. Sustainability

Advantages:

- need for less land than other alternatives protects more nearby farmland for recycling.

Disadvantage:

- extended aeration processes are more energy intensive than lagoons due to higher biosolids production and more mixing in aeration basins.

d. Reliability

Advantages:

- extended aeration processes may produce a higher quality effluent than aerated lagoons resulting in the potential for lower disinfection costs.

Disadvantages:

- SBRs typically produce dilute WAS due to lack of separate clarifier, resulting in more volume being pumped to aerobic digesters.
- SBRs are more sensitive than other processes to variations in flows and organic loads.

e. Flexibility

Advantages:

- greater process control than lagoons gives operators more opportunity to adjust system for changing conditions; and
- SBRs provide greatest flexibility for expansions by adding individual SBR basins.

Disadvantages:

- the proposed, 2-basin system would mean greater reduction in treatment capacity than lagoons, if one basin is removed from service; and
- less room for retrofit modifications in extended aeration basins than in larger lagoons.

Based on the qualitative alternatives evaluation summarized above, each secondary alternative was assigned a ranking for each nonmonetary criterion. A total score and overall rank was then established from the sum of the individual rankings. The matrix presented in Table 6-7 identifies these non-cost rankings for the secondary alternatives.

Table 6-7 Matrix of Secondary Alternatives Ranking							
Secondary Alternative	Evaluation Criteria						
	O&M Issues (1)	Land Use (2)	Sustainability (3)	Reliability (4)	Flexibility (5)	Combined Score (6)	Nonmonetary Rank
1. Aerated Lagoons	1	4	1	1	2	9	1
2. Oxidation Ditches	3	3	3	2	3	14	3
3. Compact Extended Aeration	4	2	2	2	4	14	3
4. Sequencing Batch Reactors	2	1	3	3	1	10	2

- (1) O&M Issues: relative operation and maintenance requirements and process complexity.
- (2) Land Use Impacts: land requirements, future expandability, and impacts on existing land uses.
- (3) Sustainability: energy consumption, conservation opportunities, and other environmental issues.
- (4) Treatment Reliability: process stability (sensitivity to changed treatment conditions).
- (5) Operating Flexibility: relative flexibility in process control and modifications.
- (6) Total Score: Sum of rankings for five criteria, higher ranking represented by lower score.

6.3.3 *Summary of Secondary Alternatives Evaluation*

Based on our evaluation, an aerated lagoon system is the most cost-effective secondary treatment alternative to implement at the NWWTP. The main advantages to this process are the lower O&M requirements and lower associated costs relative to the other alternatives. There is also considerable flexibility in how aerated lagoons can be configured and there are likely to be opportunities for reducing probable capital costs relative to our planning-level estimate. Therefore, we recommend the City plan to implement aerated lagoons as the secondary treatment process for a retrofit or replacement of the NWWTP and for any future expansions.

Overall, aerated lagoons and SBRs rank close in both costs and nonmonetary factors. Moreover, these alternatives compare well relative to the other alternatives. If the Federal Aviation Administration (FAA) requires covers over lagoons and basins, then the SBR process could be worthy of further consideration due to its smaller footprint.

Oxidation ditches and compact extended aeration are basically the same process with different configurations. As such, there is little difference in their relative rankings and they generally rate lower than the aerated lagoons and SBRs in both monetary and nonmonetary criteria.

6.4 **DEVELOPMENT OF DISINFECTION ALTERNATIVES**

6.4.1 *General*

The alternative disinfection systems being evaluated for expansions to the NWWTP (Table 5-9) are described in this section. Preliminary equipment requirements and sizing were developed for these alternatives to provide a basis for estimating probable life-cycle costs and space requirements. The criteria used to size the disinfection equipment and chlorine contact tank for a 0.5-MGD module are provided in Table 6-8.

Table 6-8 Preliminary Sizing Criteria – Alternative Disinfection Processes	
<u>Process Parameter</u>	<u>Criterion</u>
1. Secondary Effluent Suspended Solids	30 mg/L
2. Design Effluent Total Coliform	2.2 Organisms/100 mL
3. Peak Design Flow	1.33 MGD
4. Max. Design Chlorine Dose	12 mg/L
5. Chlorine Contact Time @ ADF	60 minutes

To provide a common basis for comparing alternatives, this evaluation has assumed a separate, CMU building on a concrete slab would house the equipment. The size of this building would vary for the different alternatives. The cost effectiveness of locating this equipment in the secondary process building can be evaluated during project development.

6.4.2 *Sodium Hypochlorite Bulk Storage and Feed*

The main components specific to this alternative include the following:

- a storage tank for bulk deliveries of sodium hypochlorite (NaOCl) solution;
- two metering pumps with accessories for feeding NaOCl into the secondary effluent; and
- a building about 10 feet by 12 feet in size.

The storage tank would be located outside and would be provided with a heat jacket and insulation for freeze protection. Interior space is required only to house the metering pumps, piping and valves, instrumentation and controls, and other accessories.

6.4.3 *Onsite Sodium Hypochlorite Generation and Feed*

The main components specific to this alternative include the following:

- a storage tank for bulk deliveries of brine solution;
- a package NaOCl generator system supplied with associated instrumentation and controls;
- a day tank to store dilute NaOCl solution as it is generated;
- two metering pumps with accessories for feeding NaOCl into the secondary effluent;
- electrical equipment for the generator system; and
- a CMU building about 12 feet by 22 feet in size.

The brine storage tank and NaOCl solution tank would be located outside and would be provided with a heat jacket and insulation for freeze protection. The building would house all the generator equipment, pumps, day tank, and process controls. A separate room would be provided in the building to house the electrical equipment.

6.4.4 *Tablet Chlorination (Calcium Hypochlorite) Equipment*

The main components specific to this alternative include the following:

- a package tablet-chlorinator system supplied with associated instrumentation and controls;
- a day tank to store hypochlorite solution;
- two metering pumps with accessories for feeding NaOCl into the secondary effluent; and
- an building about 12 feet by 16 feet in size.

The hypochlorite solution tank would be located outside and would be provided with a heat jacket and insulation for freeze protection. The building would house all the equipment, pumps, day tank, and process controls.

6.4.5 *Common Disinfection Facilities for All Alternatives*

Additional facilities that are common to all disinfection alternatives are listed below. These facilities have been excluded from the alternatives evaluation because they would not differ significantly for the different alternatives.

- a chlorine residual analyzer;
- a concrete containment area for the storage tank;
- a concrete contact tank to provide proper time for adequate disinfection;
- a mixer to disperse hypochlorite solution at the upstream end of the contact tank;
- water service connection to building and emergency eyewash; and
- access driveway and other site work.

6.5 DISINFECTION ALTERNATIVES COMPARISON AND RANKING

6.5.1 *Estimates of Probable Present-Worth Costs for Disinfection Alternatives*

Table 6-9 summarizes the preliminary estimates of probable costs for each disinfection alternative. These estimates are given as present worth costs using a 20-year planning period. The scope of each estimate is based on the alternative descriptions provided in Section 6.3. Breakdowns of these preliminary estimates are presented in Appendix L.

Table 6-9 Estimates of Probable Costs – Disinfection Alternatives					
Disinfection Alternative	Present Worth Costs (in thousands – Dec. 2016)				
	Capital ⁽¹⁾	O&M ⁽²⁾	Salvage Value ⁽²⁾	Total Life Cycle	Rank
1. NaOCl Bulk Storage& Feed	\$ 141	\$ 926	\$ 37	\$ 1,030	3
2. Onsite NaOCl Generation & Feed	\$ 421	\$ 601	\$ 77	\$ 945	2
3. Tablet Chlorination Equipment	\$ 224	\$ 735	\$ 51	\$ 908	1

(1) Capital costs are probable construction costs with allowances for contractor overhead and profit, mobilization/demobilization, and construction contingencies.

(2) Probable present worth costs were calculated using 1.5% discount over 20-year planning period.

The tablet chlorination system is estimated to have lower life-cycle costs than the other alternatives. However, the life-cycle costs of the onsite generation and feed system are estimated to be only about 4% higher. The bulk storage and feed option would require the lowest capital investment, but has the highest O&M costs due to the need to purchase liquid NaOCl solution.

6.5.2 *Evaluation of Nonmonetary Considerations*

Evaluations of the alternative disinfection treatment processes were conducted based on the applicable nonmonetary criteria presented in Subsection 6.1.2. Summaries of the key advantages and disadvantages for each alternative are provided below.

The criterion of land use impacts was not considered in the evaluation because the alternatives do not significantly differ with regard to the amount of land required and associated impacts. Process flexibility is also not a variable among the disinfection alternatives. Each alternative can be designed with similar levels of operating flexibility.

1. **NaOCl Bulk Storage and Feed**

a. **O&M Requirements**

Advantages: simple process with the least amount of equipment.

Disadvantages: requires storage and feed of more concentrated NaOCl solution than onsite generation with greater risk to personnel of burning or irritation.

b. **Land Use Impacts – no significant differences**

c. **Sustainability**

Advantages: no onsite generation of disinfectant solution results in less power being required for onsite process.

Disadvantages: requires trucking of bulk deliveries of NaOCl solution.

d. Reliability

Advantages: less equipment results in lower chances of equipment faults or failures.

Disadvantages: relies on more frequent deliveries.

e. Flexibility – no significant differences**2. Onsite NaOCl Generation and Feed****a. O&M Requirements**

Advantages: more dilute NaOCl solution generated than other alternatives posing less of a potential hazard to personnel.

Disadvantages: more complex than other alternatives with more equipment components and associated O&M required.

b. Land Use Impacts – no significant differences**c. Sustainability**

Advantages: Onsite generation involves transportation of less bulk material resulting in fewer deliveries (less brine solution than NaOCl solution by weight).

Disadvantages: requires more onsite power than other alternatives for other generation and requires trucking of bulk brine-solution deliveries.

d. Reliability

Advantages: less frequent brine deliveries than bulk NaOCl deliveries.

Disadvantages: more equipment results in higher chances of equipment faults or failures.

e. Flexibility – no significant differences**3. Tablet Chlorination Equipment****a. O&M Requirements**

Advantages: requires storage and handling of dry tablets with low hazard potential.

Disadvantages: more equipment than storage and feed of bulk NaOCl.

b. Land Use Impacts – no significant difference**c. Sustainability**

Advantages: Use of dry tablets requires no transportation of bulk solution resulting in least amount of material deliveries by weight and frequency.

Disadvantages: dissolving tablets and mixing solution requires more onsite power than bulk storage and feed of NaOCl solution.

d. Reliability

Advantages:

- lower space requirements for tablet storage allows less frequent deliveries; and
- less equipment than onsite NaOCl generation results in lower chances of equipment faults or failures.

Disadvantages: more equipment than bulk NaOCl storage and feed results in higher chances of equipment faults or failures.

e. Flexibility – no significant difference

The matrix presented in Table 6-10 identifies the non-cost rankings for the disinfection alternatives. Each disinfection alternative was assigned a ranking for each applicable criterion based on the qualitative alternatives evaluation summarized above. A total score and overall rank was then established from the sum of the individual rankings.

Table 6-10 Matrix of Disinfection Alternatives Ranking					
Disinfection Alternative	Evaluation Criteria				
	O&M Issues (1)	Sustainability (2)	Reliability (3)	Total Score (4)	Overall Rank
1. NaOCl Bulk Storage & Feed	1	3	1	5	1
2. Onsite NaOCl Generation & Feed	3	2	3	8	3
3. Tablet Chlorination Equipment	2	1	2	5	1

(1) O&M Requirements: relative operation and maintenance requirements and process complexity.

(2) Sustainability: energy consumption, conservation opportunities, and other environmental issues.

(3) Treatment Reliability: process stability (sensitivity to changed treatment conditions).

(4) Total Score: Sum of rankings for six criteria, higher ranking represented by lower score.

6.5.3 Summary of Disinfection Alternatives Evaluation

The rankings show the tablet chlorination system scores about the same in nonmonetary considerations as the bulk storage and feed system. Since the tablet chlorination system is estimated to have a lower life-cycle cost, and about 20% lower O&M costs, it appears to be the preferred disinfection alternative.

6.6 EVALUATION OF ALTERNATIVES FOR EXISTING NWWTP

6.6.1 General

The condition assessment described in Chapter 2 reports that the existing NWWTP will require significant investment within 15 years to maintain the 0.5-MGD facility in reliable service. As discussed in Chapter 5, the City could renovate, retrofit, or replace the existing facility to provide the required capacity.

We evaluated alternatives for upgrading the NWWTP in conjunction with an expansion to identify a preferred approach for addressing condition issues while maintaining industrial site readiness. The evaluation compared alternatives for providing an expanded NWWTP capacity of 1.0 MGD and considered site requirements for an additional expansion to 1.5 MGD.

Aerated lagoons were incorporated into the alternatives evaluation as the recommended secondary treatment process, consistent with our analysis of process alternatives. All options for retrofitting or replacing the existing lagoons and constructing future plant expansions included aerated lagoons for secondary treatment. Alternatives that involve renovating the existing lagoons to maintain the partially-aerated, facultative lagoon process in service were also considered.

The main considerations in evaluating these alternative upgrades are summarized below:

- the pace of development in the North Area;
- the need for flexibility to accommodate expansions for maintaining industrial site readiness;
- the impacts on the airport that expansions of open lagoons would have; and
- the availability of farmland near the NWWTP for effluent recycling by irrigation.

6.6.2 *Descriptions of Alternatives*

This subsection describes the alternative approaches for addressing upgrades and expansions to the existing NWWTP over the planning period. The main elements listed as being specific to each alternative were included in the estimates of probable construction cost. The cost estimates exclude the common elements listed at the end of this subsection.

1. **Option 1 – Full Renovation of Lagoon System and Adjacent Expansion.** A complete renovation of the partially-aerated, facultative lagoons would maintain the current 0.5-MGD capacity in the existing footprint. The renovation would occur in phases to keep the facility in service with the scheduling of the phases based on available funding and priorities for equipment replacement.

The main elements specific to a full renovation are listed below.

- Clean, replace the liner, and restore the earthen liner cover in the primary and secondary lagoon cells (Cells A thru D).
- Replace the aeration systems and influent pipes in the primary cells (Cells A and B).
- Replace the lagoon transfer structures and access walkways for Cells A thru D.
- Replace transfer piping between lagoon cells.

The two primary cells would continue to be aerated lagoons and the secondary cells would remain facultative lagoons. Yard piping at the plant site installed during the 1990s or later would be left in place to the extent feasible. The polishing lagoon is no longer needed with the installation of the DAF clarifier and would be demolished or converted to effluent storage.

A 0.5-MGD aerated lagoon module and additional effluent storage capacity would be constructed on adjacent land to expand the NWWTP capacity to 1.0 MGD.

A preliminary layout for this approach is shown in Figure 6-5. This layout assumes the renovated lagoons would be replaced with a second, adjacent treatment module either before or as part of an expansion to 1.5 MGD. This sequence would allow the existing lagoon site to be converted to an effluent storage pond.

If the pace of development requires an expansion beyond 1.0 MGD while the renovated lagoons still have significant service life remaining, the effluent storage pond for the 1.5-MGD capacity would be constructed on an adjacent site, similar to what is shown for Option 4 in Figure 6-8.

2. **Option 2 – Partial Renovation Combined with Adjacent Expansion.** As an alternative to full renovation, the City could renovate half the lagoon system (one primary and one secondary cell). The renovated lagoons would provide a 0.25-MGD capacity. Therefore, the City would need to construct a separate 0.75-MGD treatment module to expand the plant to a 1.0-MGD capacity. The remainder of the existing lagoon system would be converted to an effluent storage pond. Figure 6-6 illustrates this alternative.

The following main elements are specific to the partial lagoon renovation.

- Replace the liner in one primary and one secondary cell (Cells A and C).
- Replace the aeration systems and influent pipes in Cell A.
- Replace the lagoon transfer structures and access walkways for Cells A and C.
- Replace transfer piping between Cells A and C.
- Modify Cells B and D to convert the area to an effluent storage pond.
- Construct an adjacent treatment module to expand capacity.

This approach would increase land area required for an adjacent lagoon system expansion, but would reduce the required land area outside the existing plant footprint for effluent storage. As with the full renovation, the renovated lagoons could eventually be replaced and converted to effluent storage capacity.

3. **Option 3 – Retrofit to 0.5-MGD Aerated Lagoons with Adjacent Expansion.** The existing lagoon system can be retrofitted to convert one primary cell to a 0.5-MGD aerated lagoon system. This retrofit option would reduce the volume required to provide a 0.5-MGD capacity by increasing lagoon depth and installing a new aeration system, partitions, fixed-film media, and/or other process components.

The remainder of the existing lagoon footprint would be converted to effluent storage and an adjacent aerated lagoon system would be constructed to expand to NWWTP to 1.0-MGD. Figure 6-7 shows a preliminary layout for this option.

The following main elements are specific to this lagoon retrofit option.

- Replace the liner and influent pipes in one primary cell (Cell A).
 - Retrofit one primary cell (Cell A) by installing a higher-capacity aeration system, two or three partitions to subdivide the cell, and plastic media to promote fixed-film growth.
 - Replace the lagoon transfer structure and access walkway for Cell A.
 - Install new piping between Cell A and the DAF clarifier.
4. **Option 4 – Retrofit to Increase Capacity with Aerated Lagoons.** This second retrofit option would convert the entire lagoon system to aerated lagoons in two phases to double the plant capacity within the existing footprint. The retrofit/expansion would include similar components as described in Option 3 above and would require construction of effluent storage capacity entirely on adjacent land (see Figure 6-8).

The following main elements are specific to this lagoon retrofit option.

- Replace the liner and influent pipes in both primary cells (Cells A and B).
 - Retrofit both primary cells with a higher-capacity aeration system, two or three partitions to subdivide each cell, and plastic media to promote fixed-film growth.
 - Replace the lagoon transfer structure and access walkway for Cells A and B.
 - Install new piping to allow the two primary cells to be operated in series.
 - Install new piping from each primary cell to the DAF clarifier.
5. **Option 5 – Complete Replacement with New Facility.** The City could construct an entirely new 0.5-MGD WWTP and abandon all existing components instead of implementing a major renovation or retrofit. The replacement facilities would consist of the same preliminary, secondary and disinfection processes that are recommended for expansions.

It would make sense to combine this replacement with an expansion that would convert the existing lagoon footprint to effluent storage. If the replacement were needed before the expansion was warranted, the existing lagoons could be abandoned until an expansion is needed. However, some regrading and maintenance would be required to prevent that area from becoming an attractant for wildlife.

6. **Common Facilities.** Each option described above would include the following common elements.

- Add new headworks and replace the influent flow meter.
- Replace the DAF clarifier.
- Construct a new building to house the DAF unit(s) and accessories for weather protection.
- Replace the disinfection equipment with a tablet chlorinator.
- Coat the interior of the existing chlorine contact tank.
- Abandon and regrade the polishing pond (Cell E).
- Replace pumps and associated valves (intermediate, effluent, solids and chemical pumps).
- Renovate the existing main building and demolish the small chlorination building.
- Keep the existing sludge drying beds and expand them as part of plant expansions.

These other facilities could potentially be installed or constructed in a separate project phase from the renovation, retrofit, or replacement of the lagoon system depending on the availability of funds, equipment condition, and other factors.

6.6.3 *Estimates of Probable Present-Worth Costs for Alternative NWWTP Upgrades*

Table 6-11 lists the preliminary estimates of probable costs for the alternative approaches to upgrading the NWWTP and expanding it to a 1.0-MGD capacity. The present worth estimates are based on a 20-year planning period and the alternative descriptions provided above. Breakdowns of these preliminary estimates are presented in Appendix L. The probable cost of expanding effluent storage capacity is not included since it would be about equal for each alternative.

Table 6-11 Estimates of Probable Costs Alternative NWWTP Upgrades and Expansions					
Alternative Approach ⁽¹⁾	Present Worth Costs (in millions – Dec. 2016)				
	Capital ⁽²⁾	O&M ⁽³⁾	Salvage Value ⁽³⁾	Total Life Cycle	Rank
1. Full Renovation of Lagoons	\$ 15.67	\$ 4.77	\$ 3.53	\$ 16.91	1
2. Partial Renovation of Lagoons	\$ 16.70	\$ 5.03	\$ 3.78	\$ 17.95	4
3. Retrofit to Maintain Capacity ⁽⁴⁾	\$ 16.32	\$ 5.30	\$ 3.88	\$ 17.74	3
4. Retrofit to Expand Capacity ⁽⁴⁾	\$ 15.26	\$ 5.30	\$ 3.72	\$ 16.84	1
5. Replace w/New Facility	\$ 17.63	\$ 5.30	\$ 4.03	\$ 18.90	5

(1) Each alternative includes an expansion w/aerated lagoons to provide a 1.0-MGD NWWTP capacity.

(2) Capital costs are probable construction costs with allowances for contractor overhead and profit, mobilization/demobilization, and construction contingencies.

(3) Probable present worth costs were calculated using 1.5% discount over 20-year planning period.

(4) Retrofit would construct aerated lagoon system in place of the existing partially aerated lagoons.

6.6.4 *Evaluation of Alternative NWWTP Upgrades and Expansions*

The estimated probable present worth costs of Options 1 and 4 are essentially equal. Option 1 would involve a full renovation of the existing plant plus an adjacent 0.5-MGD expansion, whereas Option 4 would entail a complete lagoon retrofit to provide a 1.0 MGD capacity. The probable present worth costs for the other three options are estimated to be about 5% to 12% higher.

Both Options 1 and 4 would require the expansion of effluent storage capacity outside the footprint of the existing facilities for a 1.0 MGD facility. As a result, the implementation of either of these options would depend on the FAA accepting the expansion of open lagoon surfaces outside the existing footprint. However, Option 4 would eliminate facultative lagoons from the secondary process and reduce the overall surface area of treatment lagoons relative to Option 1. This smaller footprint for treatment lagoons combined with increased surface agitation in aerated lagoons can help mitigate wildlife attraction.

There is also greater variety in the equipment and process configurations that could be used with aerated lagoons. As a result, there would be more flexibility in establishing the process sizing and layout relative to the partially-aerated, facultative lagoon system.

Options 2 and 3 would allow the conversion of approximately half the existing site into effluent storage, but would still require additional effluent storage capacity outside the existing plant footprint. Option 5 would allow the expansion of effluent storage capacity for a 1.0-MGD plant entirely within the existing lagoon footprint. However, our estimates indicate this option would have the highest present worth cost.

The future expansion of the NWWTP up to 1.5 MGD would require the construction of an effluent storage pond on adjacent land for all options.

6.6.5 *Recommended Approach to NWWTP Upgrades and Expansions*

We recommend the City plan to implement Option 4 for upgrading and expanding the NWWTP for the following reasons.

- Option 4 is estimated to have probable capital and life-cycle costs that are approximately equal to Option 1 and lower than the other options.
- The full retrofit Option 4 provides would eliminate the larger facultative lagoons (secondary cells) that would remain under both Options 1 and 2. Facultative lagoons not only require more area, they also are more prone to algae growth and more likely to act as wildlife attractants.
- There is considerable flexibility in how the aerated-lagoon retrofits can be configured which allows flexibility in tailoring the design capacity of the retrofits to actual development conditions. Potentially, it could even be feasible to fit aerated lagoons with a total capacity of 1.5 MGD within the footprint of the existing lagoon system.
- Aerated lagoons receive more aeration and mixing than the existing lagoon system. This reduces the probability of algae growth and also provides a less attractive environment for water fowl due to greater surface agitation.

Table 6-12 summarizes the recommended Option 4 improvements. The retrofit to upgrade the plant and maintain the 0.5-MGD capacity will likely be completed in phases due to limitations in funding and differing conditions of existing components. The recommended phasing, our estimates of probable project costs for each phase, and project timing are presented in the capital improvements plan (Chapter 8).

Table 6-12 Summary of Proposed Improvements for NWWTP Scenario	
A. Phased NWWTP Retrofit to Maintain 0.5-MGD Capacity (Complete within 15 years). • Replace DAF clarifier, disinfection system, pumps, and related components. • Construct headworks with mechanical screen and low-energy, vortex grit chamber. • Retrofit half of the existing lagoons to convert to an aerated lagoon system as the new secondary treatment process. • Construct an aerobic digester/holding tank for biosolids. • Prioritize and complete the improvements in phases as funds are available and according to O&M priorities. • Maintain existing sludge drying beds in service. • Maintain existing effluent storage pond and associated piping in service.	
B. NWWTP Expansion(s) – Future 0.5-MGD Incremental Expansion(s) When Needed to Serve Industrial-Park Developments. ⁽¹⁾ ⁽²⁾ • Expand influent screening and grit removal processes at headworks. • Construct aerated lagoon system w/clarifier and aerobic digester. • Provide tablet chlorinator process for effluent disinfection. • Expand effluent storage capacity. • Expand sludge drying beds.	

(1) Timing of NWWTP expansions would be driven by Industrial-Park developments and any NWWTP expansion would be contingent on acceptance from the FAA.

(2) Phase 2 SWWTP expansion would allow future NWWTP expansions to be delayed or avoided by freeing up most or all the 0.5-MGD NWWTP capacity to serve the Industrial Park.

6.7 EVALUATION OF BIOSOLIDS ALTERNATIVES FOR EXISTING SWWTP

6.7.1 *Existing SWWTP Biosolids Handling*

Biosolids consisting mostly of water and microorganisms must be routinely wasted from the SWWTP SBR basins to maintain a stable process. These biosolids are pumped to adjacent aerobic digesters where the material receives further treatment. After a sufficient residence time in the digesters, the treated biosolids are pumped to a 4,300-gallon tank truck and hauled to the sludge drying beds at the NWWTP.

Dried material is removed from the drying beds and spread on farmland as a soil amendment. Although data is not currently available on the solids content of the hauled liquid, it is likely to contain approximately 1.0 – 1.5 percent solids on a dry weight basis (98.5 – 99.0 percent water).

Table 6-13 describes the average hauling efforts required to transport biosolids to the drying beds at current flow rates and biosolids production.

Table 6-13 Current Biosolids Hauling Practices	
<u>Description of Practice</u>	<u>Average Value</u>
1. Hauling Frequency	6 days/month
2. Truck Loads:	
a) Daily	7 loads/day
b) Annually	504 loads/year
3. Roundtrip Distance	10 miles/load
4. Annual Volume Hauled	1.80 MG/year
5. Approximate Annual Solids Hauled ⁽¹⁾	146,000 lbs./year
6. Annual Labor Requirement ⁽²⁾	576 hours/year
7. Annual Fuel Requirement ⁽³⁾	1,800 gallons/year
9. Estimated Hauling Cost ⁽⁴⁾	\$ 32,000 /year

(1) Assumes 1.0% dry solids content.

(2) City reports 8 hours labor for one worker required per haul day.

(3) City reports 0.5 tank consumed per haul day and 50-gallon tank capacity.

(4) Includes hauling labor, fuel cost, and annual truck maintenance.

The data in Table 6-13 show that biosolids hauling requires approximately 30% of the hours worked by one worker, after deducting leave time. As SWWTP flows increase, biosolids production in the SBR will increase and the hauling effort will need to ramp up accordingly. We estimate the time commitment would increase to approximately 65% for one worker when flows reach the capacity of the proposed Phase One SWWTP expansion.

6.7.2 Descriptions of SWWTP Biosolids Alternatives

General. The only alternative processes we evaluated for the SWWTP were the addition of either biosolids thickening equipment or biosolids dewatering equipment. Each of these options would reduce the volume of water the City would need to transport from the SWWTP and discharge onto the drying beds. Lower biosolids volume would reduce hauling costs and allow a reduction in the required drying bed capacity.

Alternative One – Continue Hauling Dilute Biosolids. The main considerations in continuing current hauling practices are the increasing labor commitment it will require and the need to expand the drying beds to provide capacity to store the high volume of liquid.

Our calculations estimate an additional 48 drying beds would need to be constructed when average flows reach the design flows for the Phase 1 expansion of the SWWTP. This expansion would provide capacity for drying through drainage and evaporation during most of the year, plus seasonal storage capacity when cold and wet conditions can prevent any net moisture loss.

Alternative Two – Biosolids Thickening. The main components specific to this alternative include the following:

- a polymer storage and feed system;
- a flocculation tank;
- the thickener unit with instrumentation and controls;
- a stainless steel discharge bin sized to hold at least 1.5 truckloads of thickened biosolids;
- a positive displacement pump for biosolids transfer to the truck; and
- continued use of the existing tank truck.

The City has identified an existing storage building at the SWWTP as a suitable location to house the thickener and associated equipment. This building is near the site where the tank truck is currently filled with liquid biosolids. Piping would need to be installed to convey thickener supernatant to the plant drain sump for recycling to the headworks.

Staff would operate the thickener on hauling days to reduce the biosolids volume before transferring it to the truck. Our analysis is based on producing material with an average solids content of 4.0%. This is a conservative estimate, since well-operated thickeners commonly produce material at 5.0% solids. We've assumed the thickened biosolids would need to be pumped from the tank truck onto the drying beds to expedite unloading.

We estimate the combined labor required for thickened-biosolids hauling and thickening system O&M would be approximately half the labor required for hauling dilute biosolids. We also estimate an additional 12 drying beds would need to be constructed for the Phase 1 SWWTP expansion, if a thickening system were installed.

Alternative Three – Biosolids Dewatering. This alternative would add a screw press to dewater the biosolids. The digested biosolids would be pumped into the flocculation tank, then flow to the screw press. A dewatered cake would discharge into a hopper and a conveyor system would load the material into a dump truck.

We have assumed the dewatering process with associated equipment would be housed in the same existing storage building identified for the thickener alternative. Similar to the thickener process, recycle piping would need to be installed to the plant drain sump.

Our analysis is based on the screw press producing material with an average solids content of 15%, since the typical production range for a well-operated process is 15 – 18% solids. Because the screw press output must be handled as a solid material, some additional labor time would be needed to spread the cake onto the drying beds to facilitate the evaporation process. It is unlikely that any drainage would occur from the dewatered material as it does most of the year from liquid biosolids.

We have included the purchase of a used dump truck with a 25 cubic yard capacity for hauling biosolids to the drying beds. Assuming the purchase of a used truck allows for a fair comparison to the other alternatives, since the existing tank truck is used. A truck with a minimum 25 cubic yards load capacity is needed since a smaller, conventional dump truck with a 10-14 cubic yard capacity would not carry enough material in each load to be cost competitive.

Since dewatered solids from a screw press would require very little winter storage capacity, we estimate existing drying bed capacity would be adequate for the biosolids produced at the proposed Phase One design flows.

6.7.3 Estimates of Probable Present-Worth Costs for SWWTP Biosolids Alternatives

Table 6-14 summarizes the preliminary estimates of probable costs for each biosolids alternative. These estimates are given as present worth costs using a 20-year planning period. Breakdowns of these preliminary estimates are presented in Appendix L.

Table 6-14					
Estimates of Probable Costs – SWWTP Biosolids Handling Alternatives					
Biosolids Handling Alternative	Present Worth Costs (in thousands – Dec. 2016)				
	Capital ⁽¹⁾	O&M ⁽²⁾	Salvage Value ⁽²⁾	Total Life Cycle	Rank
1. Haul Dilute Liquid Biosolids ⁽³⁾	\$ 1,270	\$ 910	\$ 350	\$ 1,830	3
2. Add Thickening Process ⁽⁴⁾	\$ 875	\$ 605	\$ 160	\$ 1,320	1
3. Add Dewatering Process	\$ 970	\$ 695	\$ 120	\$ 1,545	2

(1) Capital costs are probable construction costs with allowances for contractor overhead and profit, mobilization/demobilization, and construction contingencies.

(2) Probable present worth costs were calculated using 1.5% discount over 20-year planning period.

(3) Capital costs are for the addition of 48 drying beds (Total of 72 beds).

(4) Capital costs include the thickening process and the addition of 12 drying beds (Total of 36 beds).

The addition of a thickening process is estimated to have significantly lower life-cycle costs than the other alternatives. This is estimated to include both lower capital investments and lower O&M costs. We estimate the life-cycle costs of a dewatering process would more than offset the savings from not expanding the drying beds.

The combined labor required for Alternative Three (dewatered-biosolids) is estimated to be slightly less than the combined labor required for Alternative Two (thickened-biosolids). However, Alternative Three is estimated to have approximately 10% higher O&M costs than Alternative Two because of higher polymer use. Other material costs for maintenance and repairs would also be higher with the addition of a dewatering unit.

6.7.4 Evaluation of Nonmonetary Considerations

Alternatives Two and Three would increase reliance on mechanical equipment. However, neither of the processes is particularly complex or difficult to maintain. The thickening and dewatering equipment are also proven technologies that are widely used for biosolids.

Key considerations in evaluating the biosolids handling alternatives include the impacts of transporting the material between sites. Alternatives Two and Three would significantly reduce the environmental impacts and safety concerns associated with biosolids hauling. Both these alternatives would also reduce land requirements for drying bed expansions.

The thickening process would require the addition of approximately half the polymer that the dewatering unit would require. This has environmental benefits due to lower energy use for manufacturing and transporting the chemicals. There are also safety and housekeeping benefits from less handling of polymer drums and polymer feed equipment.

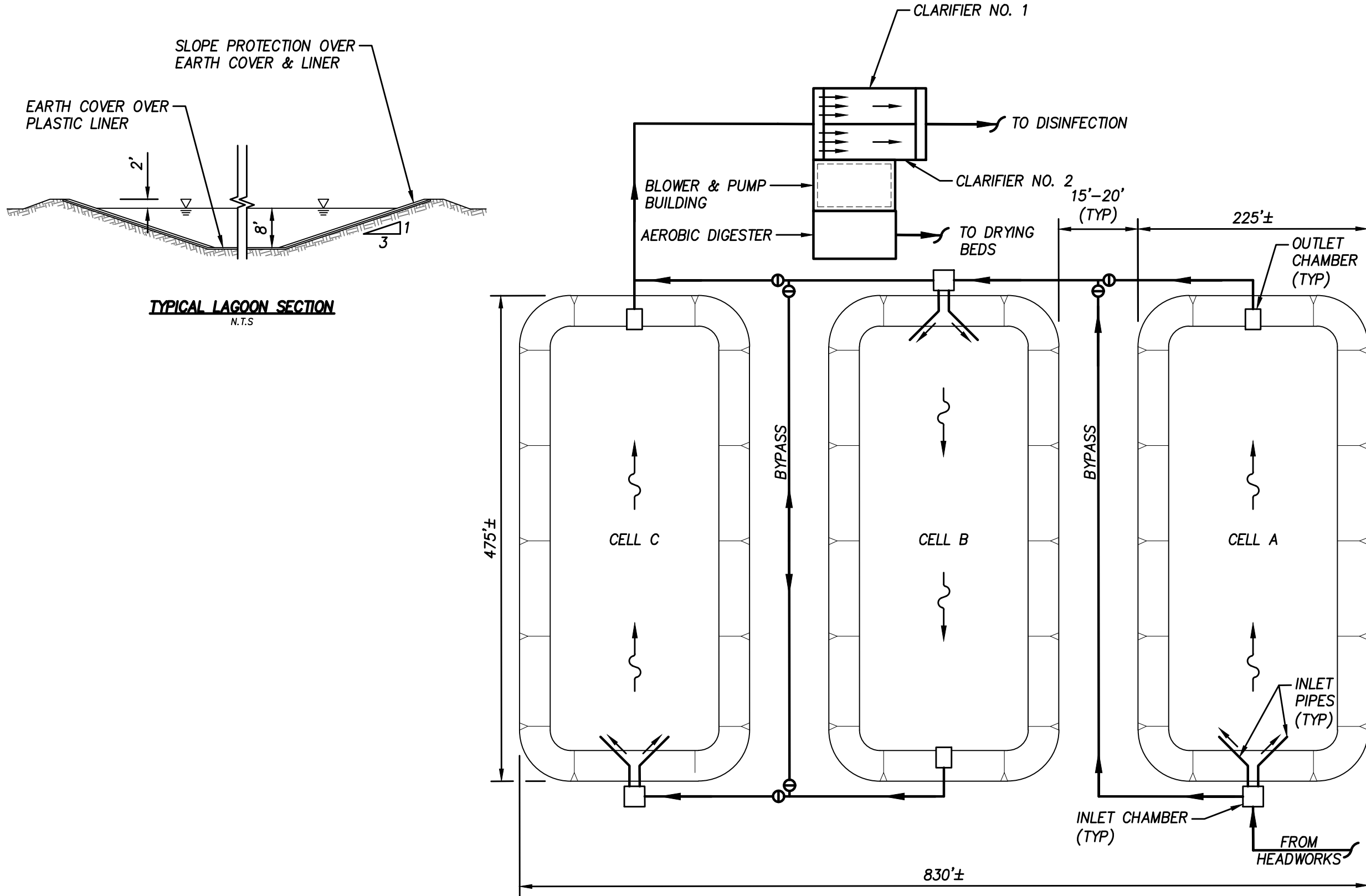
Alternative Two also has the potential operating benefit of allowing thickened biosolids to be pumped back to the digesters with the addition of return sludge piping. Circulating a portion of the digester contents through the thickener could effectively increase the volumetric capacity of the digesters by reducing water content. This practice would be more costly than decanting, but far more effective in removing water.

Overall, Alternative Two offers clear nonmonetary benefits that further establish the cost effectiveness of the biosolids thickening alternative.

6.7.5 *Recommended SWWTP Biosolids Alternative*

Given the clear monetary and nonmonetary benefits to adding a thickening process at the SWWTP, we recommend the City implement Alternative Two. Because the thickener will reduce O&M costs, we propose the City consider this a high-priority improvement.

The thickening process should be designed to accommodate the future addition of a dewatering process in case a more cost effective dewatering option is developed or other conditions change.



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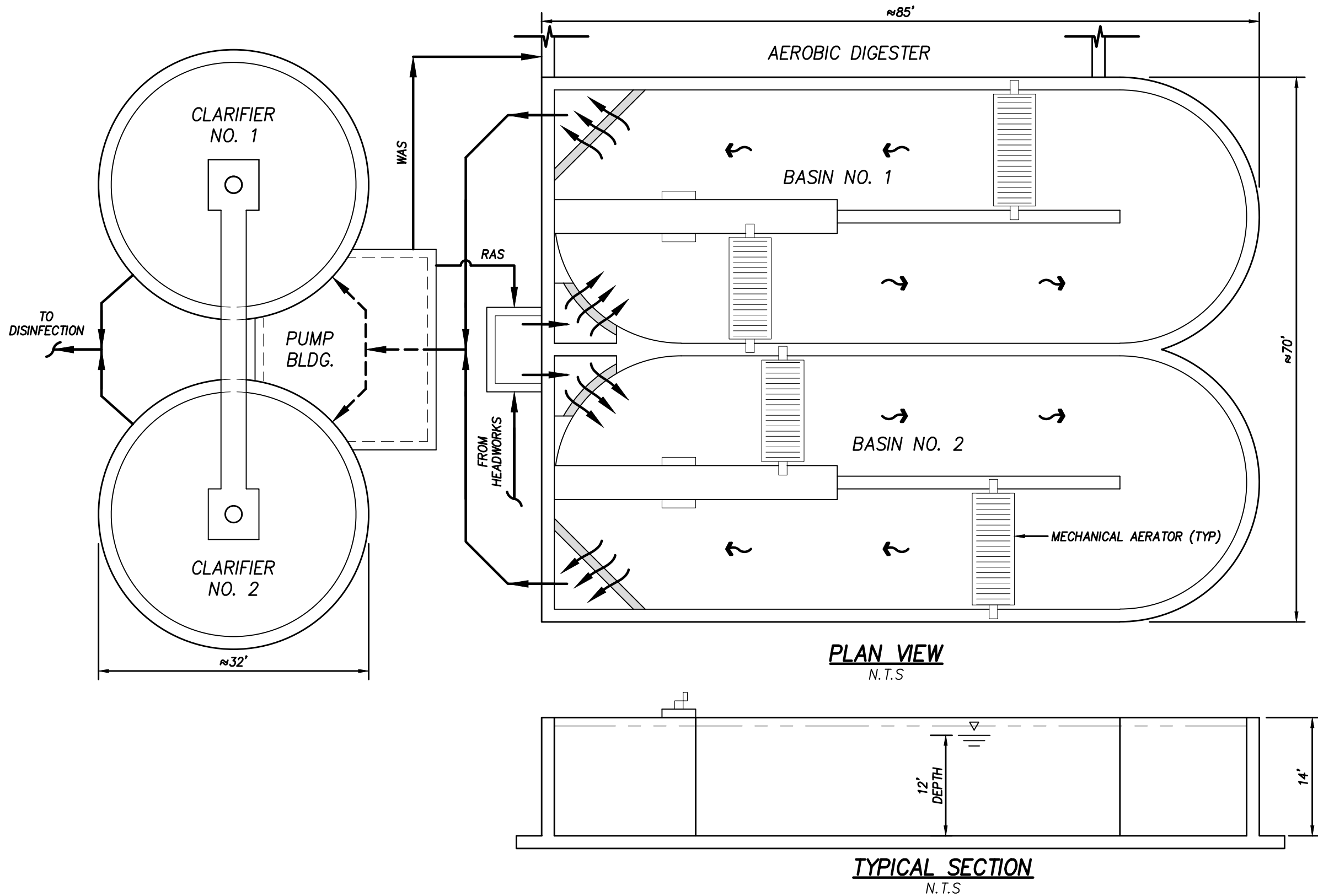
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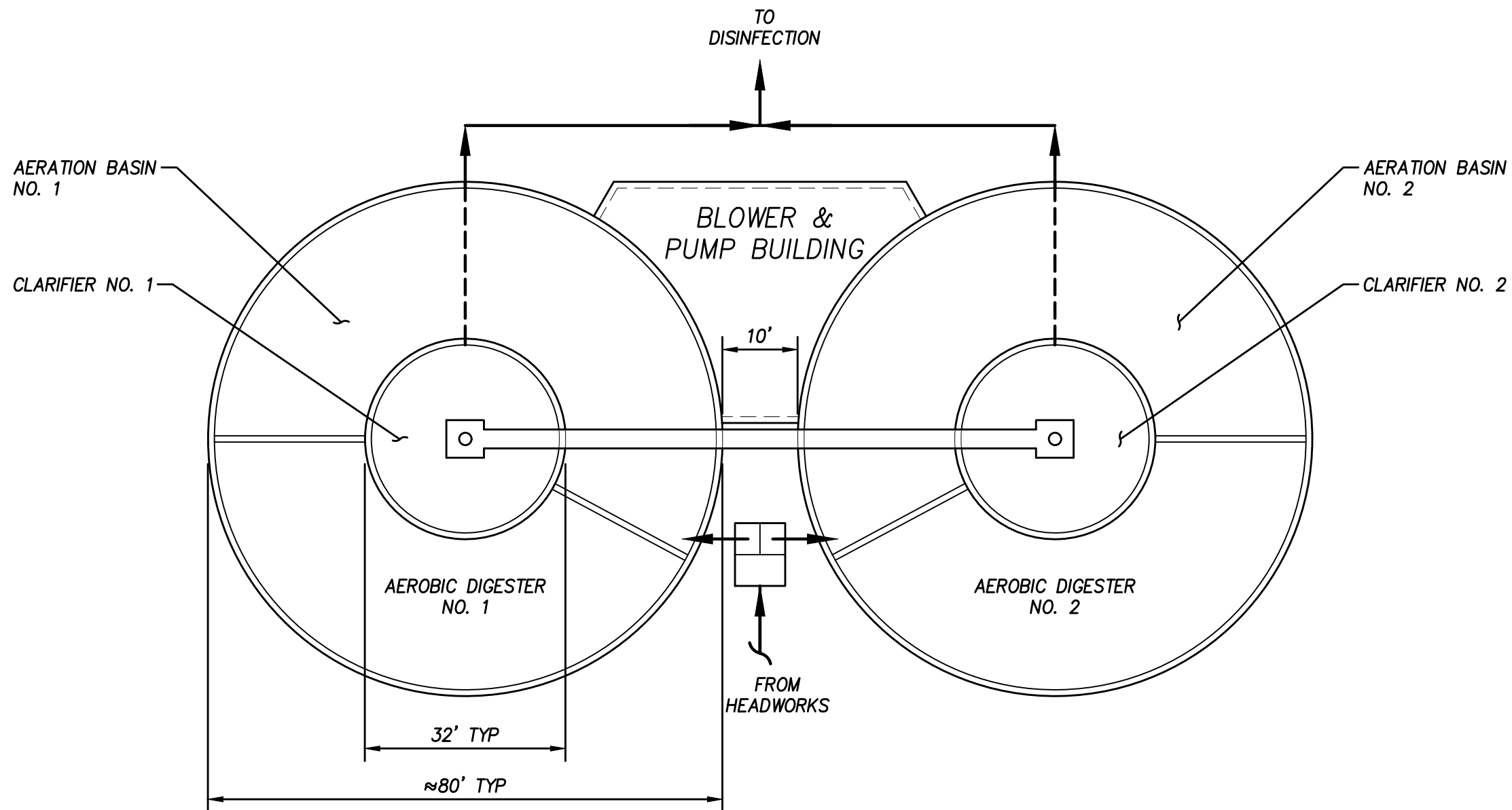
FIG 6-1

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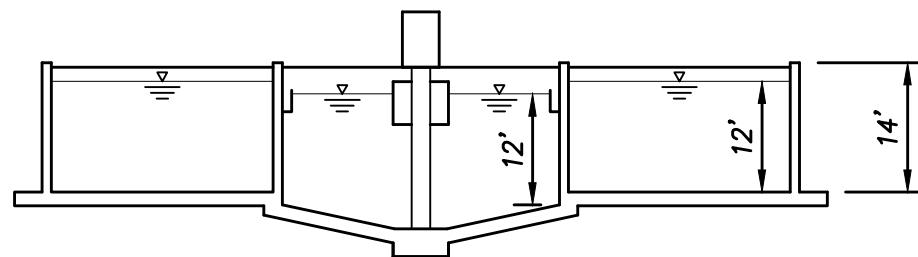
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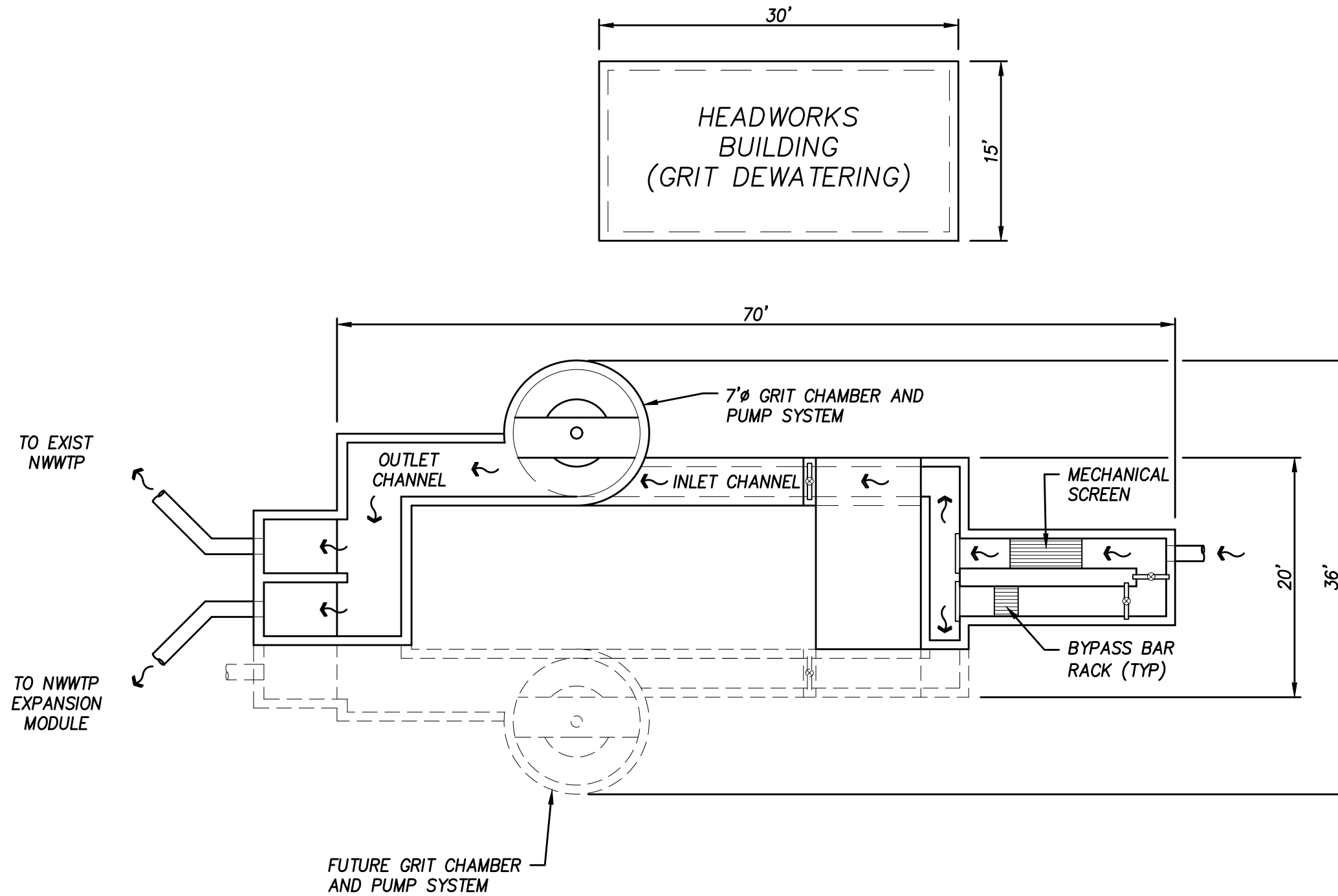
P:\MAD (City of Madras)\MAD-05 (WWS Master Plan Update)\MAD05 - DWG\SSIS\HEETS\Chan & MAD05-FIG 06-2.dwg



PLAN VIEW
N.T.S

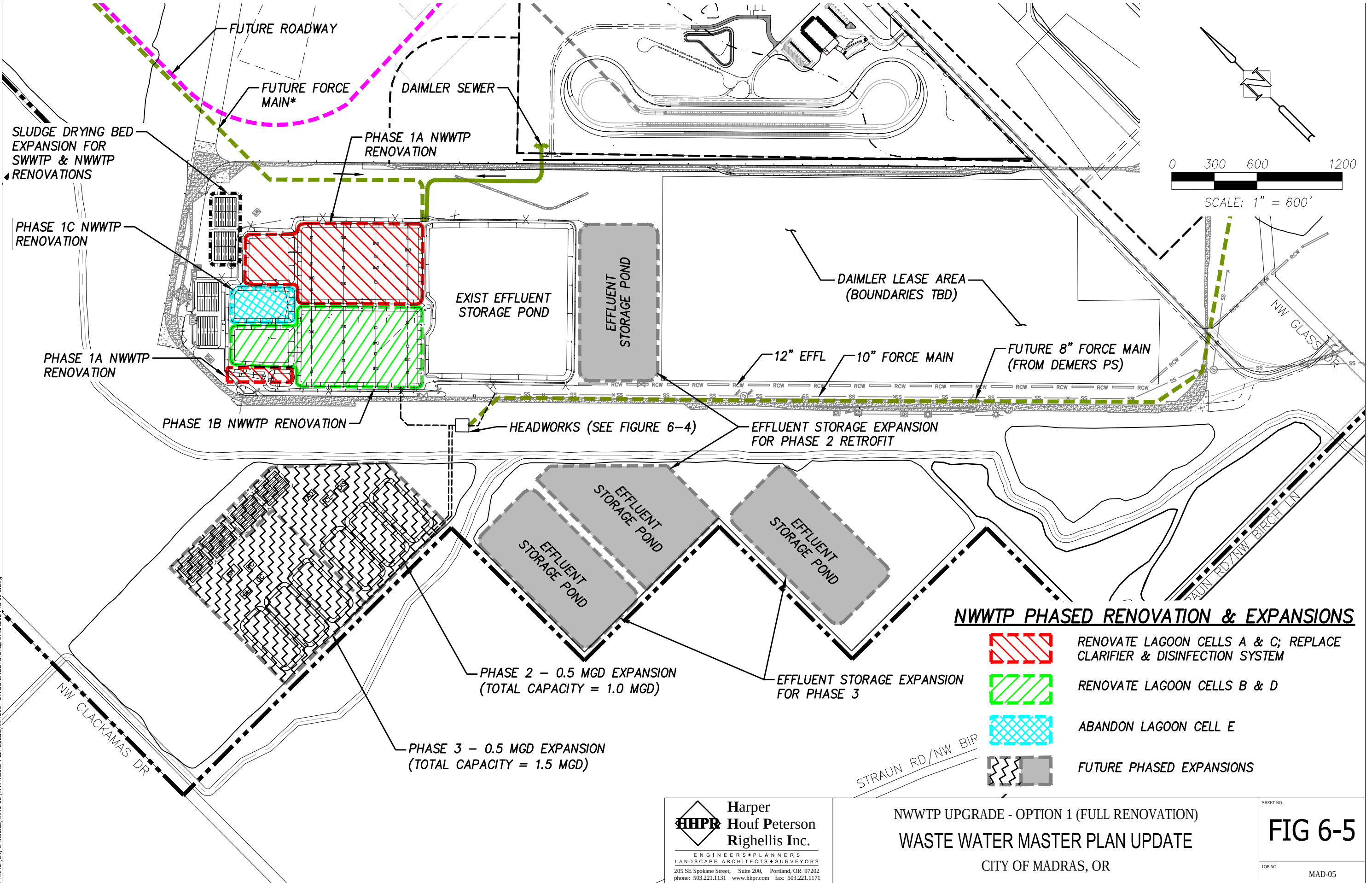


SECTION VIEW
N.T.S



PLAN VIEW
N.T.S

NOV 2017

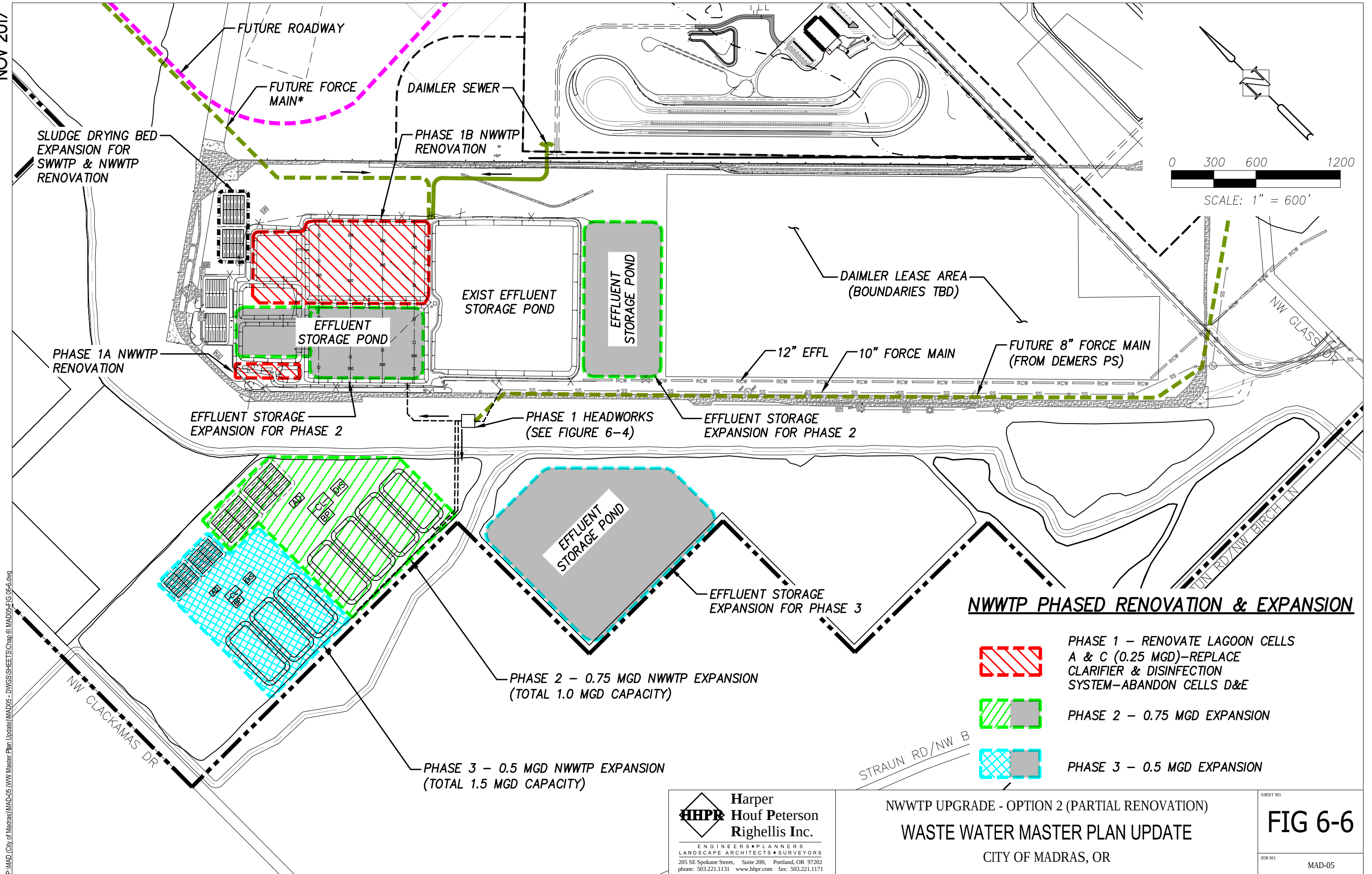


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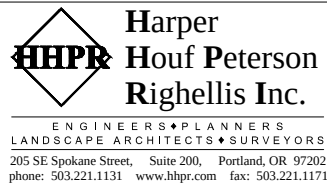
NWWTP UPGRADE - OPTION 1 (FULL RENOVATION)
WASTE WATER MASTER PLAN UPDATE
CITY OF MADRAS, OR

SHEET NO.
FIG 6-5
JOB NO.
MAD-05

NOV 2017

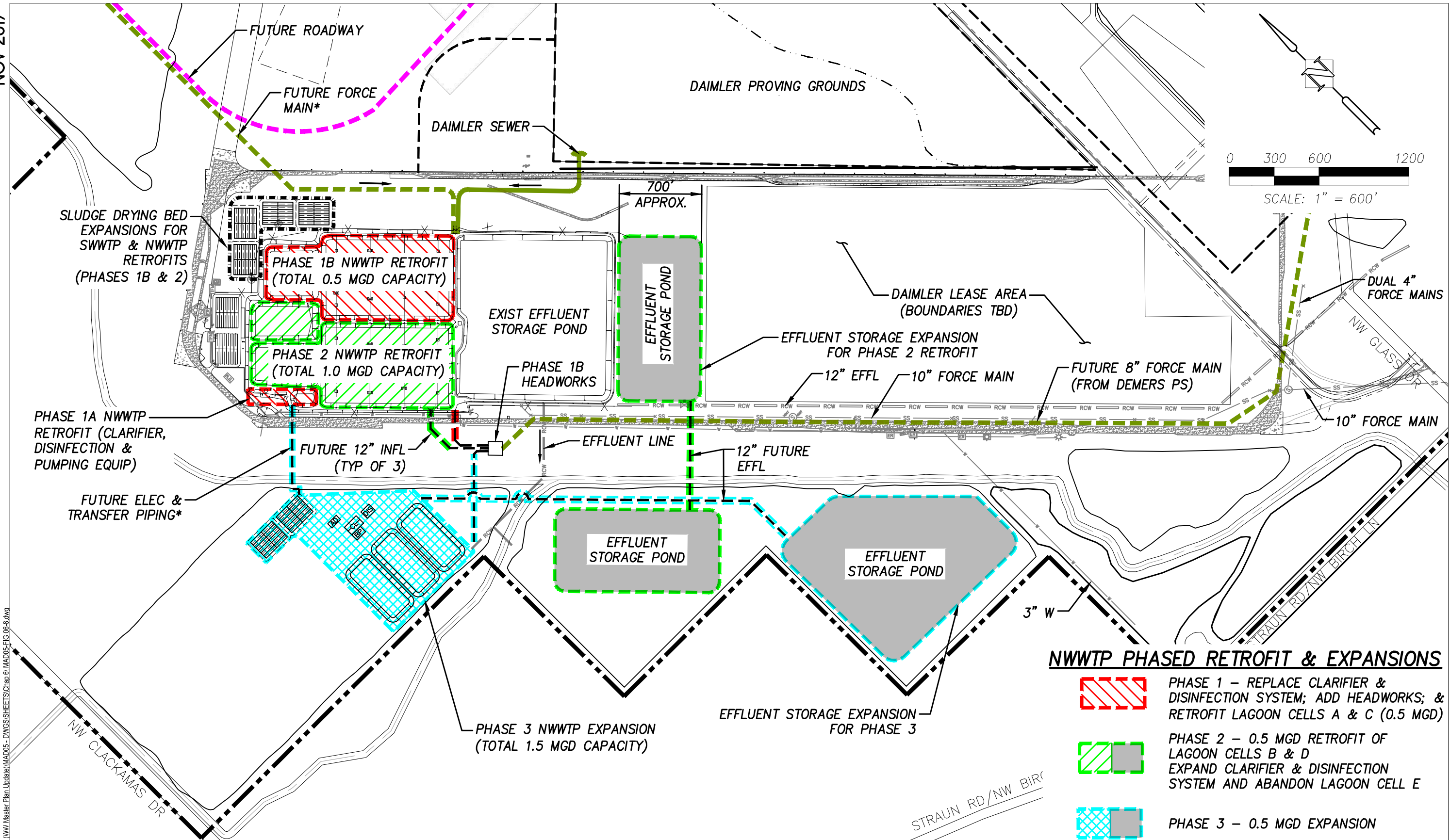


P:\MAD (City of Madras)\MAD-05 (WW Master Plan Update)\MAD05 - DWGS\SHEETS\Chap 6\ MAD05-FIG 06-7.dwg



SHEET NO.	FIG 6-7
JOB NO.	MAD-05

NOV 2017



NWWTP PHASED RETROFIT & EXPANSIONS

-  **PHASE 1 – REPLACE CLARIFIER & DISINFECTION SYSTEM; ADD HEADWORKS; & RETROFIT LAGOON CELLS A & C (0.5 MGD)**
-  **PHASE 2 – 0.5 MGD RETROFIT OF LAGOON CELLS B & D
EXPAND CLARIFIER & DISINFECTION SYSTEM AND ABANDON LAGOON CELL E**
-  **PHASE 3 – 0.5 MGD EXPANSION**

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NWWTP UPGRADE - OPTION 4 (RETROFIT TO EXPAND CAPACITY)
WASTE WATER MASTER PLAN UPDATE
CITY OF MADRAS, OR

SHEET NO.
FIG 6-8
JOB NO.
MAD-05

CHAPTER 7

EFFLUENT STORAGE AND RECYCLING

7.1 GENERAL

The City uses lined ponds for seasonal storage of effluent from each wastewater treatment plant (WWTP) and recycles the effluent through irrigation during the growing season. Effluent storage consists of a 79-million gallon (MG) pond at the North WWTP (NWWTP) site and two ponds near the South WWTP (SWWTP), an 88-MG pond about 1.8 miles east of the plant and a 25-MG pond at the plant site. Figures 7-1 and 7-2 show the locations of the existing storage ponds.

The City owns property near each WWTP that it leases as farmland and uses for effluent recycling. The municipal golf course also uses NWWTP effluent for irrigation. Effluent recycling generally supplements the use of irrigation water obtained from the North Unit Irrigation District (NUID) through existing water rights. Because this water recycling helps to conserve NUID irrigation water, the State considers recycling a beneficial use of the treated effluent produced at each WWTP.

We have developed estimates for future effluent storage and farmland requirements based on the assumption that the City will continue to recycle all effluent flows with no surface water discharge.

7.2 EFFLUENT STORAGE VOLUME REQUIREMENTS

Table 7-1 lists the effluent storage volumes needed for each incremental expansion described in Chapter 6 for the two WWTPs. The requirements are based on our estimate that a 79-MG volume is needed for each 0.5-MGD in average design capacity at either WWTP. Appendix M includes a water balance table and explanatory notes describing the basis for our estimate.

Table 7-1 Projected Storage Capacity Requirements for Both WWTPs		
Phase Description (Capacity)	Required Storage Volume (MG) ⁽¹⁾	Projected Surplus or Deficit (MG) ⁽²⁾
A. NWWTP		
Phase 1 Retrofit/Current Capacity (0.50 MGD)	79	0
Phase 2 – Expansion 1 (1.00 MGD)	158	-79
Phase 3 – Expansion 2 (1.50 MGD)	237	-158
B. SWWTP		
Current Capacity (0.54 MGD)	85	28
Phase 1 Expansion (0.65 MGD)	103	10
Phase 2 Expansion (1.15 MGD)	182	-69

NOTES:

- (1) Estimates of required storage volume are based on the following conditions:
 - Both the maximum annual precipitation (1983 Western Regional Climate Center, Madras Data) and minimum annual evaporation (1993 NUID Data) occur.
 - Average annual flows equal to 90% of WWTP design capacity (maximum monthly flow).
 - All effluent produced from October 10 through April 15 must be stored.
- (2) Surplus/deficit based on existing storage capacities of 79 MG at NWWTP and 113 MG at SWWTP.

7.3 FARMLAND REQUIREMENTS FOR EFFLUENT RECYCLING

7.3.1 *Existing Recycling Sites*

Table 7-2 summarizes the acreages of the irrigation sites the City can potentially use for recycling effluent from the NWWTP and Figure 7-1 shows the locations of these sites. The table also shows projections for the future available farmland when the NWWTP reaches the average design capacity of each proposed phase of improvements described in Chapter 6. These projections deduct lands where commercial/industrial developments would occur under current land-use plans or where NWWTP expansions are proposed.

Table 7-2 NWWTP Effluent Recycling Sites Current and Projected Availability				
Site Designation	Current Gross Site Area (AC)	Projected Net Usable Area (AC) ⁽¹⁾		
		Phase 1 Retrofit ⁽²⁾	Phase 2 Expansion ⁽³⁾	Phase 3 Expansion ⁽⁴⁾
Farm Field 6	41	37	37	37
Farm Field 7	12	11	11	11
Farm Field 8	34	31	31	20
Farm Field 9	38	34	20	--
Farm Field 11 ⁽⁵⁾	68	61	61	43
Farm Field 14	53	---	---	---
Farm Fields 21 & 22 ⁽⁶⁾	---	---	---	---
Municipal Golf Course ⁽⁷⁾	75	71	114	114
Total Area	321	245	274	225
Farmland Area	246	174	160	111

NOTES:

- (1) Net usable area assumes 90% of farmland and 95% of golf course are available for effluent recycling each season. This assumes the remaining land would be used as buffer, left fallow, or used for crops not suitable for effluent.
- (2) Phase 1 Projection (0.5-MGD Retrofit): deduct Field 14 due to airport development.
- (3) Phase 2 Projection (1.0-MGD): deduct 16 gross acres for expansion of effluent storage capacity (part of Field 9). Net usable farmland deducted = 62 acres.
- (4) Phase 3 Projection (1.5-MGD): deduct 55 gross acres for NWWTP expansion (remainder of Field 9 and parts of Fields 8 & 11). Net usable farmland deducted = 49 acres.
- (5) Field 11 is currently leased for farming, but is not irrigated with effluent.
- (6) Fields 21 and 22 are no longer available for effluent recycling due to developments that have cut off routes for effluent transmission piping.
- (7) Planned golf course expansion would result in a 43-acre increase in net usable land at golf course. We assume the expansion would occur before NWWTP flows reach the Phase 2 capacity.

Tables 7-3 summarizes the acreages of the irrigation sites the City can potentially use for recycling effluent from the SWWTP and Figure 7-2 shows the locations of these sites. Development plans for the “Upper Parcel” call for the construction of a golf course that would be irrigated with recycled water in the future. If the golf course is not developed before the City needs more land for effluent recycling, this site could be used in the interim as farmland for irrigation opportunities.

Table 7-3 SWWTP Effluent Recycling Sites Current and Future Farmland Availability		
Site Designation	Areas of Dedicated Sites (AC)	
	Total Site Area	Net Usable Area ⁽¹⁾
Existing Woodworth Farmland	76	68
Existing Deschutes Basin Farms	35	32
Current Available Farmland	111	100
Addition of Upper Parcel ⁽²⁾	193	174
Total Future Farmland Area	304	274

NOTES:

- (1) Net usable area assumes 90% of total farmland is available for effluent recycling each season. This assumes the remaining land would be used as buffer, left fallow, or used for crop not suitable for effluent.
- (2) Upper parcel, east of SWWTP, originally obtained for future golf course construction. Our analysis assumes the City would use the site as farmland for effluent recycling until golf course development occurs.

7.3.2 Projected Recycling Land Requirements

To address future recycling needs, we estimated the amount of farmland required for future effluent volumes when the WWTP expansions described in Chapter 6 are implemented. The estimates are based on information presented in the previously-prepared effluent reuse plans (CH2M Hill, 1993, and ACE Consultants, 2007). We also updated the information in these reuse plans based on input from the City regarding effluent uses. Our calculations for the NWWTP deduct effluent volume used to satisfy about 25% of the irrigation demand for the municipal golf course.

Tables 7-4 and 7-5 (end of chapter) summarize farmland requirements for recycling effluent from each WWTP when peak-monthly flows reach the capacity of each proposed WWTP improvement. The tables also show estimates of farmland required at current flow rates and Table 7-5 includes an estimate for conditions when the Deer Ridge facility would contribute its full flow allowance.

Key assumptions used in developing our estimates are described in the notes following the tables. The irrigation usage was estimated using water demand for alfalfa because the intermediate irrigation requirements for this crop are representative of average conditions. Grass crops generally have higher water demands and grains generally require lesser amounts.

A higher irrigation efficiency has been assumed for farmland around the airport because the shallow depth to rock may reduce the water losses through infiltration below the top soil with crop roots. The higher efficiency means less water can be applied to meet the crop demands.

Future farmland requirements for NWWTP effluent would be reduced relative to Table 7-4, if more than 25% of the water demand for the golf course can be met using recycled water. Also, there would be a smaller farmland shortfall for SWWTP effluent in the future relative to Table 7-5, if a golf course is developed as planned on the available parcel. This is because water demands for turf grass would be higher than those for the reference crop of alfalfa. Appendix M includes additional tables and explanations that further describe the basis for the acreage estimates.

7.4 SUMMARY

NWWTP Effluent Storage. The City will need to expand effluent storage volume in conjunction with a NWWTP expansion above current capacity. We estimate a 79-MG expansion of effluent storage capacity would be required with each proposed 0.5-MGD expansion of average capacity. Figure 6-8 identifies potential sites for the additional storage ponds near the NWWTP site.

NWWTP Farmland. As shown in Table 7-4, we project the NWWTP would have a farmland deficit of approximately 37 acres when peak monthly flows reach 0.50 MGD. This estimate accounts for the loss of fields in the airport area due to potential development. Existing irrigable lands might meet the City's needs at this flow rate, if more of the golf-course water demand were met using recycled water. However, the City should plan to identify and purchase at least 40 more acres of farmland to use for irrigation within the planning period.

Our projections also show the NWWTP site would face a farmland deficit of approximately 300 acres and 610 acres for design flows of 1.0 MGD and 1.5 MGD, respectively. These deficits would also be less, if more of the golf-course water demand can be met with recycled effluent.

SWWTP Effluent Storage. Our estimate shows there is enough effluent storage volume for a design capacity up to about 0.70 MGD at the SWWTP. Therefore, no expansion of storage capacity would be required for the Phase One expansion. However, an additional 70-MG storage pond would be required in conjunction with an expansion to 1.15 MGD. The existing site of the upper storage pond for the SWWTP has room for the future addition of another pond.

SWWTP Farmland. We estimate the existing 111 acres of total available farmland for effluent recycling can accommodate average annual flows up to about 0.24 MGD at the SWWTP. As a result, the City should plan to make the future golf course parcel available for irrigation in 5 years or sooner, if flow contributions from Deer Ridge increase significantly. Since the timing of the future golf course development is uncertain, we have included a project in the capital improvements plan (Chapter 8) to provide an irrigation system for that site to allow it to be used as farmland.

As shown in Table 7-5, we estimate the City would need to add about 125 acres of the upper parcel as farmland to handle an average annual flow equal to the current SWWTP capacity of 0.54 MGD. If the full 193-acre site is made available as farmland, there would be a surplus of 28 acres when flows reach the 0.65-MGD design capacity of the proposed Phase 1 expansion.

According to our projections, there would be a farmland deficit of almost 180 acres when flows reach the Phase 2 capacity of 1.15 MGD. This deficit could be reduced, if the future golf course is developed as planned and effluent is used to meet most or all of the irrigation demand.

Probable Costs. Chapter 8 provides estimates of probable costs for expanding effluent storage capacity as part of WWTP expansions. Chapter 8 also includes probable costs for purchasing additional farmland near the NWWTP and conveying recycled effluent to this property.

Table 7-4 NWWTP Effluent Recycling – Summary of Projected Farmland Requirements						
Phase Description	Avg. Design Flow (MGD) (Note 1)	Projected Avg. Annual Flow (MGD) (Note 2)	Net Recycled Water Volume (MG) (Note 3)	Total Farmland Requirements (Acres)		
				Farmland Required (Note 4)	Net Available Farmland (Note 5)	Projected Land Surplus or Deficit
Current Conditions	--	0.31	81	130	222	92
Phase 1 - Retrofit	0.50	0.45	132	211	174	-37
Phase 2 - Expansion 1	1.00	0.90	285	457	160	-297
Phase 3 - Expansion 2	1.50	1.35	449	721	111	-610

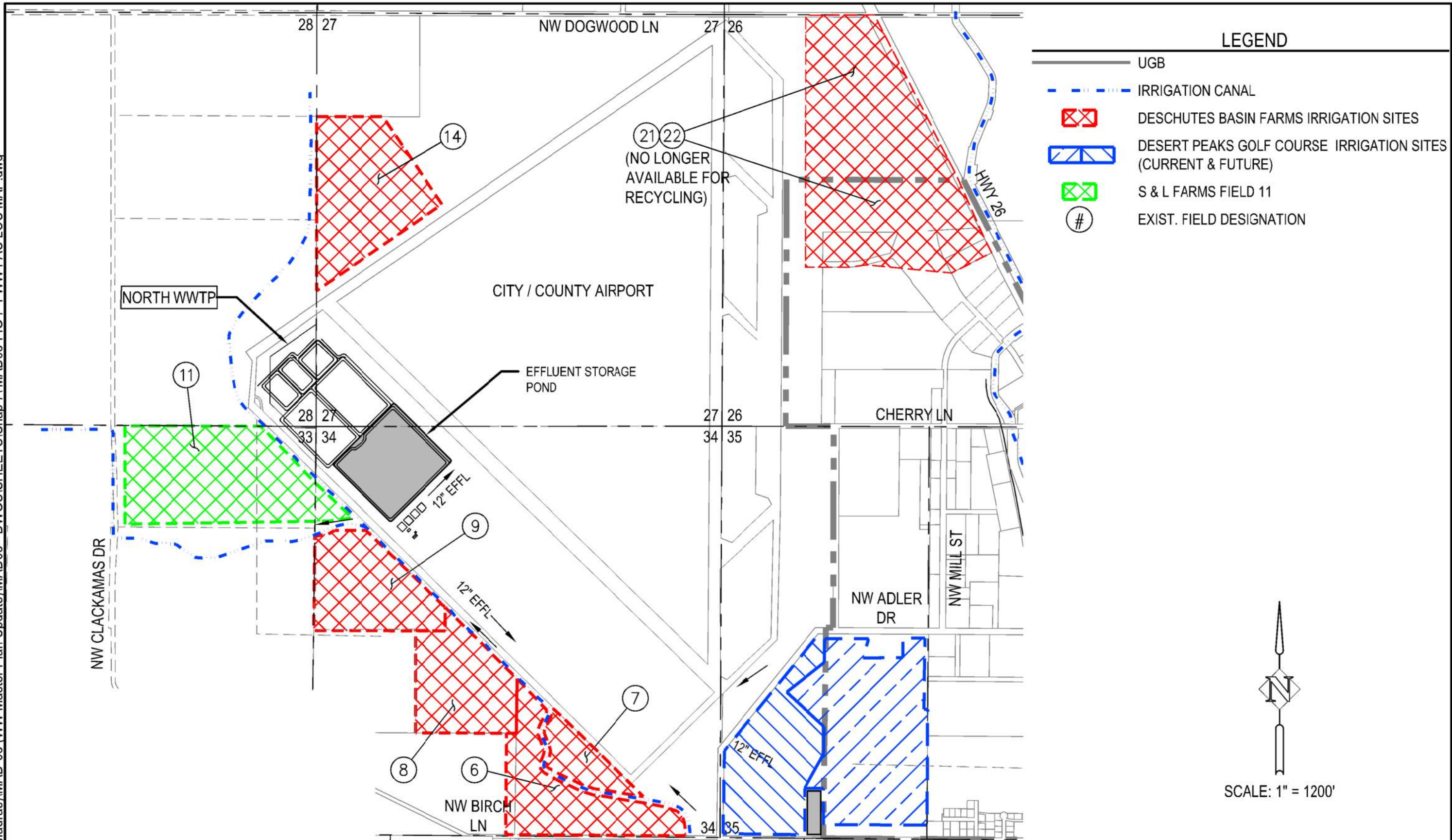
NOTES:

- (1) Average Design Flow = Projected Peak Monthly Flow.
- (2) Average Annual Flow = 90% of Peak Monthly Flow.
- (3) Recycled Water Volume = Net Available Water (Treated Effluent + Stored Precipitation - Pond Evaporation - Golf Course Irrigation).
 - a. Assume maximum year precipitation (1983 Western Regional Climate Center, Madras Data).
 - b. Assume minimum year evaporation (1993 NUID Data).
 - c. Effluent use at Municipal Golf Course assumes 25% of gross demand for turf grass. NUID water would be used for the remaining water demand.
 - d. Assume future Municipal Golf Course expansion increases acreage by 60% and occurs before NWWTP flows reach 1.0 MGD (Phase 2 capacity).
- (4) Required farmland based on average, net irrigation demand for alfalfa.
 - a. Assume irrigation efficiency = 85% .
 - b. Net irrigation demand estimated to cover 80% of gross water demand for alfalfa satisfied by recycled effluent (remainder from NUID).
- (5) Net available farmland is equal to projected available acreages shown in Table 7-2.
 - a. Assume 90% of total acreage available for effluent application (remainder left fallow or used to grow food crop).
 - b. Current available farmland assumed to be 80% of remaining fields in airport area (Fields 6, 7, 8, 9, 11, & 14). Fields 21 & 22 no longer available.
 - c. Phase 1: Subtract Field 14.
 - d. Phase 2: Subtract an additional 16 acres of land for one Phase 2 effluent storage pond (part of Field 9, in addition to Phase 1 subtraction).
 - e. Phase 3: Subtract another 55 acres of land for Ph. 3 WWTP module & effluent ponds (parts of Fields 8 & 11 & remainder of Field 9).
Also subtract an additional 120 acres of land for development of airport land (Field 11 & remainder of Fields 21 & 22).

Table 7-5 SWWTP Effluent Recycling – Summary of Projected Farmland Requirements						
Phase Description	Avg. Design Flow ⁽¹⁾ (MGD)	Projected Avg. Annual Flow ⁽²⁾ (MGD)	Net Recycled Water Volume ⁽³⁾ (MG)	Total Farmland Requirements (Acres)		
				Farmland Required ⁽⁴⁾	Net Available Farmland ⁽⁵⁾	Projected Land Surplus or Deficit
Approximate Current Flow	--	0.22	66	88	100	12
Full Prison & Current Flows	--	0.40	131	177	100	-77
Current SWWTP Capacity	0.54	0.45	150	201	274	73
Phase 1 Expansion	0.65	0.54	183	246	274	28
Phase 2 Expansion	1.15	0.86	335	450	274	-176

NOTES:

- (1) Average Design Flow = Projected Peak Monthly Flow.
- (2) Average Annual Flow = 90% of Peak Monthly Flow.
- (3) Recycled Water Volume = Net Available Water (Treated Effluent + Stored Precipitation - Pond Evaporation - Golf Course Irrigation).
 - a. Assume maximum year precipitation (1983 Western Regional Climate Center, Madras Data).
 - b. Assume minimum year evaporation (1993 NUID Data).
- (4) Required farmland based on average, net irrigation demand for alfalfa.
 - a. Assume irrigation efficiency = 80% .
 - b. Net irrigation demand assumes 90% of alfalfa gross water demand satisfied by recycled effluent (remainder from NUID).
- (5) Net available farmland is equal to projected available acreages shown in Table 7-3.
 - a. Assume 90% of total acreage available for effluent application (remainder left fallow or used to grow food crop).
 - b. Conditions listed for current flow and full-prison flow include area of Woodworth and McTaggart lots.
 - c. Listings for flows at current, Phase 1, and Phase 2 capacities include Woodworth, McTaggart and future Golf Course lands.



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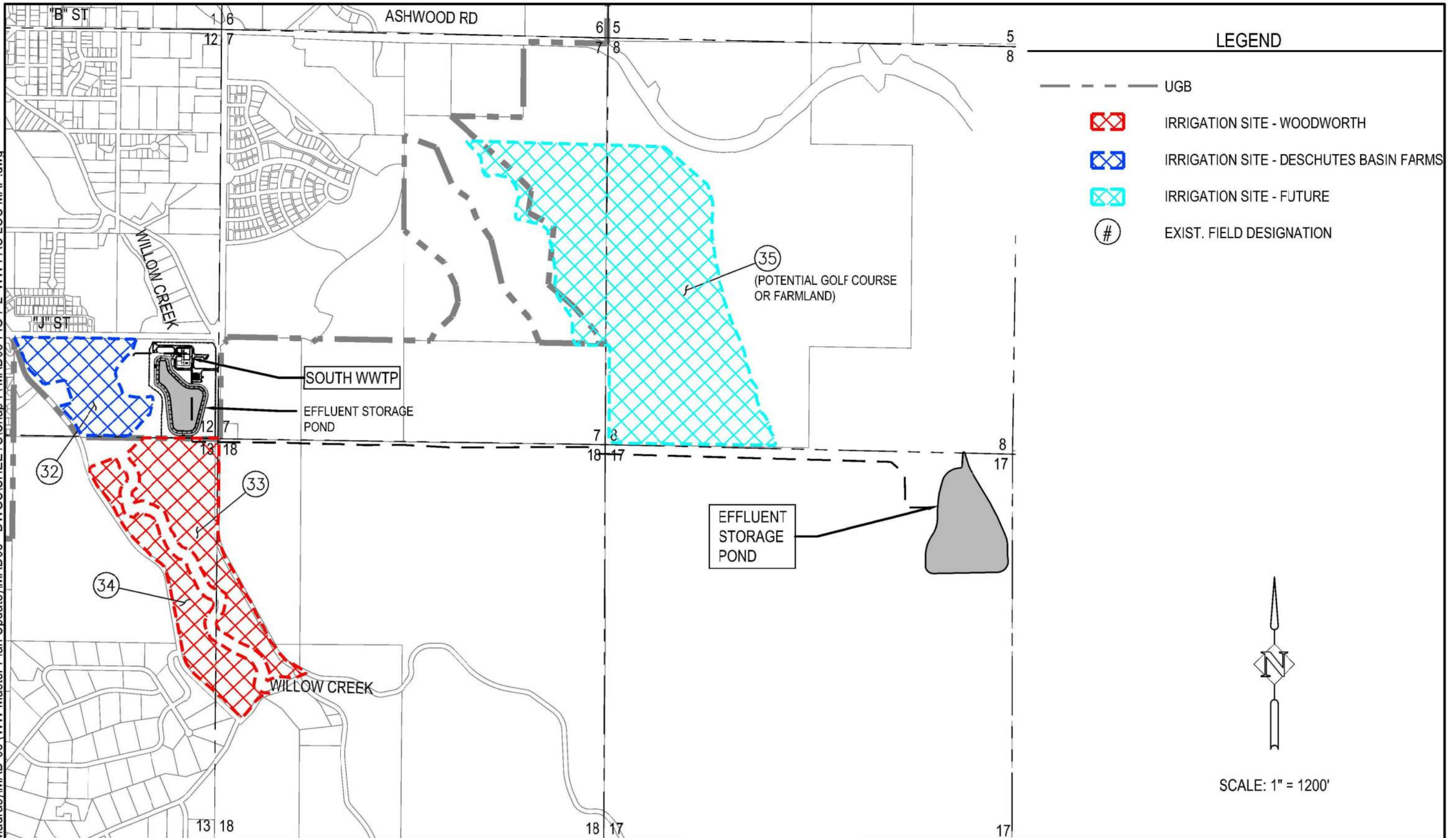
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NWWTP EFFLUENT STORAGE & RECYCLING SITES
WASTEWATER MASTER PLAN UPDATE
CITY OF MADRAS, OREGON

SHEET NO:
7-1
JOB NO:
MAD-05

NOV 2017

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SWWTP EFFLUENT STORAGE & RECYCLING SITES WASTEWATER MASTER PLAN UPDATE CITY OF MADRAS, OREGON		SHEET NO. 7-2
		JOB NO. MAD-05

CHAPTER 8

RECOMMENDED IMPROVEMENTS AND PLANNING CONSIDERATIONS

8.1 PUBLIC INVOLVEMENT

Public meetings were held on January 26, 2017 and November 29, 2017 at Madras City Hall to present the findings and initial recommendations of the 2016 Wastewater Master Plan (WWMP) Update and to solicit public comments. The main elements of the meeting presentations are briefly summarized below. Appendix N includes the meeting minutes for these meetings.

1. Reasons for preparing the WWMP Update.
2. Briefly describe the existing wastewater (WW) facilities serving the City.
3. Provide an overview of the WW facilities condition assessments.
4. Describe the population forecasts and WW flow projections used as the basis of planning.
5. Briefly summarize the collection system analysis and identify the recommended collection system improvements.
6. Briefly summarize the WW treatment alternatives evaluation and identify the recommended wastewater treatment plant (WWTP) improvements.

Questions and comments were received on the proposed improvements and related issues from individuals at the meeting. In response to questions and discussions during the meeting, additional information on the following subjects has been incorporated into this WWMP Update.

1. Regulations governing surface-water discharges of WWTP effluent (Chapter 1).
2. Key issues that affect the potential for implementing constructed wetlands and a surface-water discharge at the City's South WWTP (Chapter 5).
3. Projected farmland requirements for effluent recycling (Chapter 7).

8.2 CAPITAL IMPROVEMENTS PLAN

8.2.1 *General*

This section presents the capital improvements plan (CIP) for the recommended WW improvements identified in Chapters 4, 6 and 7. To facilitate planning, we list the proposed projects according to the anticipated time periods when the improvements will most likely be needed. The actual implementation schedule will depend on various factors, including those listed below:

- the availability of funds for capital improvements;
- the actual rates of population growth and commercial/industrial development;
- ongoing changes in the conditions & adequacy of major components; and
- future changes in regulations affecting the WW facilities.

Potential improvements identified for industrial site readiness (ISR) planning must be reevaluated based on actual development proposal(s). Project timing and scope will be development dependent.

8.2.2 *Basis of Estimates of Probable Costs*

We have developed the preliminary estimates of probable costs presented in this WWMP from information available at the time this study was prepared. The cost information used to generate the estimates was current to December 2016. The estimates were subsequently updated to December 2017 using the Engineering News-Record Construction Cost Index of 10,873.

The probable construction costs developed for this study are planning-level estimates and their level of detail falls within Estimate Class 4 as defined by the Association for the Advancement of Cost Engineering International (Recommended Practice #18R-97, Rev. March 2016). Consistent with this estimate class, the accuracy is anticipated to be within +35% to –20% of the actual cost.

Allowances for contractor overhead and profit, mobilization/demobilization, and construction contingencies are included to provide conservative estimates. A nonconstruction cost factor was also applied to each project to include an allowance for planning, engineering, and administrative costs. Because the estimates are preliminary, we used a 25% factor for construction contingences and a 35% factor for nonconstruction costs.

Actual project costs will depend on the scope identified during project development. Actual construction costs will also be affected by labor and material costs and competitive market conditions at the time bids are solicited, as well as by specific site conditions and other factors. Consequently, the final construction costs will vary from our estimates.

8.2.3 *Collection System Improvements – Gravity Sewers*

The recommended sewer improvements are listed below according to the general timing we anticipate for their implementation. Table 8-1 (following page) summarizes the estimates of probable project costs for the gravity sewers CIP.

A. Short-Term Projects (within 5 years)

1. **Mountain View/Sky Ridge Sewers** – install approximately 4,500 LF of new 8-inch sewers in a currently unserved area.
2. **Jefferson Street/North Unit Sewers** – install approximately 2,900 LF of new 8-inch sewers in a currently unserved area.
3. **Bel Air/Herzberg Heights Sewers** – install approximately 5,100 LF of new 8-inch sewers in pockets of unserved areas.

B. Mid-Term Projects (6 – 10 years)

1. **Hess Street Sewer (Industrial Park)** – install approximately 1,670 LF of new 8-inch sewer in Industrial Park.
2. **Mill Street Sewer (Industrial Park)** – install approximately 1,430 LF of new 8-inch sewer in Industrial Park.

C. Mid- to Long-Term Project (11 – 15 years)

1. **North Y Sewer** – replace approximately 2,060 linear feet (LF) of existing 8-inch sewer with 12-inch sewer by pipe bursting to increase capacity.
2. **Culver Hwy. Parallel Sewer** – install approximately 3,100 LF of new 10-inch parallel sewer to increase capacity.

D. Long-Term Projects (16 – 20 years)

1. **Juniper Heights Sewers** – install approximately 5,350 LF of new 8-inch sewers in currently unserved areas.
2. **Willow Creek North Sewer** – install approximately 2,500 LF of new 8-inch sewer to serve new developments.

E. Potential Project for Major Industrial Park Developments (Industrial Site Readiness)

Our analysis identified alternative sewer improvements that would potentially be needed to serve major Industrial Park developments (Chapter 4). To support ISR planning, Table 8-1 lists the potential North Area-Airport Way parallel sewer project as the least-cost alternative for providing sewer service to a major user. Actual sewer project(s) required for major industrial users will need to be evaluated based on development proposal(s).

Table 8-1		
Sewer Improvements – Estimated Timing and Probable Costs (December 2017 Dollars)		
Project Description	Probable Construction Cost	Probable Project Cost
A. Short-Term Projects (within 5 years)		
1. Mountain View/Sky Ridge Sewer Extensions	\$871,000	\$1,175,000
2. Jefferson Street/North Unit	\$412,000	\$555,000
3. Bel Air/Herzberg Heights	\$856,000	\$1,155,000
Subtotal for Short-Term Projects	\$2,139,000	\$2,885,000
B. Mid-Term Projects (6 – 10 years)		
1. Hess Street Sewer	\$258,000	\$350,000
2. Mill Street Sewer	\$222,000	\$300,000
Subtotal for Mid-Term Projects	\$480,000	\$650,000
C. Mid- to Long-Term Project (11 – 15 years)		
1. North Y Sewer Replacement (By Pipe Bursting)	\$652,000	\$880,000
2. Culver Hwy. Parallel Sewer	\$561,000	\$760,000
Subtotal for Mid- to Long-Term Project	\$1,213,000	\$1,640,000
D. Long-Term Projects (16 – 20 years)		
1. Juniper Heights Sewer Extension	\$1,036,000	\$1,400,000
2. Willow Creek North Sewer Extension	\$419,000	\$565,000
Subtotal for Long-Term Projects	\$1,455,000	\$1,965,000
F. Potential ISR Project (Scope and Timing Are Development Dependent)		
North Area-Airport Way Parallel Sewer	\$695,000	\$940,000
Subtotal – Potential ISR Project	\$695,000	\$940,000
Total – Proposed & Potential Sewer Projects	\$5,982,000	\$8,080,000

8.2.4 Collection System Improvements – Pump Stations and Force Mains

The recommended improvements to the collection-system pump stations and force mains are listed below according to the general timing we anticipate for their implementation. Table 8-2 (following page) summarizes the estimates of probable project costs for the PS and force mains CIP.

A. Short-Term Project (within 5 years)

“B” Street North PS Screen – add an influent mechanical screen to prevent pump clogging and increase pump service life, with no expansion of PS capacity.

B. Mid-Term Project (6 – 10 years)

Golf Course PS – replace existing PS and expand peak design capacity to 0.21 MGD.

C. Mid- to Long-Term Projects (11 – 15 years)

1. **Renovation of “B” Street North PS** with no expansion of existing capacity.
2. **Generator Set Replacement for “B” Street North and South Pump Stations** including minor building renovations.
3. **South U.S. 97 PS Renovation** with no expansion of existing capacity.

D. Long-Term Project (16 – 20 years)

1. **Demers Pump Station (PS)** – replace existing PS with a duplex, submersible PS. Design for future expansion of peak design capacity to 1.33 million gallons per day (MGD).
2. **Expansion and renovation of “B” Street South PS** – expand existing PS to peak design capacity of 2.66 MGD.

E. Potential Projects for Future Industrial Park Developments (ISR)

1. **Demers PS Force Main Replacement** – install 11,000 LF of 8-inch force main to replace existing force mains with any future expansion of Demers PS above a 0.70 MGD capacity.
2. **Airport Area PS and Force Main** – construct new PS with 1.33-MGD capacity to serve airport area and install new force main extending from PS to North WWTP.

Table 8-2		
Pump Station Improvements – Estimated Timing and Probable Costs (December 2017 Dollars)		
Project Description	Probable Construction Cost	Probable Project Cost
A. Short-Term Project (within 5 years)		
Influent Screen at “B” Street North PS – No expansion	\$345,000	\$465,000
Subtotal for Mid-Term Project	\$345,000	\$465,000
B. Mid-Term Project (6 – 10 years)		
Golf Course PS Replacement – Expanded Capacity	\$361,000	\$490,000
Subtotal for Mid-Term Project	\$361,000	\$490,000
C. Mid- to Long-Term Projects (11 – 15 years)		
1. “B” Street North PS Renovation – No expansion	\$452,000	\$610,000
2. “B” Street North & South Generator Replacement	\$181,000	\$245,000
3. South U.S. 97 PS Renovation – No expansion	\$45,000	\$60,000
Subtotal – Mid- to Long-Term Projects	\$678,000	\$915,000
D. Long-Term Project (16 – 20 years)		
1. “B” Street South PS Expansion & Renovation	\$542,000	\$730,000
2. Demers PS Replacement – Expanded Capacity	\$1,327,000	\$1,790,000
Subtotal – Long-Term Projects	\$1,869,000	\$2,520,000
E. Potential ISR Project (Scope and Timing Development Dependent)		
1. Demers Force Main Replacement – 8” Pipe	\$1,174,000	\$1,585,000
2. Airport Area PS and Force Main	\$1,735,000	\$2,340,000
Subtotal – Potential ISR Project	\$2,909,000	\$3,925,000
Total – Proposed & Potential PS Projects	\$6,162,000	\$8,315,000

8.2.5 Wastewater Treatment Plants, Effluent Storage and Effluent Recycling

The recommended improvements to the City's WWTPs and effluent handling systems are listed below according to the general timing we anticipate for project implementation. Table 8-3 (following page) summarizes the estimates of probable project costs for the WWTPs CIP.

The WWTP CIP includes expansions of effluent storage, transmission, and irrigation facilities since these projects would be required in conjunction with treatment expansions. Estimated farmland acreages for irrigation expansions include a 10% allowance for buffers and access.

A. Short-Term Project (within 5 years)

1. **North WWTP (NWWTP) Phase 1A Improvements:** Replace the existing clarifier and disinfection equipment, the in-plant pumps, and the associated valves and piping. Construct a new building to house the clarifier to allow reliable year-round operation.

** **Alternate NWWTP Phase 1A Improvements:** Complete limited upgrade within 3 years to allow reliable year-round operation in the near term (construct a new building to house the existing clarifier, protect exposed in-plant pumps, rebuild in-plant pumps, replace associated valves, refurbish clarifier equipment, and modify piping).
2. **South WWTP (SWWTP) Thickening System:** Install a biosolids thickening system and associated improvements.

B. Mid-Term Project (6 – 10 years)

1. **SWWTP Phase 1 Expansion and Partial Renovation:** Modify the existing plant to expand average design flow capacity to 0.65 MGD.
2. **Irrigation System Expansion for SWWTP Effluent Recycling:** Install an irrigation system at the 193-acre Upper Parcel to prepare the site for effluent recycling on crops.

C. Mid- to Long-Term Projects (11 – 15 years)

1. **NWWTP Phase 1B Improvements:** Retrofit the existing lagoons by constructing an aerated lagoon system and related facilities. Maintain the current average design capacity of 0.50-MGD.
2. **Irrigation System Expansion for NWWTP Effluent Recycling:** Purchase 42 acres of farmland, install an effluent transmission pipeline to the additional land, and install an irrigation system at the new site.

D. Long-Term Project (16 – 20 years)

1. **SWWTP Phase 2 Expansion/Renovation:** Construct a 0.5-MGD treatment module and renovate existing facilities to expand average design flow capacity to 1.15 MGD.
2. **SWWTP Phase 2 Effluent Storage and Irrigation System Expansions:** Construct an effluent storage pond with 70-million gallon (MG) capacity. Purchase 195 acres of farmland, install an effluent transmission pipeline to this land, and install an irrigation system at the new site.

E. Potential Projects for Future Industrial Park Developments (ISR Planning)

1. **NWWTP Phase 2 Improvements:** Construct a 0.5-MGD treatment facility with an aerated lagoon system to expand average flow capacity to 1.0 MGD. Construct a second 79-MG effluent storage pond, buy 290 acres of farmland, install an effluent transmission piping to this land, and install an irrigation system at the new site.

2. **NWWTP Phase 3 Improvements:** Construct another 0.5-MGD expansion similar to Phase 2, except that we estimate an additional 345 acres of farmland would be needed.

Table 8-3		
WWTP Improvements – Estimated Timing and Probable Costs (December 2017 Dollars)		
Project Description	Probable Construction Cost	Probable Project Cost
A. Short-Term Projects (within 5 years)		
1. NWWTP Phase 1A Improvements	\$1,766,000	\$2,385,000
<i>Alternate NWWTP Phase 1A Improvements</i>	<i>\$1,002,000</i>	<i>\$1,355,000</i>
2. SWWTP Biosolids Thickening System	\$929,000	\$1,255,000
Subtotal – Short-Term Project ⁽¹⁾	\$1,931,000	\$2,610,000
B. Mid-Term Projects (6 – 10 years)		
1. SWWTP Phase 1 Expansion/Renovation	\$921,000	\$1,245,000
2. SWWTP Irrigation System Expansion	\$382,000	\$515,000
Subtotal – Mid-Term Projects	\$1,303,000	\$1,760,000
C. Mid- to Long-Term Project (11 – 15 years)		
1. NWWTP Phase 1B Improvements ⁽²⁾	\$6,082,000	\$8,210,000
2. NWWTP Land Acquisition & Irrigation System ⁽³⁾	\$1,185,000	\$1,600,000
Subtotal – Mid- to Long-Term Projects	\$7,267,000	\$9,810,000
D. Long-Term Projects (16 – 20 years)		
SWWTP Phase 2 Expansion		
a. Treatment Module w/SBR process	\$7,426,000	\$10,025,000
b. Effluent Storage Pond	\$6,815,000	\$9,200,000
c. Farmland Acquisition & Irrigation System ⁽³⁾	\$2,530,000	\$3,415,000
Subtotal – Long-Term Projects	\$16,771,000	\$22,640,000
E. Potential ISR Projects		
1. NWWTP Phase 2 Expansion		
a. Treatment Module w/aerated lagoons	\$7,915,000	\$10,685,000
b. Effluent Storage Pond	\$9,286,000	\$12,535,000
c. Farmland Acquisition & Irrigation System ⁽³⁾	\$3,478,000	\$4,695,000
2. NWWTP Phase 3 Expansion		
a. Treatment Module w/aerated lagoons	\$9,253,000	\$12,490,000
b. Effluent Storage Pond	\$9,286,000	\$12,535,000
c. Farmland Acquisition & Irrigation System ⁽³⁾	\$3,981,000	\$5,375,000
Subtotal - Potential NWWTP ISR Projects	\$43,199,000	\$58,315,000
Total – Proposed & Potential WWTP Projects	\$70,471,000	\$95,135,000

- (1) Subtotal includes the alternate NWWTP Phase 1A Improvements project as the lower cost option.
- (2) Overall scope and probable project cost of NWWTP Phase 1B project may change if City implements the Alternate NWWTP Phase 1A Improvements project.
- (3) Probable costs include additional farmland and 1.0-mile effluent transmission line to this new land. Further investigation into effluent transmission requirements will be needed once potential properties are identified for purchase.

Potential expansions to the NWWTP would be dependent on future commercial/industrial developments. Project requirements must be evaluated based on actual development proposals. The financial analysis addresses development charges and other funding options for these projects.

The NWWTP projects are also contingent on Federal Aviation Administration (FAA) reviews and acceptance. Additional mitigation measures beyond those incorporated into these probable project costs may be required based on FAA input.

8.3 ADDITIONAL PLANNING CONSIDERATIONS

8.3.1 *Planning Effort for Surface-Water Discharge*

Before the City proceeds with the Phase 1 SWWTP expansion and renovation, it may be worthwhile to further investigate the feasibility of an indirect discharge to Willow Creek. The first step would be to communicate with DEQ regarding the potential for obtaining a discharge permit.

Depending on the response DEQ provides, the City may want to proceed with a study to evaluate the feasibility and potential cost effectiveness of a surface-water discharge. Such a feasibility study is beyond the scope of this WWMP Update. However, Chapter 5 of this WWMP (Section 5.2.2) provides some general background information on the issues affecting a surface-water discharge and outlines some key issues that a Feasibility Study should address.

8.3.2 *Capacity, Management, Operations and Maintenance (CMOM) Program Planning*

Introduction. The City is vested with the responsibilities of providing adequate sewer system capacities and properly managing, operating and maintaining the systems. A formal Capacity, Management, Operations and Maintenance (CMOM) program can support these efforts by providing a framework to help owners adopt and follow accepted wastewater-industry practices.

The general goals of a formal CMOM program are described below.

- Assure that adequate sewer capacity remains available under projected conditions.
- Develop a proactive (predictive) approach to system management, operations and maintenance and reduce reliance on being reactive to unplanned or emergency events.
- Provide quality customer service that helps generate support for dedicating the revenues needed to sustain reliable, sewer service.

The following paragraphs provide an overview of CMOM programs. This information is intended as a starting point for efforts by the City to plan and adopt a formal CMOM program. A detailed analysis of the City's current practices and specific recommendations for improvements to these practices are beyond the scope of this study.

CMOM Program Planning. Since the City has been providing sewer service for over 40 years, it has been following many recognized CMOM practices for an extended period. Some examples of these practices are summarized below.

- The City has adopted a sewer use ordinance, user rates and other charges, and system design and construction standards to establish a legal framework for the systems management.
- The City prepares and submits reports to the DEQ and operates and maintains the WWTPs in compliance with the City's permit requirements.

A key part of CMOM planning is the periodic collection of information on current facilities and CMOM practices. Key information in this WWMP Update that can be particularly useful to a CMOM program review and upgrade are listed below:

- Chapter 2 and the associated appendices, conditions assessments of WW facilities and initial maintenance plan;
- Chapter 4, the collection system analysis; and
- Chapter 9, the financial analysis.

It is important for the City to make use of this WWMP Update as a tool in CMOM program planning and regularly update the WWMP to keep CMOM activities current to actual conditions.

General Requirements of a CMOM Program. It is important that a CMOM program is a formal set of practices that are tailored specifically to the City's circumstances and are understood by the responsible staff members. Staff must also view the program as a dynamic tool that designated personnel should update regularly to keep it current to actual conditions and priorities.

Seven key elements that a CMOM program should incorporate are:

- a clear purpose;
- a well-defined overall goal;
- a written program document;
- implementation by trained, or contracted, personnel;
- clear performance measures to track program effectiveness; and
- plans for periodic reviews and updates.

The CMOM program should be developed and implemented to provide owners and operators with the following capabilities:

- The understanding of how the system components and the overall systems are performing.
- The ability to identify specific performance objectives based on current conditions and needs.
- The organizational structure, staffing, training, and other resources needed to allow personnel to meet performance objectives.
- A recognition of the need to review and adjust activities such as sewer cleaning, inspections, and other maintenance responsibilities to respond to changing conditions.
- An understanding of the need to conduct system capacity evaluations, including flow monitoring when necessary, and implement a capacity assurance program.
- The responsibility to conduct periodic reviews that evaluate CMOM program implementation, identify deficiencies, and take corrective steps.
- The training, equipment, and other preparations needed to respond to emergency events.
- An ability to communicate what efforts are required for CMOM program performance.

CMOM Program Review. The City can improve current practices by conducting a thorough review of their CMOM practices, developing a written CMOM program, and establishing specific objectives for CMOM performance.

The United States Environmental Protection Agency (USEPA) has published a “*Guide for Evaluating CMOM Programs at Sanitary Sewer Collection Systems*” (2005) to support such efforts. This document describes the main elements of CMOM, the procedures for conducting reviews, and the process of implementing a successful program. The City should find this guidance helpful in program planning.

The USEPA guidance focuses on collection system CMOM as part of a regulatory effort to prevent sanitary sewer overflows. However, many CMOM practices in Madras can be effectively applied to all wastewater services the City provides.

We recommend the City conduct an assessment of current CMOM practices, either internally or with the help of a consultant, to identify areas of improvement. Appendix O includes a CMOM Program Self-Assessment Checklist published by the USEPA that provides guidance in performing this program review.

Each CMOM program is specific to the utility that provides the sewer service. Therefore, only applicable aspects of the checklist should be used in the program review and other considerations not reflected in the checklist may need to be addressed.

Performance Objectives for CMOM Program. USEPA guidance emphasizes the importance of measurable performance objectives as part of a successful CMOM program. The City should establish performance objectives in coordination with current priorities for addressing areas of concern.

The following is a brief list of potential areas the City may want to emphasize in defining CMOM performance objectives.

- Maintenance Plan – formally establish and implement a plan that prioritizes maintenance needs, supports budgeting, and tracks progress.
- Sewer Cleaning Program – develop a plan that prioritizes system needs and adopts a schedule.
- Recordkeeping – establish goals for updating and maintaining records such as safety procedures, emergency response plans, O&M manuals, and/or record drawings.
- Training Program – develop goals for periodic safety and/or emergency response training.

The City may prefer to emphasize other aspects of system management, operations and maintenance in the performance objectives. Regardless, the key is to clearly define the objectives, develop a process for tracking progress, and determine whether staff are can meet these objectives.

8.3.3 *Solar Power Array Funding and Partnering Opportunities*

Background. Solar photovoltaic (PV) panels have become increasingly cost competitive as a way to generate electricity onsite and reduce the amount of electricity purchased from a utility. Additionally, the State of Oregon has established policies and implemented incentives programs to encourage the development of renewable energy sources (renewables) such as solar power. The State offers these incentives through the Oregon Energy Trust and the Oregon Department of Energy. Currently these State programs are augmented by Federal tax incentives offered in the form of credits and accelerated depreciation for renewables investments.

Due to the availability of State and Federal incentives, it may be cost effective for the City to install an array of PV panels at either or both WWTP sites as a means for lowering power costs. The following paragraphs describe key issues affecting the installation of solar arrays at each site.

Type of Solar Array Installations. Solar arrays can potentially be installed on ground-based supports, on roof-mounted supports, or on anchored floats in a pond. Given the power consumption and building sizes at the City's WWTPs, roof-mounted arrays would not provide enough PV capacity to reduce electricity costs significantly. Therefore, we recommend the City focus any investigation into solar arrays on ground-based and floating solar arrays.

An array mounted on the ground would generally be preferred due to the cost and access issues associated with a floating array. However, installing a floating array on an effluent storage pond at the NWWTP could potentially provide the benefit of mitigating wildlife attraction by covering part or most of the pond surface. A floating array would also leave surrounding land available for industrial development, farming or other uses. In addition, the pond water can help to cool PV panels and improve power-generating efficiency.

Solar Power Purchase Agreements. Federal tax credits and depreciation incentives would not be directly available to a government entity. Therefore, to take advantage of these benefits, a City could enter into a power purchase agreement (PPA) with a private, solar service provider.

Under a PPA, the service provider would furnish, install, own and maintain the power-generating system. The provider would also be responsible for project financing and would receive the incentives and tax benefits for which the project qualifies. In exchange, the City would agree to purchase the electricity produced by the system for the term of the PPA, usually 20 years in Oregon.

The rate established by the PPA for the solar-generated power would typically be less than the current electricity rate to give the City an incentive to enter the PPA. At the end of the PPA term, the City would have the option to renew the PPA, purchase the system, or remove the system.

Appendix P contains a fact sheet on PPAs published by the National Renewable Energy Laboratory. The main advantages to a PPA for the City include:

- low initial cost for a solar array;
- the ability to benefit from Federal tax benefits;
- a predictable cost of electricity for the term of the PPA;
- no need to deal with the solar system permitting, design, and construction; and
- no operations and maintenance responsibilities for the array.

State Incentives. The following is a brief summary of the incentives available through the State.

- *Energy Trust Incentives.* Pacific Power customers can receive cash incentives from the Energy Trust that lower capital costs based on the power-generating capacity. This incentive program is primarily funded by the utility in response to State legislation requiring utilities to meet goals for renewables development.

Current Energy Trust incentives for Pacific Power customers start at \$0.80/watt for a generating capacity of 10 kW or less and decrease to \$0.45/watt for a capacity of 200 kW or more.

The Energy Trust does not have an agreement with the Central Electric Cooperative for cash incentives. Therefore, this incentive is only available at the SWWTP site.

- *Renewable Energy Development (RED) Grants.* The Oregon Department of Energy (ODOE) currently offers RED grants annually through a competitive application process. ODOE has been advertising the availability of RED grants near the end of a calendar and then accepting applications through the early part of the following year. In the past, grants have been limited to 35% of project cost and \$250,000. The State authorizes and funds the RED grant program through legislation and the resulting statute.

Federal Tax Incentives. The following is a brief summary of the incentives for renewables investment currently offered by the Federal government.

- *Tax Credit.* The Business Energy Investment Tax Credit can currently cover 30% of the capital cost of a solar system. However, this percentage is scheduled to begin dropping in 2020.
- *Accelerated Depreciation.* Federal tax law allows businesses to depreciate the cost of a solar system following the Modified Accelerated Cost Recovery System. This approach can provide a tax deduction that offsets up to 25% of the project cost.

Solar Arrays at Airports. The FAA recognizes the benefits of installing solar power projects at airports and generally supports their implementation. To promote aviation safety, the FAA has studied the visual impacts of solar panels and ways to prevent hazards from panel installations.

These studies have resulted in FAA guidance that addresses technical considerations for solar projects (*Technical Guidance for Evaluating Selected Solar Technologies on Airports*, 2010). The FAA also published an interim policy in the Federal Register (Volume 78 – No. 205) that addresses standards for glint, a brief flash of light, and glare, a sustained source of brightness, from solar panels (*FAA Review of Solar Energy System Projects on Federally Obligated Airports*, 2013).

Because the NWWTP is located adjacent to the airport, a solar array installed at the plant site would need to be designed according to FAA standards and technical guidance. Therefore, an evaluation of solar arrays at the NWWTP site should consider the impacts of FAA policy and guidance.

8.3.4 Master Plan Updates

We recommend the City update this WWMP and the CIP on a regular basis, particularly when changes occur that are likely to have significant impacts on previous recommendations. In general, the City should plan to have a WWMP update prepared every 5 years. The City should tailor the scope of each update to address any changes in the basis of planning (population growth and wastewater production), conditions of existing components, regulatory requirements, effluent handling practices, biosolids management, and other issues described in this report.

CHAPTER 9

WASTEWATER SYSTEM DEVELOPMENT CHARGE UPDATE

9.1 INTRODUCTION

As part of this master planning effort, FCS GROUP performed an analysis to update the City's system development charges (SDCs) for wastewater services. This chapter includes a summary of the SDC report FCS GROUP prepared to describe their analysis and present their conclusions. The complete report is contained in Appendix Q. The City plans to have a full financial analysis of the wastewater facilities and a sewer rate study completed separately at a later date.

9.2 GENERAL DESCRIPTION OF SYSTEM DEVELOPMENT CHARGES

The City levies SDCs as one-time fees on new developments to recover a fair share of the cost of facilities that provide capacity to serve future growth. The authority to levy local SDCs was established by the Oregon Legislature and incorporated into the Oregon Revised Statutes. The law allows SDCs only for the recovery of eligible capital costs and not the recovery of costs to operate and maintain facilities.

The City's SDCs consist of the following three main components:

1. A **reimbursement fee component** is included to recover a share of the costs associated with existing wastewater facilities or capital projects under construction.
2. An **improvement fee component** is included to recover the estimated costs of planned capital improvements that will expand capacity for future users.
3. An **administrative cost component** is included to recover the portion of SDCs spent on compliance with applicable rules, including costs to determine SDCs and costs to account for SDC expenditures.

The SDC calculation requires a determination of the eligible costs for capital improvements and an estimate of the growth in customer base that the improvements can accommodate. In general terms, the SDC is established by dividing the eligible capital costs by the estimated number of new customers that can be served.

9.3 CUSTOMER BASE

Since there are different types of customers with differing service needs, the existing and expanded customer base are each estimated on an equivalent basis using water meter size. A standard residential meter size of 5/8-inch has a meter capacity equivalent (MCE) of 1.0 and larger meters have higher MCEs based on their relative capacity. Using the current breakdown of water meter sizes for customers in Madras, FCS GROUP identified an existing MCE of 3,074 for the City's wastewater facilities.

FCS GROUP used the following flow data to establish a future customer base:

- the planned total future capacity (1.65 MGD total – 1.04 MGD existing and 0.61 MGD expansion);
- the existing average wastewater flows (0.493 MGD);
- capacity reserved for the Deer Ridge facility (0.230 MGD); and
- net available capacity (0.927 MGD).

The existing MCE of 3,074 and existing average flow of 0.493 MGD produce a unit wastewater flow of 160 gallons per MCE. Based on this unit flow, the net available capacity of 0.927 MGD translates to 5,779 MCEs in future growth that could be served. FCS GROUP used this available growth in customer base as the denominator for calculating the reimbursement and improvement components of the SDCs.

9.4 REIMBURSEMENT FEE

The reimbursement fee is based on the City's eligible costs for constructing the unused part of the existing facilities capacity. Existing average flows and the dedicated capacity for the Deer Ridge facility constitute 70% of the existing facilities capacity. Therefore, 30% of the existing capacity is unused and available for future growth.

FCS GROUP compiled and analyzed data from the City on previous capital expenditures for wastewater facilities to estimate the total original cost of providing the existing capacity. A 30% share of this total amount was used to identify the eligible cost basis. Further deductions were then made to account for the current balance in the reimbursement SDC fund and to avoid duplicate charges in the reimbursement fee.

The analysis determined that the City's net eligible cost for unused existing capacity is approximately \$3.4 million. Dividing this cost by the future MCE of 5,779 results in an updated reimbursement fee of \$586.

9.5 IMPROVEMENT FEE

The improvement fee cost basis is the portion of the Capital Improvements Plan presented in Chapter 8 that constitutes net eligible costs. Capital costs that are not eligible for SDCs include the following planned expenditures:

- costs for projects not expected to be funded by the City; and/or
- costs for projects, or portions of projects, that do not expand capacity.

Once these ineligible costs were deducted, FCS GROUP incorporated approximately \$25 million into the improvement fee calculation. After deducting the current balance in the City's Improvement SDC Fund, the net eligible future costs were divided by the future MCE of 5,779. The result is an updated improvement fee of \$4,281.

9.6 ADMINISTRATIVE COST ADJUSTMENT FOR COMPLIANCE

The cost adjustment for compliance with existing statutes includes an estimate of annual administrative costs related to the SDCs and the amortized costs for the SDC study prepared by FCS GROUP (Appendix Q). The combined total for these annual compliance costs is \$279 per new connection.

9.7 SUMMARY OF RECOMMENDED SYSTEM DEVELOPMENT CHARGES

Table 9-1 on the following page summarizes the recommended wastewater SDC components and fee schedule. FCS GROUP recommends the City periodically adjust these proposed SDCs to account for inflation by using the Engineering News Record 20-City Average Construction Cost Index.

Table 9-1 Summary of Recommended Wastewater System Development Charges		
A. Updated Wastewater SDC per MCE		
Reimbursement Fee		\$586
Improvement Fee		\$4,281
Administrative Cost Recovery		\$279
Total Updated SDC		\$5,146
Net Decrease from Existing SDC		(\$143)
B. Wastewater SDC Fee Schedule Based on Meter Size		
<u>Meter Size</u>	<u>MCEs</u>	<u>Updated SDC</u>
5/8 inch	1.0	\$5,146
1 inch	2.5	\$12,864
1.5 inches	5.0	\$25,729
2 inches	8.0	\$41,166
3 inches	17.5	\$90,051
4 inches	30.0	\$154,373
5 inches	62.5	\$321,611
6 inches	90.0	\$463,120
C. Wastewater SDC Fee Schedule – Multi-Unit Developments		
<u>Customer Type</u>	<u>Assumed MCE/Unit</u>	<u>Updated SDC/Unit</u>
Multi-Unit Residential	0.71	\$3,654
Lodging Facility	0.74	\$3,808

