Wastewater Treatment Plant Facility Plan

City of Ridgecrest, CA

October 2015

REVIEW DRAFT

Prepared for:

City of Ridgecrest

Prepared by:

Provost & Pritchard Consulting Group
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APPENDICES

Appendix A: Existing Wastewater Treatment Plant Waste Discharge Requirements

Appendix B: Water Balances

ABBREVIATIONS and ACRONYMS

AAD	annual average daily flow
ADMM	average day maximum month flow
AF	acre-feet
BOD_5	5-day biochemical oxygen demand
CEQA	California Environmental Quality Act
CMAS	complete mix activated sludge
EC	electrical conductivity
EPA	Environmental Protection Agency
ET	evapo-transpiration
ExAAS	extended aeration activated sludge
FDB	Food and Drug Branch, Division of Drinking Water, SWRCB
gpcd	gallons per capita per day
gpd	gallons per day
gpm	gallons per minute
HRT	hydraulic retention time
I/I	infiltration and inflow
IWVWD	Indian Wells Valley Water District
MBR	membrane bioreactor
MCL	maximum contaminant level
MG	million gallons
mgd	million gallons per day
mg/L	milligrams per liter
MLQ	mixed liquor
MPN	most probable number
NAWS	China Lake Naval Air Weapons Station
NOI	notice of intent
PHF	peak hour flow
ppd	pounds per day

RAS .... return activated sludge
RWQCB .... Regional Water Quality Control Board
SBR .... sequencing batch reactor
SF... square feet
SRT .... solids retention time
SWRCB .... State Water Resources Control Board
TSS .... total suspended solids
μg/L ....micrograms per liter
VOC .... volatile organic compounds
WAS .... waste activated sludge
WDR .... waste discharge requirements
WWTF………………………………………………………...wastewater treatment facility
WWTP .... wastewater treatment plant

..

EXECUTIVE SUMMARY

To be completed when review comments are received from the City.

1 INTRODUCTION

1.1 Background

The City of Ridgecrest (City) operates wastewater collection, treatment, and disposal facilities serving residential and commercial development within City limits and the China Lake Naval Air Weapons Station (NAWS). The City is located approximately 110 miles east of the City of Bakersfield, in the Indian Wells Valley situated in the High Mojave Desert of California. The City is located in the northeastern portion of Kern County, with portions of City situated adjacent to the San Bernardino County line. A vicinity map is included as **Figure 1-1.**

The City authorized Provost and Pritchard Consulting Group (P&P) in June 2011 to prepare a Draft Facility Plan report for the wastewater treatment plant project. That report was a follow-up report to one originally prepared by Carollo Engineers in September 2008. The City has requested that P&P update the draft facility plan prepared in 2011 to include two wastewater treatment plant site alternatives. This facility plan update considers construction of new wastewater treatment facilities (WWTF) on City owned land within the City limits and adjacent to the existing WWTF at the China Lake NAWS site. This report will utilize background information from the previous reports, with updated analyses of population and flows, treatment and disposal alternatives, and site location alternatives. Both site alternatives will be considered and evaluated in the environmental review process through the California Environmental Quality Act (CEQA).

According to the *City of Ridgecrest Wastewater Treatment Plant Final Project Report* prepared by Carollo Engineers in September 2008, Ridgecrest Sanitation District (RSD) was established in the mid 1950s to serve the small civilian service community that had developed outside of the NAWS base. At that time, wastewater from the NAWS was treated at a facility located on the base, and RSD operated a separate smaller plant within the City. In the mid-1970s the population began to shift from NAWS to the City, creating capacity problems at the RSD treatment plant. The Environmental Protection Agency (EPA) mandated that the City and NAWS consolidate their wastewater treatment facilities and treat the combined flows at a common plant. The existing plant located in the City was therefore abandoned, and the City has been operating a treatment plant at the NAWS site since 1974. That facility, including an expansion completed in 1976, provides capacity to treat both the City and NAWS current flows (Carollo, September 2008). The site of the current facility will be referred to in this report as the “NAWS site”. The NAWS site is shown in **Figure 1-2**. The City owned land in the area of the former WWTP that provided treatment prior to 1974 will be referred to herein as the “City site”. The City owned land is shown in **Figure 1-3.**

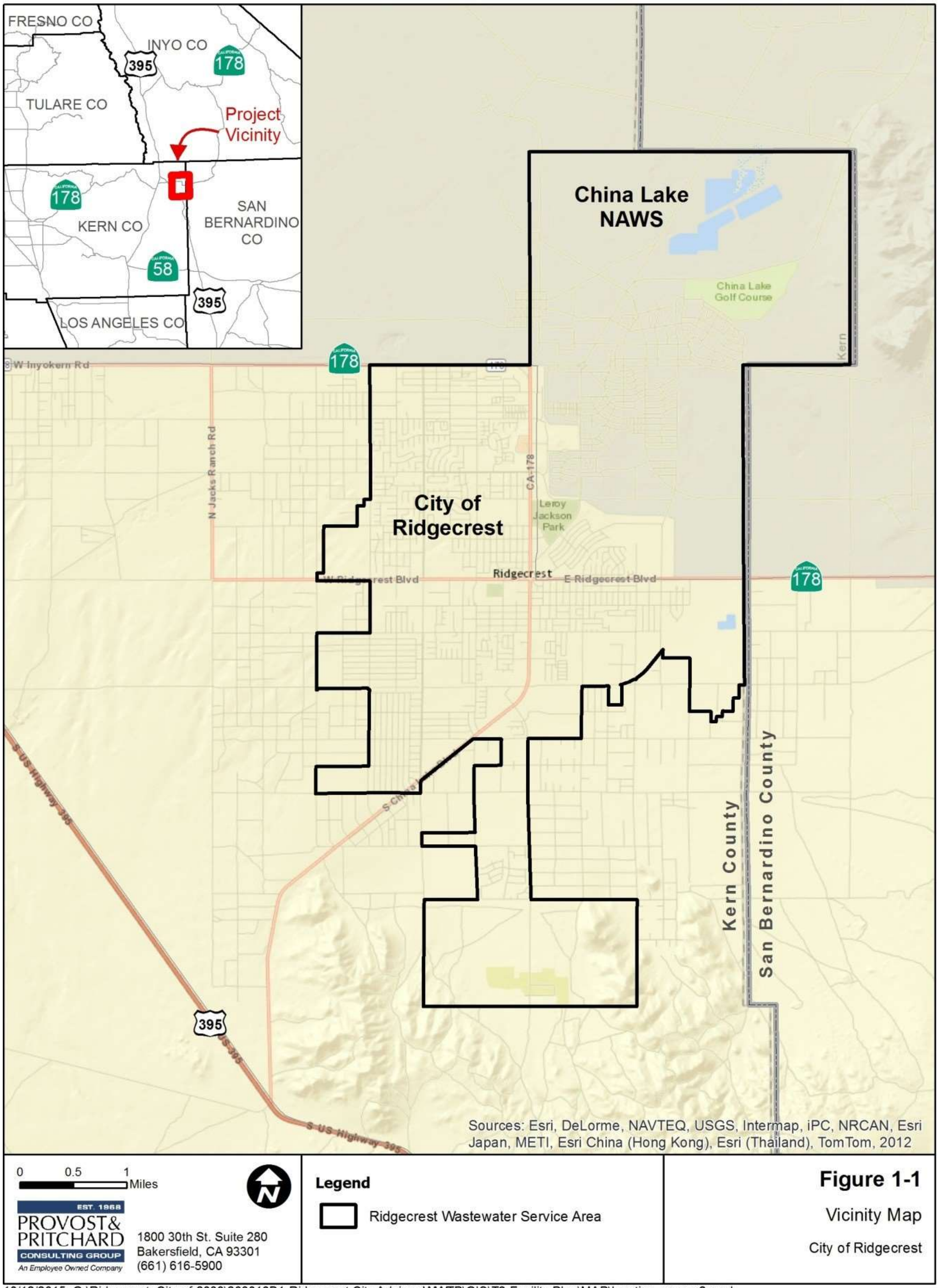

Figure 1-1
Vicinity Map
City of Ridgecrest

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Figure 1-1. Vicinity Map

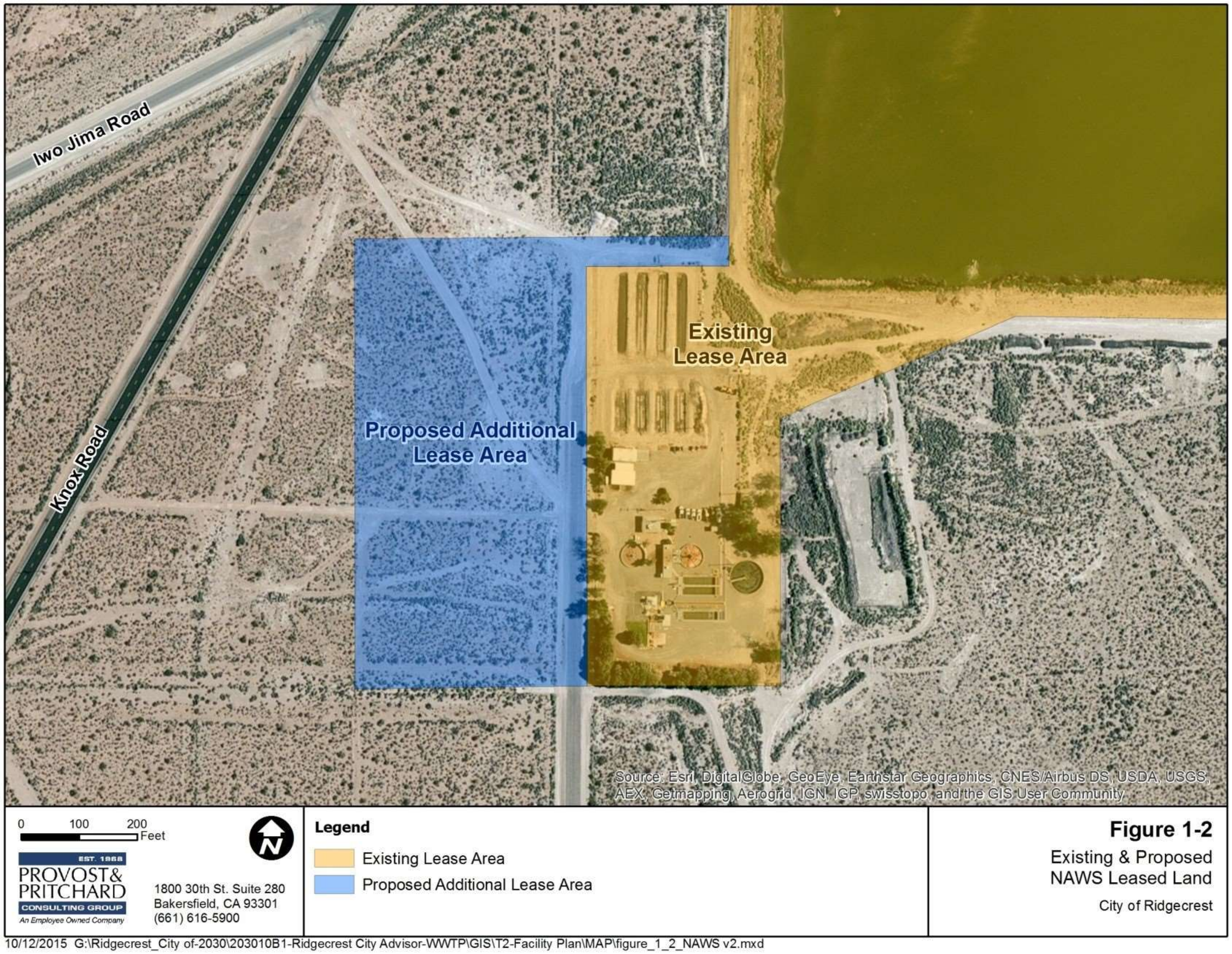


Figure 1-2. Existing & Proposed NAWS Leased Land

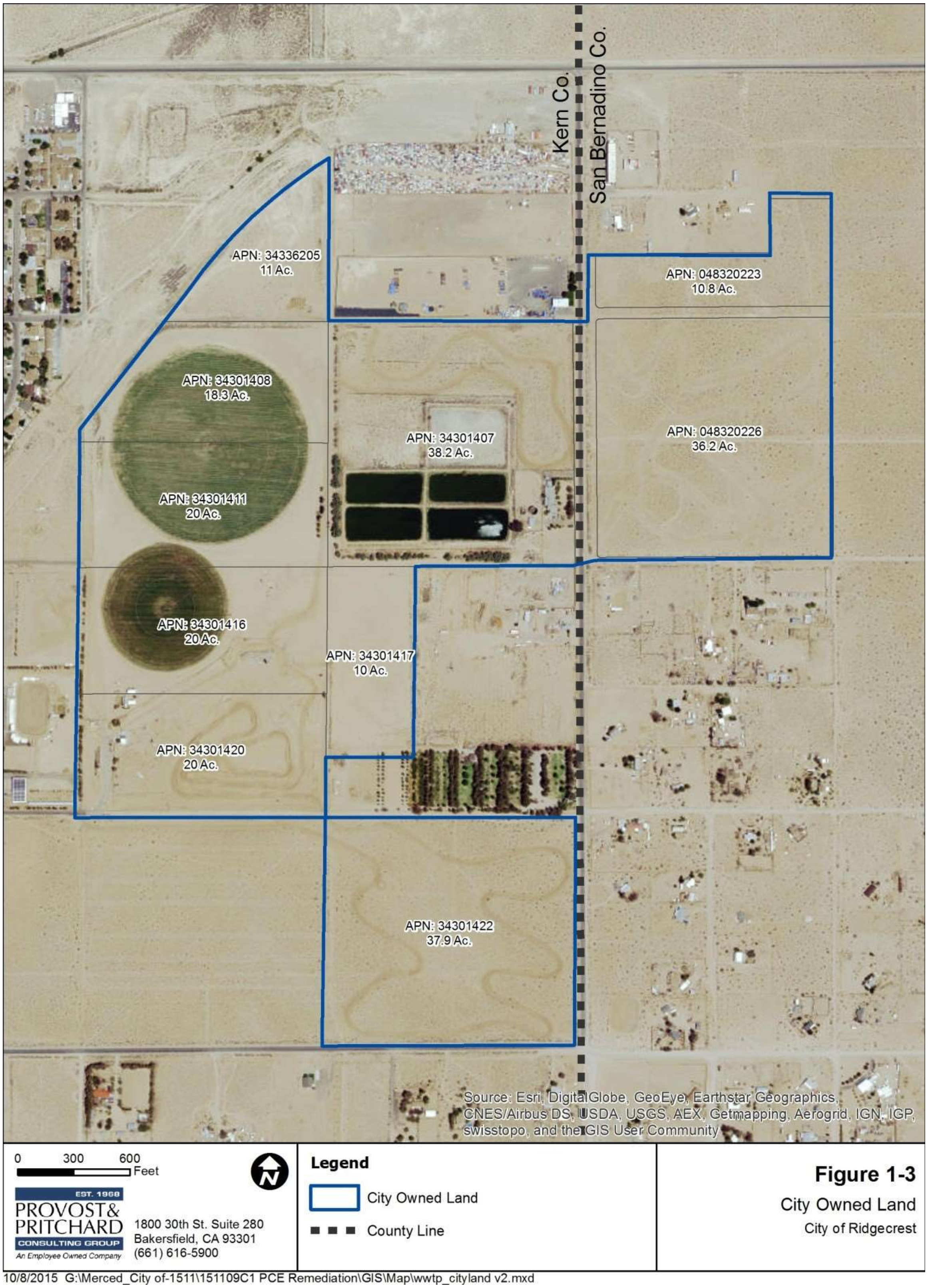


Figure 1-3. City Owned Land

1.2 Need for the Project

The existing WWTP located on the NAWS site has been in service for many years and is in need of significant repair to maintain its functionality. The latest WWTF upgrade was the reconstruction of the headworks screening and grit removal processes in the mid 2000's. Most other facilities including the primary sedimentation basins, anaerobic digesters, and wastewater disposal ponds were originally constructed in the 60's and 70's and have not been upgraded. Fundamental portions of the plant process components are deteriorated, and certain conditions may be considered hazardous. The electrical system is very outdated and does not meet current code requirements. Concrete and steel show visible signs of corrosion and deterioration. Considering the condition of the existing WWTP, it is clear that the existing facility will not continue to adequately serve the City without substantial improvements, or replacement.

The existing WWTF is a primary plant followed by facultative ponds for supplemental oxidation with disposal by evaporation and percolation. The facility provides for removal of settleable solids and, biochemical oxygen demand (BOD) but does not provide nutrient removal. Because of the age of the plant and the relatively low level of treatment provided, it is not well suited for wastewater effluent recycling. A new WWTF will provide the necessary plant reliability, alarms and operating features as required by California Recycled Water Regulations (Title 22) to allow for production of a higher quality recycled water, including unrestricted reuse for park and landscape irrigation.

Additionally, the existing WWTP has approached its permitted capacity of 3.6 million gallons per day (mgd). The existing waste discharge permit only allows a daily maximum discharge of 3.6 mgd. Because daily flows vary, the effective average daily flow capacity is limited to approximately 3.3 mgd. In 1993, flows reached 3.3 mgd, before population began to decline in the mid 1990s. Flows through the WWTP over the past five years averaged approximately 2.6 mgd. Recent drought conditions and water conservation has further reduced flows to a current level of 2.1 mgd. The current WWTF provides limited reserve capacity for growth.

1.3 Purpose and Scope of Study

This report considers the wastewater treatment and disposal facilities needed to accommodate population and commercial growth for the City of Ridgecrest and China Lake NAWS, through approximately year 2050. This will require substantial upgrades and improvements of the existing WWTP or construction of a new wastewater treatment plant.

The existing WWTP has a permitted maximum capacity of 3.6 mgd, however it is near the end of its useful life and is not expected to be capable of effectively treating wastewater at its permitted flow. Although some useable capacity may remain at the existing WWTP, the age and condition of the existing WWTP dictate that the best alternative will be to construct a new facility at either the NAWS site or the City site, allowing the existing WWTP to be retired. This also allows the existing WWTP to remain in operation while a new WWTP is constructed and commissioned, reducing costs associated with maintaining wastewater treatment capability during construction.

This report evaluates wastewater treatment, effluent disposal and biosolids disposal alternatives, and potential sites for the new wastewater treatment plant. Recommended projects, in compliance with current and anticipated future waste discharge requirements as promulgated by the RWQCB – Lahontan Region are proposed. As stated previously, the City has asked P&P to include options of upgrading and expanding both the NAWS WWTP site and the existing City WWTP site to provide wastewater treatment alternatives for the City.

Funding mechanisms for the Project are not yet fully defined. It is probable that the City will pursue funding from the Clean Water State Revolving Fund Program (CWSRF). Other funding sources such as City issued bond financing may be considered. To the extent possible, this study is directed at fulfilling and satisfying the requirements of these funding and procurement programs.

Previous Studies and Reports

The following studies and reports previously prepared have been utilized in the preparation of this report, including:

- Master Sewer Plan for the City of Ridgecrest, 1981
- Revised Master Sewer Plan for the City of Ridgecrest, 1987
- Master Plan for Wastewater Treatment Facilities, 1994
- City of Ridgecrest Wastewater Treatment Plant Expansion, 1998
- City of Ridgecrest – Sewer Study Letter, 2002
- Groundwater Management in the Indian Wells Valley Basin – Ridgecrest California, 2003
- Indian Wells Valley Water District – 2005 Urban Water Management Plan, 2005
- City of Ridgecrest Wastewater Treatment Plant Final Project Report, 2008 (“Carollo Report”)
- Winter 2010 Groundwater Monitoring Report, City of Ridgecrest Wastewater Treatment Facility, Ridgecrest, California, Tetra Tech, 2010.
- Winter 2014 Groundwater Monitoring Report, City of Ridgecrest Wastewater Treatment Facility, Ridgecrest, California, BSK Associates, February 6, 2014
- Indian Wells Valley Water District 2013 Consumer Confidence Report

1.4 General Information

1.4.1 Climate

The City of Ridgecrest is located within the Indian Wells Valley, approximately 110 miles east-northeast of the City of Bakersfield, on Business Route 395 in Kern County. The climate is typical of a California high desert, characterized by hot, dry summers and mild winters. The summer temperatures range from daytime highs averaging over 100 degrees Fahrenheit (°F) to nighttime lows typically in the upper 60's to low 70's °F range. Winter temperatures are mild during the day, but often drop below 32 °F at night.

Precipitation in the Indian Wells Valley is scarce. Precipitation averages less than 5 inches per year, with most of the rain falling between November and April. Summer thunderstorms are common. Some winter precipitation can fall as snow. Evaporation rates are high due to clear skies, low humidity, wind and high temperatures. Crop evapo-transpiration rates are among the highest in California.

1.4.2 Topography

The average ground elevations in the City of Ridgecrest are approximately 2,300 feet above sea level, with mild slopes of about 0.5 percent, falling generally to the northnortheast. The City generally drains to dry lake beds located on the NAWS site. The NAWS site has elevation of about 2215 feet and the City site has an elevation of about 2280 feet.

1.4.3 Geology

The NAWS WWTP site overlies layers of recent alluvium consisting of silt, sand, and freshwater marl cemented with calcareous tufa. The alluvium comprises the upper portion of the valley fill, which extends to a depth of at least 1,350 feet (RWQCB, WDR 6-00-56).

The City WWTP site overlies layers of silty sand, sand, clayey sand, and sandy clay, as well as some areas of caliche (BSK 2014).

1.4.4 Water Supply

All residential and commercial water service for the City of Ridgecrest is provided by Indian Wells Valley Water District (IWVWD). The IWVWD is a public water district which provides potable water from groundwater wells. Groundwater in the basin is not adjudicated. Capital planning and source adequacy of water supplies for the area surrounding Ridgecrest is performed exclusively by IWVWD, and are not specifically addressed herein

According to the Carollo Report, the normal depth to water in 2003 was approximately 160 feet. According to a IWVWD Consumer Confidence Report from 2013, source water from their wells is of good quality, with average levels of EC of 503 µmhos/cm, sodium of

92 mg/L, chlorides of 63 mg/L, total hardness of 64 mg/L, and nitrate concentration of 5.9 mg/L.

China Lake NAWS owns and operates its own potable water system, consisting of groundwater wells and storage tanks. Capital planning and source adequacy of that system are the responsibility of the US Navy and are not specifically addressed herein.

The City owns and operates several groundwater wells that are used to irrigate park sites and landscaped areas at the Ridgecrest City Hall.

Groundwater in the Indian Wells Valley has been extensively studied and monitored. The availability of good quality groundwater is limited. IWVWD is currently utilizing water treatment plants to remove arsenic. IWVWD has also planned for future brackish water treatment.

1.4.5 Groundwater

As mentioned in the Water Supply section above, the deep groundwater aquifer below the City, which is the potable water supply, is of good quality. There is a shallow perched aquifer below the City site that is of poor quality. This shallow groundwater contains high chloride, total dissolved solids (TDS), and arsenic concentrations.

According to the Winter *2014 Semiannual Groundwater Monitoring Report*, prepared by BSK Associates Engineers & Laboratories, there are four groundwater monitor wells at the NAWS WWTP site and three monitor wells at the City site (**Table 1-1**). Depth to water in the NAWS WWTP site monitor wells ranges from approximately 7 feet to 20 feet below ground surface (bgs). Depth to water in the City site monitor wells ranges from approximately 107 feet to 171 feet bgs (BSK 2014).

Concentrations of total dissolved solids (TDS) in the monitor wells at the NAWS WWTP site ranged from 1,100 to 1,500 mg/L, and chlorides ranged from 170 to 390 mg/L for the winter 2014 semiannual groundwater monitoring event. Arsenic was not analyzed during that event, however during the summer 2013 semiannual groundwater monitoring event, arsenic concentrations ranged from 260 to 550 μg/L.

Concentrations of TDS in the monitoring wells at the City site ranged from 740 to 780 mg/L, and chlorides ranged from 210 to 320 mg/L for the winter 2014 semiannual groundwater monitoring event. Arsenic was not measured during the winter 2014 event, however during the summer 2013 semiannual groundwater monitoring event, arsenic concentrations ranged from 9.9 to 43 μg/L.

Table 1-1. Monitor Well Groundwater Quality Summary

	NAWS WWTP Wells				City Site Wells		
	26S40E14B01	26S40E13D03	26S40E13C02	26S40E13M02	CR-MW01	CR-MW02	CR-MW03
Groundwater Elevation	2179.57	2178.71	2182.59	2180.55	2159.05	2161.09	2159.36
Ammonia (NH4) mg/L	0.35	0.13	0.15	0.18	0.16	0.17	0.13
Chloride mg/L	290	390	270	170	320	220	210
Nitrate as N mg/L	<1.1	<2.2	<1.1	<0.6	2.5	1.2	1.5
TKN mg/L	1.4	<1.0	<1.0	<1.0	<1.0	<1.0	<1.0
TDS mg/L	1400	1500	1200	1100	740	760	780
Surfactants (MBAS) mg/L	0.21	0.24	0.096	0.09	<0.05	<0.05	<0.05

2 EXISTING AND PROJECTED SERVICE AREA

2.1 Sewer Service Areas

The existing City of Ridgecrest WWTP is located approximately 3.5 miles northeast of the center of the City, within the boundary of China Lake NAWS.

The local topography divides the City into two distinct sewer service areas (watersheds); each watershed area is able to collect flows from residents by gravity without the need for pumping facilities. The natural outlet of the northern service area lies at the site of the NAWS treatment plant, while the natural outlet of the southern service area lies at the City site. Existing topography would support locating a new treatment facility at either location. Both watershed areas are shown in **Figure 2-1.** Sewers currently in place carry all flow from the southern watershed area to NAWS without pumping. Topography will not allow flows from the northern watershed area to flow by gravity to the City site. A discussion of flow generated in each of the watershed areas is provided in Section 3.4.

The City has requested that the US Navy grant an easement for a 7.39 acre site adjacent to the existing WWTF for the construction of a new WWTF (Figure 1-2). The US Navy is currently processing this request but an easement has not yet been granted. The City owns the City site, which could be utilized for wastewater treatment and disposal facilities.

2.2 Historic and Projected Populations

The Ridgecrest population is closely tied to NAWS operations and growth. As a consequence, City population growth has been somewhat erratic, rising and falling with NAWS expansions or cut backs, rather than a steadier growth pattern experienced in other, similar sized communities. The timing and magnitude of population growth in Ridgecrest is difficult to predict; however, long term growth projections can still be made.

The 2010 population within the City of Ridgecrest was 27,616 according to the U.S. Census Bureau, 2010 Census. This population includes only that population residing within the corporate limits of the City. The census tract containing China Lake NAWS had a 2010 population of 2,440, yielding a total population of 30,056 for the entire service area. The Kern Council of Governments has estimated a growth rate for the Ridgecrest area of 1.8 percent annually (Kern Council of Governments, *Final Regional Growth Forecast Report*, October 2009). Based on this population projection, it is estimated that the population of the City of Ridgecrest, including NAWS, will be approximately 39,300 in 2025 and 61,400 in 2050, as shown in Table 2-1. Historic populations referenced prior to 2010 include only the population within the corporate City limits, and do not include population within NAWS. The 2010 population includes the City and NAWS populations, and this total population is used for future population projections. Historic and projected populations for the City of Ridgecrest are also illustrated in Figure 2-2. Wastewater flow projections will be based on these population estimates.

The Carollo Report projected a 2010 population of 34,166. Population growth in Ridgecrest obviously did not occur as projected, thus reducing the urgency for a WWTP expansion. The age and condition of the WWTP remains a larger concern.

Table 2-1. Historic and Projected Population

Year	Population	Annual Percent Growth
1985	21,700[1]	
1990	27,600[1]	
1995	27,900[1]	
2000	24,927[2]	
2001	25,219[2]	1.2
2002	25,533[2]	1.2
2003	25,587[2]	0.2
2004	25,842[2]	1.0
2005	26,666[2]	3.2
2006	26,515[2]	-0.6
2007	27,944[2]	5.4
2008	28,631	2.5[4]
2009	29,335	2.5[4]
2010	30,056[3]	2.5[4]
2014	32,279	1.8
2015	32,860	1.8
2020	35,926	1.8
2025	39,278	1.8
2030	42,942	1.8
2035	46,949	1.8
2040	51,329	1.8
2045	56,118	1.8
2050	61,354	1.8

1. California Department of Finance population data, as referenced in 2008 Carollo Report.
2. California Department of Finance, Demographic Research Unit - City/County Population and Housing Estimates, 2000-2007 (City of Ridgecrest only; does not include NAWS population).
3. U.S. Census Bureau, 2010 Census for City of Ridgecrest and NAWS.
4. Annual percent growth calculated based on perceived growth between 2007 and 2010, however 2007 population does not include NAWS population and 2010 population value does.

The U.S. Census Bureau, 2010 Census for the City of Ridgecrest states there are 27,616 people and 10,781 occupied housing units within the City. The average number of persons per housing unit is thus 2.56. This is somewhat lower than the average for Kern County, which is approximately 3.03 persons per housing unit. The 2010 Census for China Lake NAWS indicates that there are 2,440 persons in 603 occupied housing units. The average number of persons per housing unit for NAWS is therefore 4.05, which is significantly higher than the average for the City of Ridgecrest or Kern County.

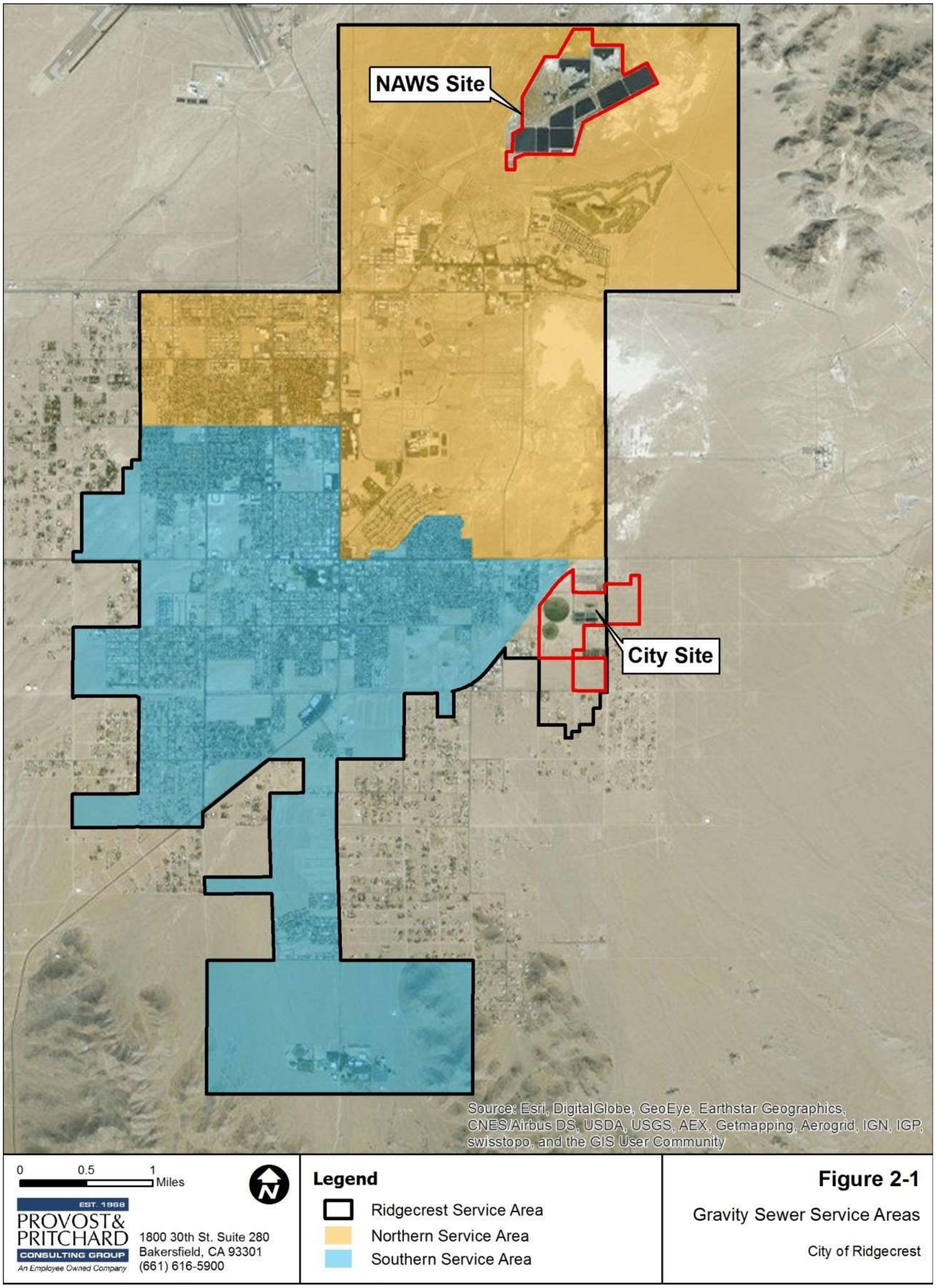
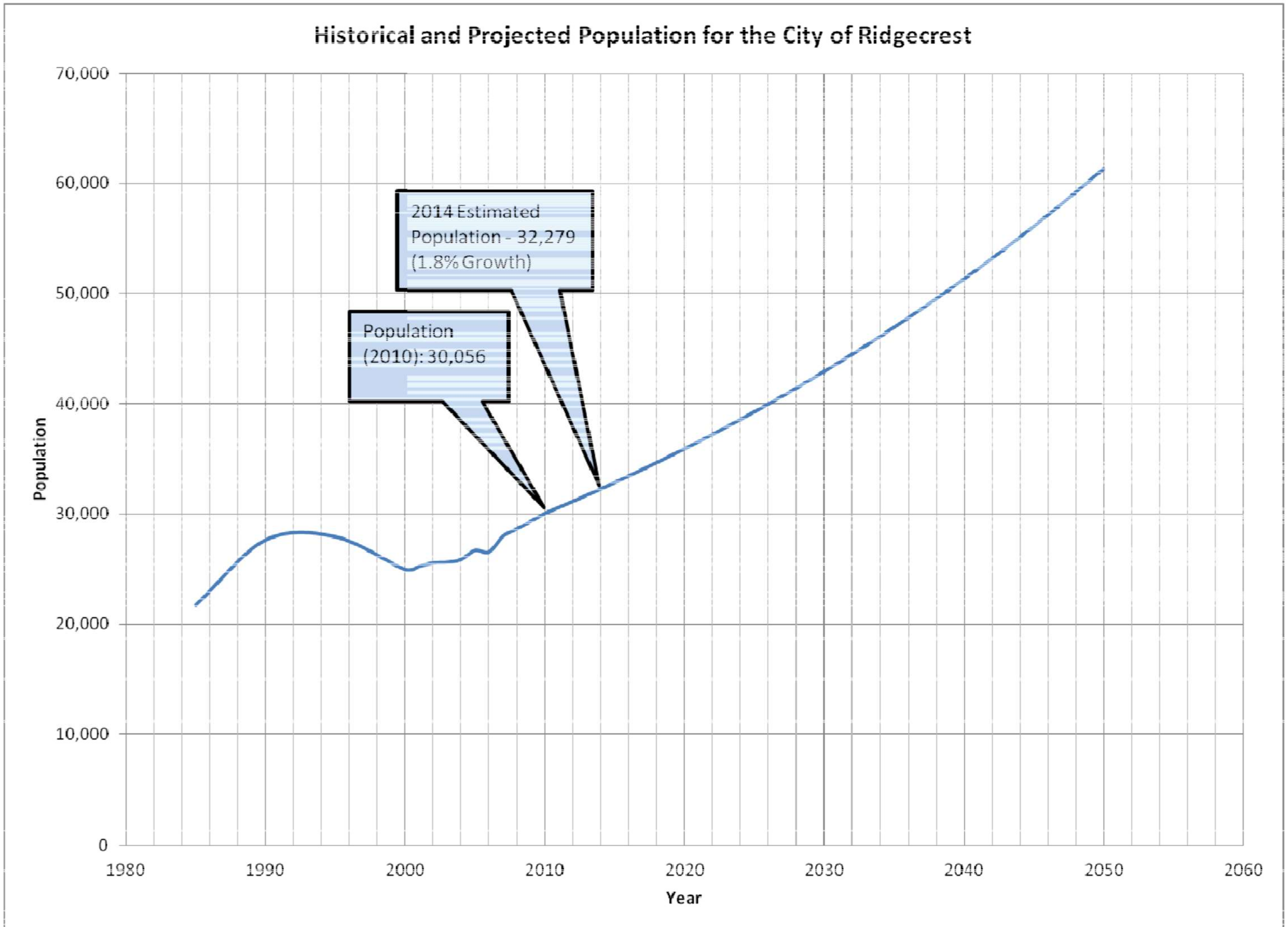


Figure 2-1. Gravity Sewer Service Areas

Figure 2-2. Historic and Projected Population



Note: Historic populations prior to 2010 include only the population within the corporate City limits. Population projections beginning in 2010 include the total population within the City.

3 HISTORIC AND PROJECTED FLOWS AND LOADINGS

3.1 Historic Influent Flows

3.1.1 Annual Average Daily Flow

Population estimates are critical in determining existing per capita wastewater flows and future wastewater flow projections. The existing WWTP has a permitted capacity of 3.6 mgd, and currently treats an average annual flow of about 2.2 mgd of wastewater (based on 2015 flows). This is down from 2.6 mgd in 2010.

Annual average daily (AAD) flows are projected based on historic populations, as discussed in the previous section, and historic flows. Historic population and AAD flow data is used to develop wastewater flows in gallons per capita per day (gpcd). Per capita flows are then used in conjunction with projected future populations to calculate future AAD flows. Historic AAD flows for the last thirteen years as well as corresponding populations and calculated per capita flows are shown in **Table 3-1**.

Table 3-1. Historic Annual Average Daily (AAD) Flows

Year	Population	AAD Flow (mgd)	Per Capita Flow (gpcd)
2001	25,219	2.52	100
2002	25,533	2.5	98
2003	25,587	2.58	101
2004	25,842	2.52	98
2005	26,666	2.51	94
2006	26,515	2.57	97
2007	27,944	2.49	89
2008	28,631	2.57	90
2009	29,335	2.55	87
2010	30,056	2.62	87
2011	30,597 [1]	2.46	80

2012	31,148	[1]	2.50	80
2013	31,708	[1]	2.30	73
2014	32,279	[1]	2.31	72
			14-Year Average	89.0
			14-Year Median	89.5

1. Population based on 1.8% growth projection

The per capita flow estimates in the above table should be qualified for several reasons. They are based on a smoothed estimate of service population although the actual population changes were probably more abrupt; this could explain the annual variation in per capita flow contributions. In addition, historic populations from 2007 and prior include only the population within the corporate City limits, as mentioned previously. Because the actual total population served was higher, the per capita flow estimates are conservative. The 2010 population and per capita flow rate is based on the total population including NAWS, and is therefore a more realistic representation of per capita flow rates. Per capita flow rates after 2010 may be lower than actual as the population projection is based on average population growth rates and not actual data. Population increases shown for 2011 – 2013 may be more than the actual increase, resulting in an artificially low per capita value. It should also be noted that new housing added to the City is now required to use water saving plumbing fixtures and appliances. As the percentage of newer housing increases with time, the per capita flow contribution should decline. The overall trend shown in the table above seems to support this concept. As shown in

Table 3-1, the average per capita flow rate over the past thirteen years (with the qualifications mentioned) was about 90 gpcd. However, since this value is not based on total population numbers for the majority of the decade, the 2010 per capita flow rate of 87 gpcd may more realistically represents the current per capita flow rate. Future projections for new populations are expected to be at a reduced per capita flow rate due to water conservation requirements in new housing.

The per capita flow estimates are based on the assumption that the NAWS will not generate significant industrial flow. Historically, minimal industrial flows have been reported from NAWS. The City of Ridgecrest also does not have a significant industrial flow contribution.

3.1.2 Average Day Maximum Month Flow

Wastewater treatment facilities are often designed based on the average day maximum month (ADMM) flows, meaning that the facility will have the capacity to treat the average daily flows from the month producing the maximum wastewater flows.

Table 3-2 includes average monthly flow data from January 2009 through December 2013, to show the fluctuation in flows throughout the year. A graph of flows during the same time period is shown in **Figure 3-1.** The future ADMM is projected based on a historic ratio of ADMM flow to average annual daily (AAD) flow. The design ADMM to AAD ratio, as was determined based on an analysis of five years of plant flow records from 2010 through 2014. The data is summarized in **Table 3-3**. The ADMM to AAD ratio proposed for design is 1.1 based on the five year average flow and the highest ADMM flow during that time period.

Table 3-2. Average Monthly Flows (January 2009 through December 2014)

	Average Monthly Flow (mgd)					
	2009	2010	2011	2012	2013	2014
January	2.41	2.69	2.42	2.54	2.09	2.26
February	2.49	2.68	2.41	2.54	2.08	2.30
March	2.51	2.61	2.37	2.54	2.25	2.34
April	2.55	2.69	2.43	2.50	2.30	2.41
May	2.64	2.71	2.50	2.51	2.30	2.37
June	2.41	2.62	2.48	2.49	2.39	2.25
July	2.47	2.67	2.50	2.54	2.52	2.31
August	2.61	2.71	2.55	2.58	2.49	2.37
September	2.68	2.70	2.48	2.53	2.47	2.39
October	2.59	2.60	2.40	2.42	2.22	2.23
November	2.65	2.44	2.51	2.41	2.24	2.21
December	2.63	2.35	2.49	2.35	2.26	2.27

Figure 3-1. Historic Flows (January 2009 Through December 2014)

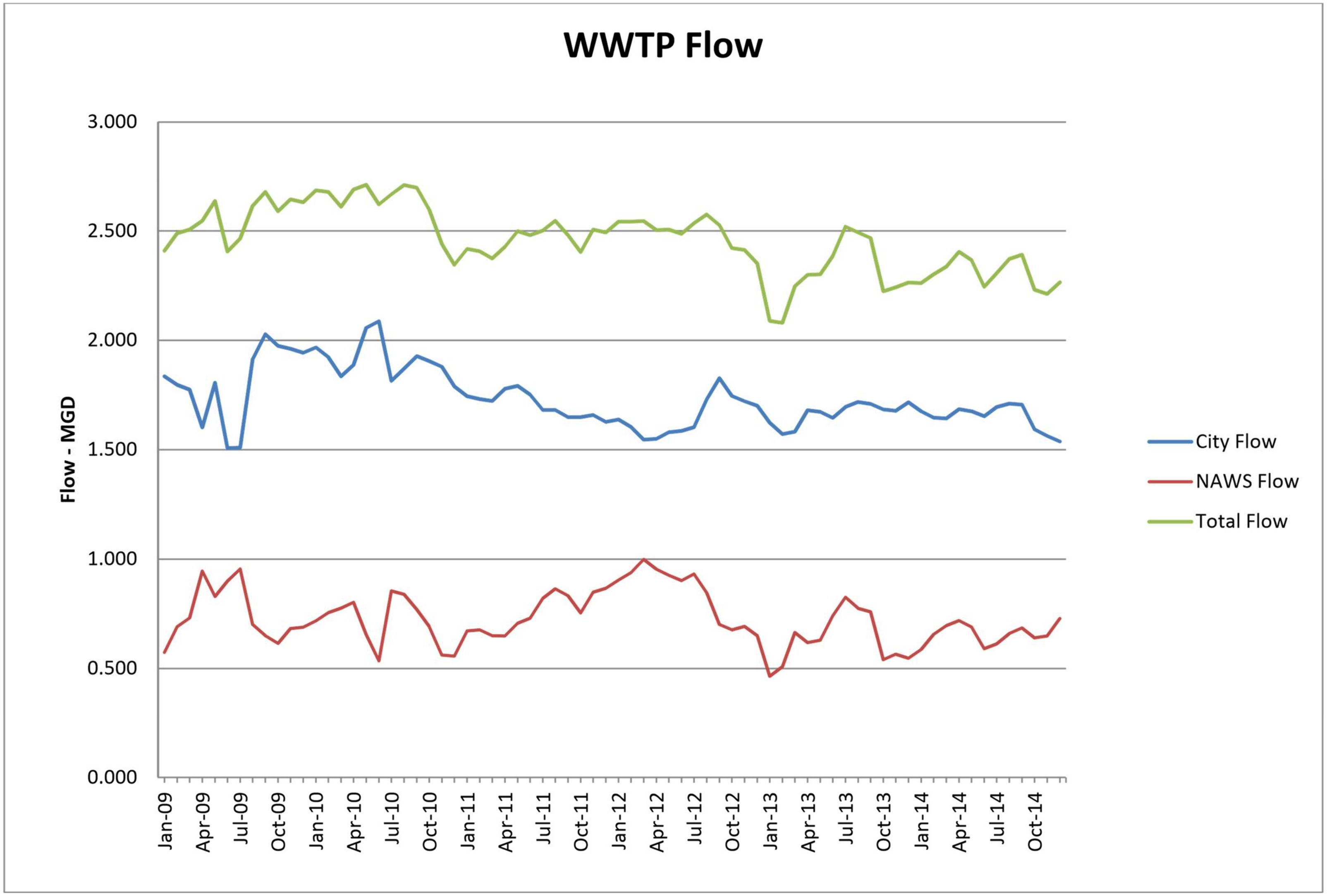


Table 3-3. Historic AAD and ADMM Flows

Year	AAD Flow (mgd)	ADMM Flow (mgd)	ADMM:AAD Factor
2010	2.62	2.71	1.03
2011	2.46	2.55	1.04
2012	2.50	2.58	1.03
2013	2.30	2.52	1.10
2014	2.31	2.41	1.04
5 –year	2.46	2.71 max	1.10
Average			1.07

3.1.3 Peak Hourly Flow

Flow through processes that are hydraulically critical, such as headworks, influent pumping, splitter boxes, pipelines, flow meters, and valves, must be sized based on the peak hourly flow (PHF). The historic PHF to AAD flow ratio is 1.9, as supported by analysis of five years of plant flow records.

3.1.4 Inflow/Infiltration (I/I) Evaluation

The groundwater table is approximately 100 feet below the level of the collection sewers and thus infiltration is minimal. A review of the influent flow records does indicate an inflow issue during periods of rainfall. As much as a 25 percent increase in flows has been correlated with a rainfall event. For example, on December 20, 2010, flows at the WWTP spiked at 3.12 mgd. Flows on December 19 and December 20, 2010 were 2.48 mgd and 2.52 mgd, respectively. However, the total annual inflow is minimal in comparison with the total flow, and it is therefore concluded that excessive infiltration/inflow does not exist in the City of Ridgecrest; no corrections to the wastewater collection system are required to be funded with the proposed WWTP project. The City will, however, continue to monitor and maintain the collection system and repair any locations identified as contributing to I/I. The City has recently video inspected the entire sewer system to assess its condition.

3.2 Projected Influent Flows

The 2013 daily flow to the City of Ridgecrest WWTP was approximately 2.30 mgd. The flow originates from a projected population of approximately 31,700 persons, yielding an average per capita flow of 73 gpcd. The projected population figure may be higher than actual as it was developed based on a project growth rate of 1.8 percent applied to the 2010 census population. This would drive the per capita wastewater flow lower than actual.

Current building codes require the use of water conserving fixtures in all new housing units. In addition, existing homes will reduce indoor water consumption as they replace old fixtures and old appliances. It is therefore assumed that the future growth will have a lower per capita flow than experienced prior to 2010. For planning purposes, this report assumes a per capita flow contribution of 85 gpcd for future growth. Implementation of the California 20:20 plan, which mandates a 20 percent reduction in water consumption by 2020 will also result in reduced flow from the existing population. It is not known how much this may reduce wastewater flow, as much of the savings could occur form reduced landscape irrigation. If significant in-home savings are achieved, additional wastewater treatment capacity may be available longer and thereby extend the period between wastewater facility expansions.

Table 3-4 presents a summary of future flow projections based on the assumed population growth and per capita flow through year 2050.

Table 3-4. Projected Influent Flows

	Population	AAD Flow (mgd)	ADMM Flow (mgd)	PH Flow (mgd)
2015	32,860	2.8	3.1	5.6
2020	35,926	3.1	3.4	6.2
2025	39,287	3.3	3.6	6.6
2030	42,942	3.7	4.1	7.4
2035	46,949	4.0	4.4	8.0
2040	51,329	4.4	4.8	8.8
2045	56,118	4.8	5.3	9.6
2050	61,354	5.3	5.8	10.3

Based on the projected influent flows summarized in Table 3-4, it is recommended that a new WWTP be constructed in phases, with a first phase ADMM design flow of 4.0 mgd. The Phase 1 expansion will provide capacity sufficient for growth through approximately 2030. The AAD flow rating will be 3.6 mgd. Design provision for a future plant expansion to 5.9 mgd (ADMM) with a corresponding 5.4 mgd AAD flow is proposed and is projected to accommodate flows through approximately year 2050. Capacity requirements for the plant expansion should be reevaluated at the time of the expansion based on updated population and flow projections. A PHF design of 10.3 mgd will be used for hydraulically critical components. The AAD flow capacity will be used to size wastewater disposal and recycling facilities.

3.3 Existing Plant Retirement

Once the first phase construction of the new secondary process of the WWTP is complete, the existing plant will be retired. Decommissioning of the existing plant will include:

- Removal and disposal of wastewater sludges, solids, and biosolids from the site.
- De-energizing electrical service to all components and demolition and disposal of all above grade electrical equipment.
- Demolish and disposal of all process mechanical equipment and removal of above grade piping.
- Demolish and disposal of framed structures.
- Demolition of concrete structures including puncturing the bottoms such that they drain, demolition of above grade structures, and filling existing basins with concrete rubble and earth fill.

Facilities to be incorporated into the replacement WWTP include the facultative ponds and effluent storage ponds.

After the existing WWTP is demolished at the NAWS site, the tertiary treatment process will be constructed (if the NAWS site is selected as the site for the WWTP).

3.4 Distribution of Wastewater Flow

Section 2.1 described the two gravity sewer service areas existing in the City of Ridgecrest. Flows from each sewer service area will vary with changes in population within the area. According to an analysis of flow records since 2010, approximately 70 percent of the current flow is attributed to the City of Ridgecrest service area, while the remaining 30 percent is from China Lake NAWS service area. Of the 70 percent attributed to the City, a portion to the south flows naturally past the City site and a portion to the north flows naturally to the NAWS site as shown on **Figure 2-1**.

Projected flows for each sewer service area based on current wastewater flows from the City, currently developed and undeveloped acreage within the City, and general topography of the City are summarized in **Table 3-5**. The southern sewer service area (Sewer Service Area #2) has been defined to capture wastewater flowing by gravity toward the City site. The northern watershed area (Sewer Service Area #1) includes all of China Lake NAWS, plus flows from developed parts of the City located north of Drummond Avenue, which flow by gravity to the existing WWTP. Other flow generated within the City north of Ridgecrest Blvd and east of China Lake Blvd naturally flows to the NAWS site, bypassing the City site. The watershed areas were determined based on the naturally occurring drainage, assuming 100 percent capture of the wastewater flowing toward each of the sites. Those flows naturally captured at the City site (Sewer Service Area #2) will flow by gravity to the NAWS site if the City site is not utilized, as is currently done. Wastewater from Sewer Service Area #1 would need to be pumped to the City site if the new WWTP is located on the City site. Flow projections are based on the assumption of 1.8% annual population growth occurring in both service areas.

Table 3-5. Flows by Sewer Service Area @ 2050 Population Projection

	Population	AAD Flow (mgd)	ADMM Flow (mgd)	PH Flow (mgd)
Sewer Service Area #1	20,719	1.8	2.0	3.5
Sewer Service Area #2	40,635	3.5	3.9	6.7
Total	61,354	5.3	5.8	10.3

3.5 Influent Loadings and Plant Performance

Wastewater is typically characterized by its 5-day biochemical oxygen demand (BOD_5), total suspended solids (TSS), and nitrogen content. Historic influent BOD loading and historic effluent BOD and removal rates are summarized in **Table 3-6** and **Table 3-7**.

Table 3-6. Historic Influent BOD Loading

	Annual Average		Maximum Month		ADMM:AAD
Year	mg/L	ppd	mg/L	ppd	Factor
2005	158	3,307	200	4,187	1.27
2006	150	3,215	200	4,287	1.33
2007	166	3,438	250	5,659	1.65
2008	138	2,839	200	4,937	1.74
2009	132	2,814	230	5,198	1.85
2010	150	3,259	400	9,111	2.80
2011[1.]					
2012[1.]					
2013	111	2,129	188	3,951	1.86

2014	162	2,121	211	4,241	2.00
Average	146	2890	234	5196	1.61

1. Information unavailable

Table 3-7. Historic Effluent BOD and Removal

	Annual Average		
Year	mg/L	ppd	Percent Removal
2005	17	356	89%
2006	16	343	89%
2007	18	366	89%
2008	14	295	90%
2009	14	307	89%
2010	8	165	95%
2011[1.]			
2012[1.]			
2013	11	211	93%
2014	9	173	94%
Average	13	277	91%

1. Information unavailable

The average BOD removal rate from 2005 through 2014 was approximately 91 percent.

3.6 Projected Influent Loadings

Influent BOD concentrations are projected to remain similar to current loadings. As shown in Table 3-6 the average day maximum month BOD concentration averaged approximately 234 mg/L with a very high level in 2010. A maximum month concentration of 270 mg/L is proposed be used for future projections to provide for conservative design.

Projected influent BOD loading through 2050 is shown in **Table 3-8**. There is no data available on total suspended solids (TSS) loading. It is proposed that the TSS loading be 250 mg/L based on typical values for domestic wastewater. Total nitrogen loading is proposed to be based on 45 mg/L based on composite sampling completed in September 2015. The average total N and TKN was 39 mg/L based on 3 samples taken over a 3 week period.

Table 3-8. Projected Influent BOD Loading

	Projected ADMM Flow	Projected ADMM Loading	
Year	mgd	mg/L	ppd
2015	3.1	270	6,981
2020	3.4	270	7,656
2025	3.6	270	8,106
2030	4.1	270	9,232
2035	4.4	270	9,908
2040	4.8	270	10,809
2045	5.3	270	11,935
2050	5.8	270	13,060

3.7 Historic and Projected Biosolids Production

The present WWTP has a single primary digester and a single secondary digester, each 40 feet in diameter. Both are anaerobic digesters. The two digesters are adequate for the primary sludge generated at the present facility. Regardless of the treatment process chosen for the new facility, the mass of biosolids to be treated and disposed annually will be significantly greater than at present.

The present plant collects and processes only the sludge generated from primary treatment, which includes that portion of the BOD and suspended solids that is easily settleable. Sludge generated from biological treatment currently remains in the facultative lagoons at the NAWS site and is not collected. The biological portion of the sludge will be collected from the new WWTP, along with the settleable portion currently collected. The future volume of biosolids collected will therefore be significantly greater than that currently collected. The following **Table 3.9** summarizes the portion of the historic biosolids production that is currently collected, and **Table 3-10** summarizes projections of sludge generated by a typical secondary treatment plant at the flows and loadings projected for the City of Ridgecrest.

Table 3-9. Historic Biosolids Production

Year	AAD Flow (mgd)	Biosolids (dry tons/yr)	Biosolids (lbs/MG)1.
2005	2.51	47	103
2006	2.57	33	70
2007	2.49	26	57
2008	2.57	30	64
2009	2.55	42	90
2010	2.62	30	62
2011	2.46	42	97
2012	2.50	35	77
2013	2.30	39	93
2014	2.31	36	85

[1.] Includes only solids from primary clarifier, solids from secondary process currently remain in ponds.

Table 3-10. Projected Biosolids Production

(mgd)	AAD Flow (dry tons/yr)	Biosolids (lbs/MG)[1.]	Biosolids Year
2020	3.1	362	640
2025	3.3	385	640
2030	3.7	432	640
2035	4.0	467	640
2040	4.4	514	640
2045	4.8	561	640

2050	5.2	607	640

1. Includes solids from secondary process.

As defined in **Table 3-10**, the new facility will require significantly greater capacity to treat and handle biosolids than the current operation. The new WWTP, by the year 2025 will produce over 10 times the average sludge production of the current WWTP.

4 EXISTING WASTEWATER TREATMENT FACILITIES

4.1 General

The City's existing WWTP is located approximately 3.5 miles northeast of the City center, on the China Lake NAWS base. The WWTP was originally constructed in 1946. In 1976, Clarifier No. 3 was constructed, and the headworks was upgraded in 2006. The WWTP facilities include a headworks, primary sedimentation tanks, facultative oxidation ponds, and evaporation/percolation ponds. Primary sludge is treated through two anaerobic digesters and biosolids are dewatered and dried on unlined solar drying beds. Selected design criteria for the existing WWTP and effluent disposal system are summarized in **Table 4-1**. A process flow diagram of the existing WWTP is included in **Figure 4-1**.

As discussed earlier, the present facility was constructed and placed in service during the mid-1970's; capital costs at that time were shared between the City and the Navy, and all associated debt for the capital costs has now been retired.

Table 4-1. Selected Design Criteria - Existing WWTP

Item	Unit	Design
General		
Design Flow (ADMM)	mgd	3.6
Design Peak Hour Flow	mgd	5.7
Influent and Effluent Flow Metering		
Influent Parshall Flume (No. 3)		
Throat Size	Inches	12
Capacity	mgd	10.4
Effluent Parshall Flumes (Nos. 1 and 2)		
Throat Size	Inches	18
Capacity, each	mgd	15.9
Headworks		
Auger Grinders (2), each	mgd	3.6
Vortex Grit Chamber	mgd	7.2
Grit Classifier	mgd	7.2
Primary Sedimentation Tanks		
Primary Clarifiers 1 and 2		
Length, each	Feet	66
Side Wall Depth, each	Feet	10

Overflow, maximum rate	gal/sf/day	600-1,200
Detention Time, each	Hours	1-3
Primary Sedimentation Tanks, Con't.		
Primary Clarifier 3 (not in operation)		
Diameter	Feet	55
Side Wall Depth	Feet	10
Overflow, maximum rate	gal/sf/day	600-1,200
Detention Time	Hours	1-3
Primary Anaerobic Digester Tank		
Diameter	Feet	40
Hydraulic Retention Time	Days	10-20
Volatile Solids Loading	lbs/day/cf	0.1-0.4
Secondary Anaerobic Digester Tank		
Diameter	Feet	40
Solids Retention Time	Days	30-60
Volatile Solids Reduction	%	50-60
Secondary Treatment –Facultative Ponds		
Unit A – Ponds 1-4	acres	62
Unit B – Ponds 5-7	acres	52
Minimum pond depth	feet	5
Sludge Beds		
Total Area (8 beds)	sq ft	14,100
Effluent Pump Station (to City site)		
Number of Pumps		1
Size, each	hp	25
Effluent Disposal – Irrigation and Evaporation		
Effluent Disposal Ponds (9 & 10)	acres	72
Effluent Disposal Ponds – Out of Service (8 & 11)	acres	53
Effluent Disposal Ponds at City Site (5)	acres	10.5
Alfalfa Reclamation Area	acres	33.3
Total effluent to golf course	ac-ft/yr	750

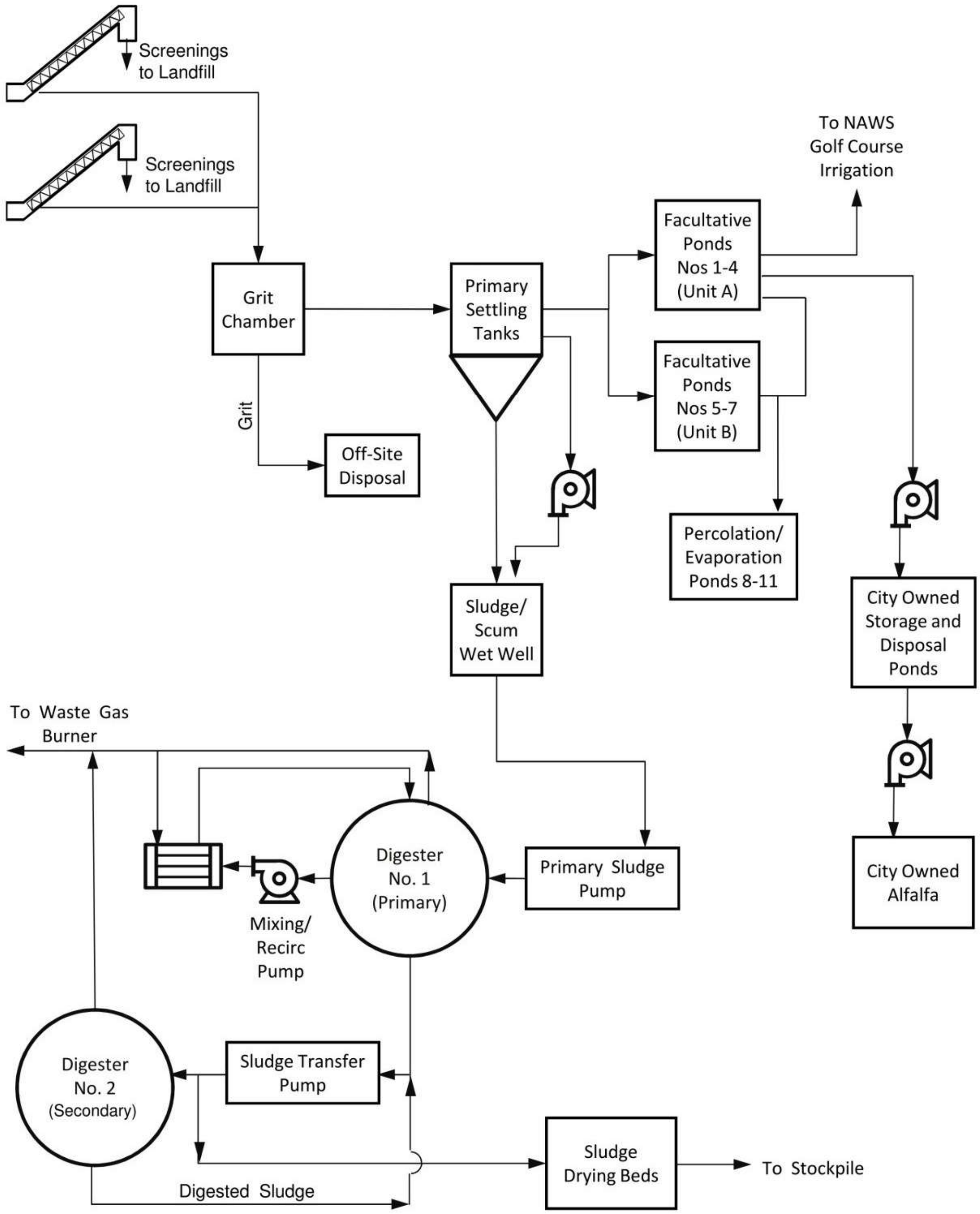


Figure 4-1. Existing WWTP Process Flow Diagram

4.2 Wastewater Treatment Plant

4.2.1 Influent and Effluent Flow Metering

Plant influent flow from the City's sewer trunk main is measured before entering the headworks through a 12-inch throat Parshall Flume (Flume No. 3), shown in **Figure 4-2**. Total plant flow is measured through two 18-inch throat Parshall Flumes located between the primary sedimentation tanks and the facultative ponds. Flume No. 1 measures the flow to pond Unit A (Ponds 1-4) and Flume No. 2 measures flow to pond unit B (Ponds 5-7). A photo of one of the effluent Parshall flumes is shown in **Figure 4-3**. Flow from China Lake NAWS service area is not directly measured, but is calculated by subtracting the City flow measured in Flume No. 3 from the total flow measured by Flumes No. 1 and 2.

Figure 4-2. Influent Parshall Flume (Flume 3)

Figure 4-3. Effluent Parshall Flume (1 of 2)

4.2.2 Headworks

The headworks facility includes two influent channels, each with an auger grinder; they are followed by a single vortex grit removal system and grit classifier; from there, flow passes to a group of primary sedimentation tanks (clarifiers). The headworks is designed to handle peak flows of up to 7.2 mgd through the two channels. Photographs of the influent channels and auger grinders are shown in **Figure 4-4** and **Figure 4-5**, respectively. The headworks was upgraded in 2006. The existing WWTP headworks does not include influent pumping; the entire flow through the WWTP to the facultative ponds and the NAWS disposal ponds is by gravity flow. The existing headworks is hydraulic capacity limited. Occasional short term flooding can occur during high flow.

4-4. Influent Channels

Figure 4-5. Auger Grinders Figure

4.2.3 Primary Sedimentation Tanks

Primary sedimentation facilities include three rectangular clarifiers (Tanks No. 1, 2 and 4) and one circular clarifier (Tank No. 3). The rectangular clarifiers are shown in **Figure 4-6**. Tanks No. 1 and 2 were constructed in 1946 and Tank No. 3 in 1976. A fourth clarifier (Tank No. 4) was also constructed in 1946, but has been retired from service. The three remaining tanks are beyond their expected life due to concrete degradation, worker safety concerns, and old/obsolete equipment. If the tanks are retained in service for some time interval, extensive improvements are recommended to avoid frequent service outages. Access to valves for sludge pumping is poor and in its present form is unsafe for entry without proper confined space entry permits. The flights and chains wear excessively for unknown reasons and require overhauling to extend the life of the system.

Figure 4-6. Primary Sedimentation Tank

4.2.4 Anaerobic Digestion

Two 40-foot diameter anaerobic digesters (primary and secondary) are used to reduce volatile organic solids (VOCs) in the primary waste sludge. Both digesters have floating covers; the primary digester is heated and mixed to increase efficiency. Gas from the digesters is used to fuel the hot water heat exchanger, which is used to heat the digester contents. The digesters are also beyond their expected life. If retained in service, even for a limited time, extensive improvements are recommended to avoid future service outages. The digester floating roof structure is in particularly poor, highly corroded condition and is in need of immediate replacement. **Figure 4-7** and **Figure 4-8** illustrate the extensive corrosion on the digesters.

Figure 4-7. Corrosion On Digester Cover

Figure 4-8. Anaerobic Digester

4.2.5 Primary Sludge Pumping

A primary sludge pump station collects sludge from the clarifiers and pumps to the digesters. The pumps lie below grade in a confined space, with very poor access, as shown in **Figure 4-9**. Routine servicing and maintenance of the units poses a worker safety concern. If the primary clarifiers are retained in service, even for a limited time, extensive pumping station improvements are recommended to avoid frequent service outages and worker safety concerns.

Figure 4-9. Primary Sludge Pump Station

4.2.6 Facultative Ponds

The primary effluent from the clarifiers flows by gravity to the facultative ponds. Flow is split and diverted either to pond Unit A (Ponds 1-4) or Unit B (Ponds 5-7). The seven facultative ponds total approximately 114 acres. Effluent in Unit A begins in Pond 1 and flows through Pond 2 to Pond 4. From Pond 4, effluent may either flow to the evaporation/percolation ponds or to Pond 3 before being pumped for irrigation of the China Lake Golf Course, or by the City for irrigation at the City site. Flow through Unit B is discharged to the evaporation/percolation ponds. The facultative ponds are not aerated except for one small aerator located in Pond 3 (**Figure 4-10**). The ponds are reported to be clay lined, limiting infiltration and percolation.

Figure 4-10. Facultative Pond No. 3

4.2.7 Sludge Solar Drying Beds

Sludge from the anaerobic digesters is applied to eight solar sludge drying beds, totaling approximately 14,100 square feet (**Figure 4-11**). The sludge drying beds provide sludge storage during the winter when weather conditions do not support sludge drying, and sufficient drying capacity during summer months to account for sludge accumulated during the winter as well as freshly digested sludge. The resulting biosolids are stockpiled on site and tested before they are land applied on the City's agricultural fields.

Figure 4-11. Sludge Solar Drying Beds

4.2.8 Effluent Pumping

A pump located at Pond 3 delivers the treated wastewater effluent through a 4-mile long, 20-inch diameter force main to the City site. The force main discharges into one of 4 ponds. A pump located at the City site delivers effluent to a center pivot irrigation system for alfalfa crop irrigation. The City currently irrigates approximately 33 acres of alfalfa hay, which is cut, dried and baled for sale.

A second pump at Pond 3, operated by the Navy, is used to deliver effluent for turf irrigation at the China Lake Golf Course. The Navy constructed a series of pressure sand filters and a chlorination system to provide disinfection. Due to the high algae content of the effluent in Pond 3, these filters have never operated successfully and are currently bypassed. Chlorine gas is fed from one ton cylinders. Disinfection occurs at a concrete chlorine contact structure. It is reported that very high doses of chlorine are needed to meet the RWQCB and Title 22 State Water Resources Control Board, Division of Drinking Water (formally the California Department of Public Health - CDPH) disinfection requirements. The City, by agreement with the Navy, must supply 750 acre-feet of effluent to the Navy for golf course irrigation or other uses.

4.2.9 Overall Plant

In general the existing WWTP is beyond its expected life. The major components, with the exception of the headworks, were constructed from 40 to over 70 years ago. Most facilities of similar vintage have long since been retired from service. Various improvements may be implemented to extend the life of the existing plant for another 5 to 10 years, but it is not anticipated to be functional, nor sufficient in capacity, beyond that time period. Even with selected improvements, the overall plant will still have many components of the original plant. In many cases, the real condition cannot be assessed because those components such as pipe and buried structures cannot be evaluated without complete exposure. The plant structural components most likely do not meet current building code requirements and may be vulnerable to damage from earthquakes. The electrical infrastructure does not meet current code requirements and is generally inadequate for any significant improvements.

4.3 Effluent Storage, Reuse, and Disposal

Effluent disposal currently occurs in three different mechanisms, at several locations:

- Percolation from ponds at the existing treatment plant and the City site, plus leaching from irrigated lands. Percolated water will ultimately end up in the underlying groundwater. Percolation rates are highly variable, and depend on the nature of soils, the underlying geology, and the level of maintenance provided to the percolation pond bottoms. Percolation rates will be discussed further in Section 7. Existing ponds on the City site are shown on Figure 1-3. Existing ponds on the NAWS site are shown on **Figure 4-12**.
- Evaporation from existing China Lake NAWS ponds and ponds at the City site. Evaporation rates from open ponds have been closely studied for desert areas. The Western Regional Climate Center (WRCC), Desert Research Institute (DRI) publishes evaporation rates for various regions. Published pan evaporation rates from www.wrcc.dri.edu were used for this site. According to WRCC, evaporation from a basin is approximately equal to 0.7 or 0.8 times the pan evaporation for the region. A value of 0.8 times the pan evaporation was assumed for this analysis. Evaporation rates for Ridgecrest are among the highest in California. The City maintains a Class A pan evaporation measurement station at the NAWS site. Evaporated water is lost to further beneficial use.
- Evapo-transpiration (ET) for irrigated crops. Evapo-transpiration is a term used to describe the combination of evaporation and plant transpiration (the release of water from plant leaves into the atmosphere). The quantity of water used by crops has been studied for desert areas. The Cal Poly Irrigation Training and Research Center (ITRC) presents an accepted summary of transpiration (uptake) rates, which were used in this analysis. Water transpired and evaporated in the process of irrigating crops is a beneficial use of water.

It is noted that the use of effluent for irrigation also demands that storage be provided. In practice, some of the flows generated during winter months cannot be immediately used to meet agronomic plant needs during winter, and must be stored for application during the growing season. Although this seasonal storage component of the present facilities is not quantified at present, it is in fact provided by the present ponds at the existing WWTP site. If irrigation continues to be a method of effluent disposal, then additional seasonal storage will be necessary.

A 1993 Memorandum of Agreement between the China Lake Naval Air Weapons Station and the City of Ridgecrest (Agreement) establishes policies and procedures whereby the City is allowed to use the 20-inch effluent pipeline from the NAWS site to the City site. The existing 20-inch effluent pipeline from the NAWS ponds to the City site is shown in **Figure 4-13**. This Agreement also allows the NAWS to ensure that treated effluent of appropriate quality and quantity are available, through pond seepage, to the nearby endangered fish species, the Mohave tui chub, and its wetland habitat in Lark Seep, to the north of the existing WWTP ponds.

At present, the City uses four different disposal sites with the following acreages:

Table 4-2. Current Effluent Disposal Areas

Site	Approximate Acreage
WWTP Ponds @ NAWS[1.]	186 in use (239 total)
China Lake Golf Course and Ponds (irrigated portion)	70
City Site- Ponds	11
City Site- Alfalfa irrigation	33

[1.] Ponds 8 and 11 are not currently used due to excessive seepage from those ponds. Ponds 1-7 evaporate only.

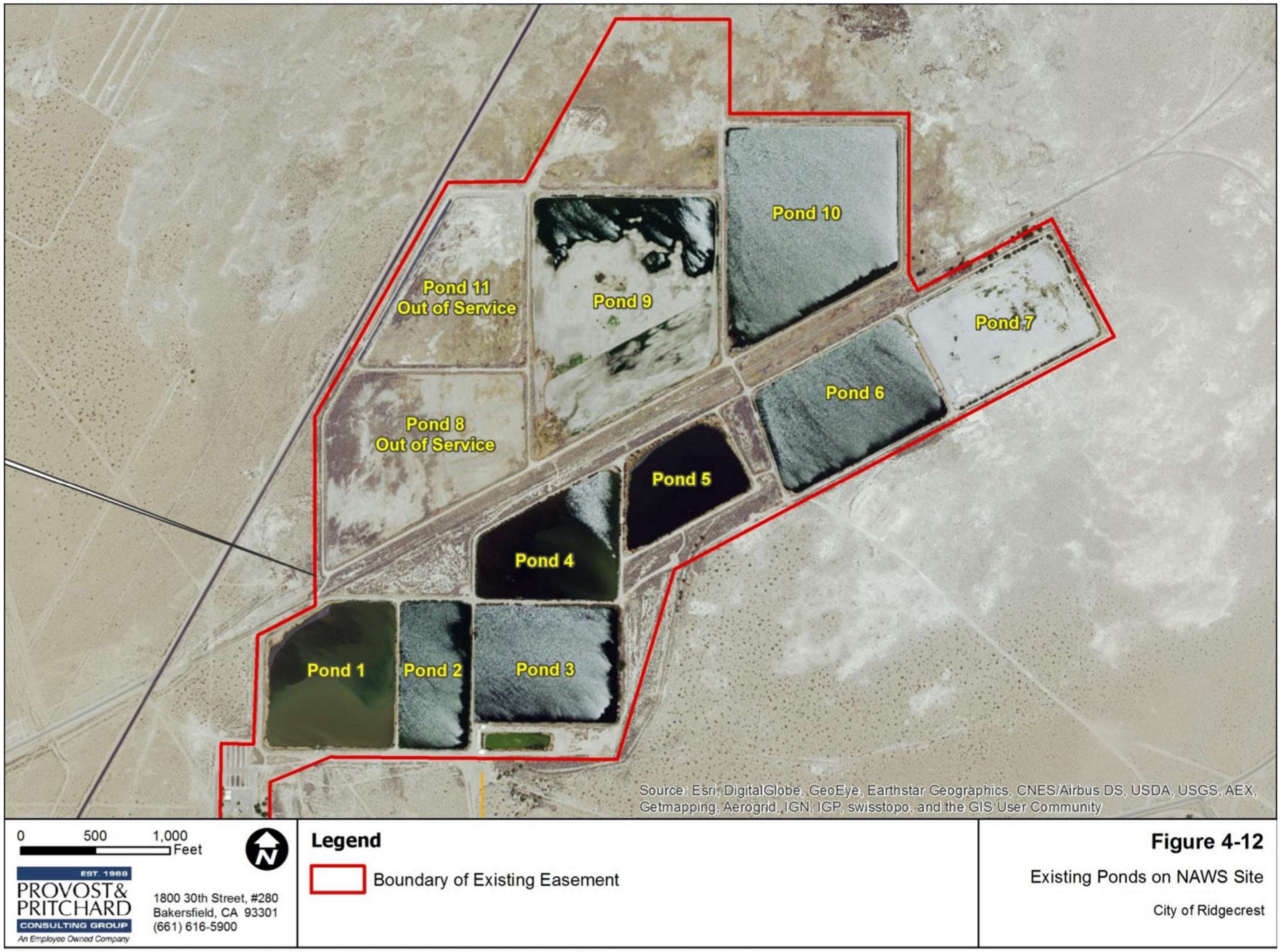


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Figure 4-12. Existing Ponds on NAWS Site

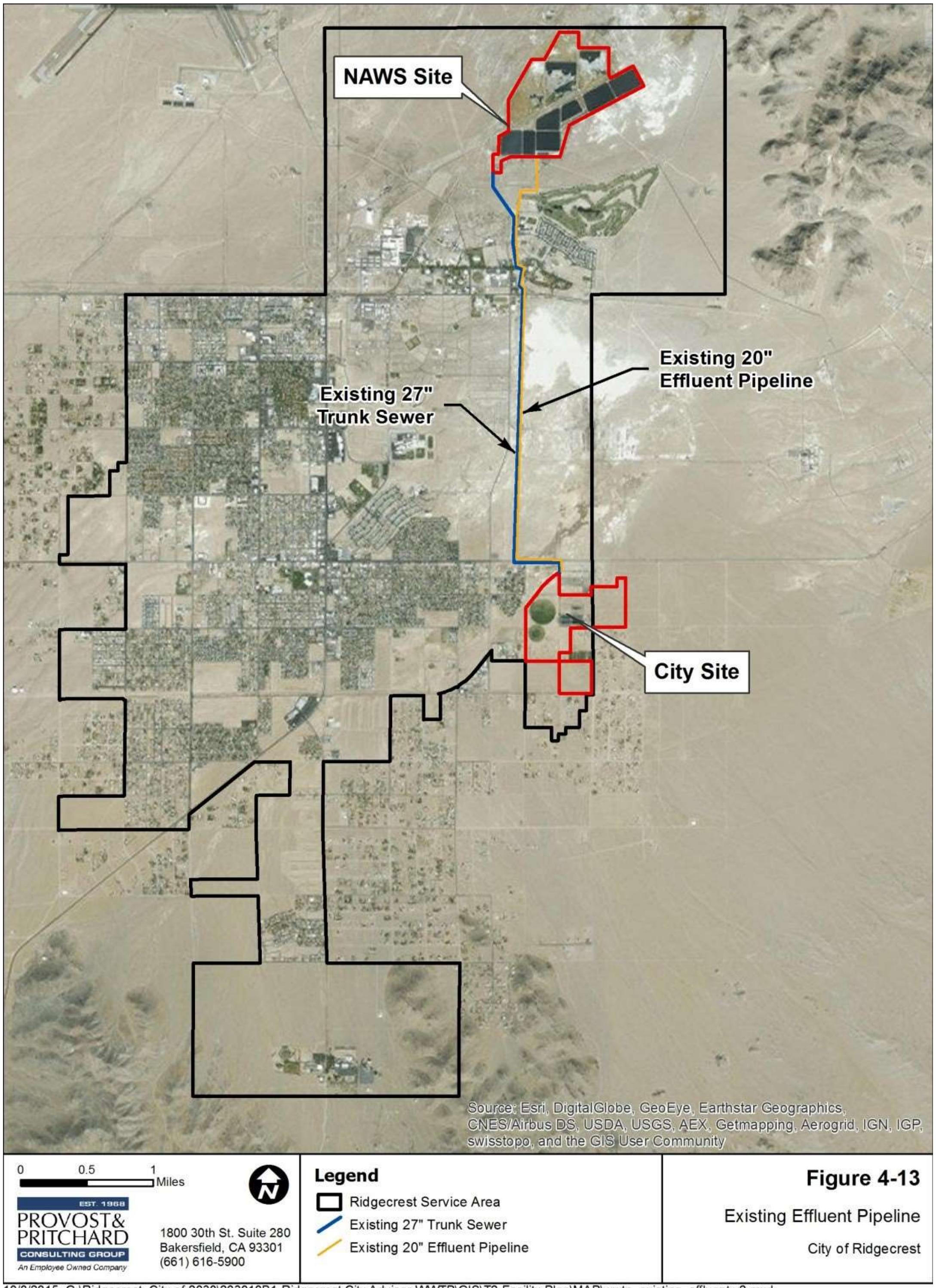

Figure 4-13

Existing Effluent Pipeline

City of Ridgecrest

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Figure 4-13. Existing Effluent Pipeline

5 EXISTING AND FUTURE REGULATORY REQUIREMENTS

5.1 Introduction and Summary

Wastewater treatment facilities in California are regulated by Regional Water Quality Control Boards (RWQCB) through the issuance of permits entitled “Waste Discharge Requirements” (WDRs). The local RWQCB is responsible for developing and issuing WDRs to wastewater treatment facilities that discharge to land and groundwater, and National Pollutant Discharge Elimination System (NPDES) permits for facilities that discharge to surface waters. The RWQCB and the Division of Drinking Water jointly regulate projects that use recycled water, with the RWQCB being responsible for issuing recycled water permits. The RWQCB is also responsible for approving sludge disposal applications for dischargers in California. The Lahontan RWQCB, located in Victorville, regulates waste discharges from the City of Ridgecrest.

The City of Ridgecrest WWTP currently discharges all of its effluent to land through a series of percolation/evaporation ponds, golf course irrigation, and alfalfa irrigation.

5.2 Current Waste Discharge Requirements

The existing WWTP currently operates under Waste Discharge Requirements (WDR) Board Order No. 6-00-56. The WDR set limits on pollutants in the wastewater effluent discharged from the WWTP with the goal of protecting public health and beneficial uses of the groundwater. WDR 6-00-56 regulates only that effluent discharged to the evaporation/percolation ponds located on the NAWS site. Wastewater recycling requirements for the China Lake Golf Course are established under separate WDR Board Order No. 6-84-36, and discharge to the City of Ridgecrest irrigation site (also the City site) is regulated under separate WDR Board Order No. 6-93-85.

Copies of the WDR’s are included in Appendix A.

5.3 Future Waste Discharge Requirements

5.3.1 Discharge Limits

The effluent quality for land application must meet the objectives developed in the RWQCB Lahontan Regional Basin Plan (Basin Plan), which contains water quality objectives for both surface and groundwater. The Basin Plan addresses constituents in the discharge that are potentially harmful to beneficial uses of the groundwater.

5.3.2 Groundwater Limits

Groundwater is the sole source of potable water supply for the City of Ridgecrest and China Lake NAWS, and is therefore designated for municipal use. The Basin Plan includes water quality objectives for waters designated as municipal supply. Since the groundwater is designated as a municipal supply, the discharge of wastewater effluent must not cause the groundwater under or around the WWTP and discharge areas to exceed the Maximum Contaminant Levels (MCLs) for drinking water standards. These MCLs are specified in the Title 22, California Code of Regulations.

Specifically associated with wastewater, the nitrogen concentrations in the effluent shall not cause the underlying groundwater to exceed the MCL for nitrate concentrations. Nitrogen removal to less than 10 mg/L will likely be required to protect the groundwater quality in the Ridgecrest area. Under current operations, nitrogen removal is not specifically required by the WDRs.

5.3.3 Monitoring Requirements – Anti-Degradation Analysis

Protecting underlying groundwater is a key objective of RWQCBs in California. Monitoring and studies are usually necessary to show that the groundwater below and near the WWTP and discharge areas will be protected. In order to show that discharge from the WWTP will not unacceptably degrade the groundwater, the background groundwater quality must first be characterized.

In order to characterize the background groundwater quality and determine potential degradation, dischargers are often required to install a network of groundwater monitor wells approved by the RWQCB. Proposed WWTP improvements must also be shown to be in compliance with the Best Practicable Treatment and Control (BPTC) measures. Groundwater discharge limits need to reflect implementation of BPTC, with respect to various constituents of concern that will be identified in the evaluation.

The BPTC policy is the result of the SWRCB Resolution No. 68-16, known as the "Anti-Degradation Policy". Resolution No. 68-16 states the following:

1. *Whenever the existing quality of water is better than the quality established in policies as of the date on which such policies become effective, such existing high qualities will be maintained until it has been demonstrated to the State that any change will be consistent with maximum benefit to the people of the State, will not unreasonably affect present and anticipated beneficial use of such water and will not result in water quality less than that prescribed in the policies.*

2. *Any activity which produces or may produce a waste or increased volume or concentration of waste and which discharges or proposes to discharge to existing high quality waters will be required to meet waste discharge requirements which*

will result in the best practicable treatment or control of the discharge necessary to assure that (a) a pollution or nuisance will not occur and (b) the highest water quality consistent with maximum benefit to the people of the State will be maintained.

As stated above, Resolution No. 68-16 dictates that where waters are of higher quality than required by State policies, such higher quality shall be maintained. BPTC applies to both treatment and control of wastewater. Treatment processes and facilities at the WWTP must be designed to remove constituents from the wastewater to levels that will not degrade the quality of receiving groundwater.

5.4 Water Quality Objectives

The effluent quality from the proposed new WWTP must meet the objectives developed by the Basin Plan, as discussed in Section 6.3.

5.5 Best Practicable Treatment and Control

The existing network of monitor wells has demonstrated the adequacy of the present treatment and disposal system to protect the groundwater. A new facility will be selected to provide equal or greater levels of treatment, and should therefore be expected to satisfy the BPTC requirements for Ridgecrest.

5.6 Recycled Water Regulations

The water recycling regulations dictating effluent quality criteria are contained in the California Code of Regulations, Title 22, Division 4, Chapter 3, Section's 60301 through 60355. The regulations are intended "...to establish acceptable levels of constituents of recycled water and to prescribe means for assurance of reliability in the production of recycled water in order to ensure that the use of recycled water for the specified purposes does not impose undue risks to health."

5.6.1 Recycled Water Quality

The Division of Drinking Water has established four levels of recycled water quality that are appropriate for different end uses. The differing qualities require different levels of treatment. The four categories of recycled water that are currently permitted are referred to as follows:

- Disinfected tertiary (highest quality category).
- Disinfected secondary-2.2 (high- intermediate quality).

- Disinfected secondary-23, (low-intermediate quality, currently required at the China Lake Golf Course).
- Undisinfected secondary (lowest acceptable quality category - as now produced at the existing WWTP for alfalfa irrigation).

Article 3 of the Water Recycling Criteria details the acceptable uses of recycled water and the corresponding quality of water required for each such use. Acceptable uses include irrigation of various crops and landscapes, impoundment, and cooling.

5.6.2 Un-Restricted Use for Landscaping, Park and School Use

Recycled water specifically for the irrigation of un-restricted use parks and playgrounds, school yards, residential landscaping, and golf courses must be disinfected tertiary recycled water. Disinfected tertiary treated water is defined as wastewater that has been treated using, oxidation, coagulation and filtration, and is subsequently disinfected, producing an effluent with total coliform bacteria not exceeding a most probable number (MPN) of 2.2 per 100 milliliters of water. The turbidity of the filtered wastewater must also meet specified limits.

5.6.3 Food Crop Irrigation

Recycled water used for the irrigation of food crops where the irrigation water comes into contact with the consumed portion of the crop must also be disinfected tertiary treated water. If the consumed portion of the food crop is produced above ground and recycled water does not contact the edible portion of the food crop, then disinfected secondary-2.2 water may be used as a minimum standard. Although undisinfected secondary treated water is allowed by Title 22 for orchard and vineyard irrigation, the Food and Drug Branch (FDB) of the Division of Drinking Water has taken the position that undisinfected secondary recycled water is not suitable for these crops. The FDB stated that "…orchard and vineyard crops will quite likely come into contact with recycled water or soil irrigated with recycled water through typical harvesting practices..." The FDB therefore recommends that orchard and vineyard crops be irrigated with disinfected secondary-2.2 recycled water, at a minimum. Disinfected secondary-2.2 treated water is recycled water that has been oxidized and disinfected so that the median concentration of total coliform bacteria in the disinfected effluent does not exceed an MPN of 2.2 per 100 milliliters utilizing the bacteriological results of the last seven days for which analyses have been completed, and the number of total coliform does not exceed an MPN of 23 per 100 milliliters in more than one sample in any 30 day period.

5.6.4 Nursery and Restricted Access Golf Course Irrigation

Recycled water used to irrigate unrestricted access ornamental nursery stock and recycled water used to irrigate restricted access golf courses must be treated to a minimum level of disinfected secondary-23. Disinfected secondary-23 recycled water is recycled water that has been oxidized and disinfected so that the median concentration of total coliform does not exceed an MPN of 23 per 100 milliliters, and the number of total coliform bacteria does not exceed an MPN of 240 per 100 milliliters in more than one sample in any 30 day period. “Restricted access golf course” means a golf course where public access is controlled so that areas irrigated with recycled water cannot be used as if they were part of a park, playground, or school yard, and where irrigation is conducted only during periods when the golf course is not being used by golfers. The China Lake Golf Course currently uses disinfected secondary-23 recycled effluent.

5.6.5 Non Food Crops

Non food-bearing trees, fodder and fiber crops, seed crops, and food crops that are required to undergo commercial pathogen-destroying processing before being consumed, may be irrigated with recycled water with a minimum standard of undisinfected secondary recycled water. Un-disinfected secondary recycled water is defined as oxidized wastewater. This includes fodder, fiber and pasture for animals not producing milk for human consumption. It also includes ornamental nursery stock or sod farms provided that no irrigation with recycled water occurs 14 days before harvesting or retail sale. Alfalfa irrigation is included within this category.

5.7 Probable Future Discharge Regulations

It is assumed that with the proposed project, the RWQCB will impose limitations, requiring BOD, TSS and Total nitrogen to effluent levels of 10 mg/L or below.

5.8 Biosolids Disposal Regulations

The City of Ridgecrest WWTP generates biosolids which are currently disposed by land application. Biosolids disposal is regulated by both Federal and State regulations, as discussed in this section.

5.8.1 Federal Regulations

The Federal Sewage Sludge Regulations, 40 CFR 503, came into effect in 1994. These regulations establish requirements for the facilities that produce sewage sludge, as well as the land appliers. 40 CFR 503 includes pollutant limits, operational standards, management practices, and monitoring, record keeping, and reporting requirements.

The Federal Regulations impose three major restrictions that must be implemented to qualify for land application of sludge.

1. The sludge must be within the maximum limitations for ten metals;
2. The sludge must not pose a public health risk, and must therefore satisfy pathogen reduction requirements; and
3. Provisions must be made to reduce the attraction for vectors, such as insects or animals that may transmit infectious agents.

5.8.2 General Order

In 2004, the SWRCB adopted general WDRs for the discharge of biosolids as a soil amendment. The WDRs contained in Water Quality Order No. 2004-0012-DWQ (General Order), are intended to simplify the regulatory process for land application sites throughout the state.

While 40 CFR 503 is self-implementing and does not require an application or preapproval, in accordance with the General Order, a Notice of Intent (NOI) must be submitted to the RWQCB for approval. The General Order imposes the 40 CFR 503 pollutant limits, as well as some additional limitations. Sludge must be monitored annually for pesticides and chlorinated hydrocarbons in addition to metals and nutrients. The owners of the property on which the land application occurs are ultimately responsible for ensuring compliance with the General Order.

Kern County has an ordinance that prohibits the discharge of biosolids to land within the County of Kern jurisdiction. The City can, however, discharge biosolids to land within the City limits.

6 TREATMENT ALTERNATIVES

6.1 General

This section reviews and evaluates various secondary and tertiary wastewater treatment process alternatives. Effluent disposal, biosolids disposal and wastewater site alternatives will be discussed in subsequent sections.

While current effluent disposal methods, with the exception of the China Lake Golf Course, are suitable for un-disinfected secondary effluent, the City of Ridgecrest may decide to recycle wastewater effluent as reuse opportunities develop. When it is feasible for the City to recycle wastewater, the existing and new percolation ponds can be used for effluent storage.

Title 22, which regulates the use of recycled water, requires redundancy in treatment processes and components and also requires alarms and other provisions to insure the safety and quality of effluent. It is the intent to include the required redundancy, alarms and other components to the secondary and tertiary treatment systems so that tertiary quality effluent can be supplied when appropriate. The secondary treatment process must also be capable of nitrogen removal to meet anticipated water quality objectives for protection of groundwater.

6.2 Design Criteria

Design criteria for the analysis of treatment alternatives is presented in **Table 6-1**.

Table 6-1. Design Criteria

Design Criteria	Phase 1	Phase 2
Average Annual Daily Flow (AAD)	3.6 mgd	5.4 mgd
Average Day Maximum Month Flow (ADMM)	4.0 mgd	5.9 mgd
Maximum Day Flow (MDF)	4.7 mgd	7.1 mgd
Peak Hour Flow (PHF)	6.8 mgd	10.3 mgd
Average Influent BOD_5 Loading @ ADMM Flow	270 mg/L 9,000 ppd	270 mg/L 13,300 ppd
Average Influent TSS Loading @ ADMM Flow	270 mg/L 9,000 ppd	270 mg/L 13,300 ppd
Average Influent TKN Loading @ ADMM Flow	40.5 mg/L 1,350 ppd	40.5 mg/L 2,000 ppd
Effluent BOD_5	10 mg/L	10 mg/L

Effluent TSS	10 mg/L	10 mg/L
Effluent Ammonia	1 mg/L	1 mg/L
Effluent Total Nitrogen	8 mg/L	8 mg/L

6.3 Facilities Common to all Alternatives

Several facilities are common to all of the WWTP process alternatives. These common components are summarized as follows:

Table 6-2. Summary of Components Common to all Alternatives

Component	No. of Units[1]	Dimensions or Design Capacity per Unit
Influent Pump Station	1	10.3 mgd
Influent Pumps	3 (4)	2,375 gpm
Headworks/Mechanical Screens	1	10.3 mgd
Grit Removal	1	10.3 mgd
Septage Receiving Station	1	30 loads/month
Operations/Lab Building	1	2,175 SF
Maintenance Building	1	2,720 SF
Electrical Building	1	680 SF
Effluent Pump Station	1	1.8 mgd
Effluent Pumps	2	1,250 gpm
Site Improvements	1	N/A

[1]Future criteria at 5.4 mgd are presented in parentheses.

6.3.1 Influent Pumps and Headworks

The headworks is the general term for the structures and process equipment that receives wastewater flow from the trunk sewers at the head of the WWTP. A headworks typically provides grinding or screening, flow metering, grit removal, and influent pumping. The proposed headworks facility will screen and remove plastics and non-degradable objects from the wastewater. Material removed by the screen and the grit chamber at the headworks will be washed to remove organics, compacted, and deposited in a container, and periodically hauled off-site for landfill disposal. The headworks will also include the influent lift station to lift the wastewater to the subsequent treatment process. This allows the headworks to be at a moderate depth, reducing construction costs and operation and maintenance issues. The influent lift station will include three variable speed drive solids handling pumps in Phase 1 (3.6 mgd), with a fourth pump added in Phase 2 (5.4 mgd). The headworks structure will be designed to handle the peak hour flow for Phase 2 without further expansion. Flow meters will be installed to measure and record City and NAWS flows entering the WWTP. The headworks will also include a septage receiving station.

6.3.2 Operations and Maintenance Buildings

An operations building will be constructed to include office space with provision for SCADA controls and alarms, laboratory, restroom with shower, and a lunch/conference room. A separate maintenance building will also be provided to provide storage for equipment, spare parts and vehicles will also be provided for maintenance, repair and tool storage. The maintenance shop will include a bridge crane. A 2,175 square foot office/lab building and a 2,720 square foot maintenance building is planned. A separate 680 square foot building will be constructed to house electrical switchgear, motor controls and variable speed drives.

6.3.3 Site Improvements

The WWTP will include an all weather access road, paved parking area, security fencing, limited outdoor lighting, stormwater drainage facilities, gravel surfacing and other site improvements. Potable water from the NAWS or City water systems will be extended to the site. Power and communications service will be brought to the site.

6.3.4 Emergency Power

A diesel engine emergency power generator will be provided for the plant. It will be sized to operate all essential pumps, process equipment, and control systems.

6.3.5 Biosolids Handling Facilities

All process alternatives will produce biosolids that will require stabilization, dewatering and drying. The biosolids produced from the treatment processes will be digested and stabilized before dewatering and disposal. Currently at the existing WWTP, sludge from only the primary clarifiers is collected, digested and pumped to sludge drying beds for dewatering. Biosolids are also generated in the facultative treatment lagoons, but it remains in those basins.

There are two types of digestion processes: aerobic and anaerobic. The aerobic digestion process keeps the sludge in an aerobic environment by introducing dissolved oxygen, usually by blowers and coarse bubble diffusers. The reduction of the volatile solids concentration is achieved by oxidizing microbial protoplasm, which releases the energy for microbial cell functions. This endogenous respiration process is efficient in breaking down microbes in the sludge.

Dissolved oxygen is excluded in an anaerobic system, instead, anaerobes access oxygen from sources by breaking down the organic material itself. When the oxygen is derived from the organic material itself, the volatile solids are broken down to the 'intermediate' products like alcohols, aldehydes, and organic acids along with carbon dioxide. In the presence of methanogens, the intermediates are converted to the 'final' end products of methane, carbon dioxide with trace levels of hydrogen sulfide. The majority of the energy contained within the volatile solids in sludge is released by methanogenic bacteria as methane, which can be used for digester heating and electricity generation.

Anaerobic digestion usually requires a longer sludge age and larger tank volume for the same amount of sludge flow and volatile solids reduction rate. It also requires additional equipment and processes to treat the biogas for both hydrogen sulfide and moisture to protect the heating and electricity generation equipment. The operation cost is relatively low without the need of aeration, but routine monitoring of pH and alkalinity levels is needed from the operator for maintain the required operating conditions. Anaerobic digestion is much more difficult to operate than aerobic digestion.

At an ADMM design flow of 4.0 mgd with no primary clarification, anaerobic digestion would consist of two enclosed circular tanks as opposed to the two open top rectangular tanks for aerobic digestion. The size of the digesters will depend on the process selected, but the capital cost of the anaerobic process would be approximately $3 million more than the cost of the aerobic process. However, the operation and maintenance costs would be approximately $31,000/year lower for the anaerobic digesters due primarily to the aeration requirements of the aerobic digester. The biogas from anaerobic digestion can also be used to heat the digester and potentially produce electricity.

The capital cost of anaerobic digestion is significantly greater than for aerobic digestion. The benefit of anaerobic digestion is the potential for electric energy production when looked at over a 20 year lifecycle. However, most WWTP's of this size utilize aerobic digestion due to its simplicity and reduced non-energy operations costs. Cogeneration with digester gas has been problematic for many plants because of lower than expected gas generation and difficulty in maintaining engines and effective scrubber systems to clean digester gas prior to combustion. Thus anaerobic digestion is generally a better candidate for larger WWTPs.

6.3.6 Biosolids Disposal

After the biosolids have been stabilized and dewatered, it will require disposal. The disposal method will be permitted by the RWQCB. A discussion of potential disposal methods is presented in Section 8 of this report.

6.4 Candidate Secondary Wastewater Treatment Alternatives

Secondary wastewater treatment is primarily a biological process where naturally occurring microorganisms are encouraged to grow and consume the organic matter (BOD reduction), and transform the chemical nature (nitrification/denitrification) of the wastewater. As mentioned in Section 4, all selected processes will be designed and operated for biological nitrogen reduction in anticipation of future regulations.

6.4.1 General Overview of Biological Processes

Biological treatment of City wastewater presently occurs within a series of facultative ponds, in which bacteria slowly consume organic contaminants. The resulting new bacteria gradually accumulate on the floor of the pond, where they degrade and decompose. A facultative pond system provides acceptable levels of treatment, with several notable drawbacks:

- Pond systems require a large acreage.
- Ponds are subject to occasional odors, especially in spring and autumn, as water temperatures change and the ponds “turn over”.
- Ponds naturally produce algae; for the NAWS irrigation system, algae quickly became an obstacle to filtration prior to golf course irrigation. Algae will result in high TSS concentrations and make subsequent tertiary treatment more costly.
- Few methods of control exist for facultative ponds; the process cannot be easily adjusted for upsets or effluent quality concerns.
- Nitrogen reduction with ponds is difficult to obtain on a consistent basis.

As the Ridgecrest service area grows, reliable use of facultative ponds for the entire flow becomes more and more difficult. Use of a mechanically-assisted type of treatment process with a smaller footprint is necessary. Small footprint, mechanically based biological treatment processes are usually a variation of the “Activated Sludge” process. Many different arrangements of the process exist.

The activated sludge process is an aerobic biological treatment system. The system consists of two primary components: a reactor basin and a solids separator with a return activated sludge (RAS) system. The solids separator can be a clarifier or membranes, either internal or external to the reactor basin. The activated sludge process uses microorganisms in suspension to oxidize soluble and colloidal organic solids. Oxygen is required to support the biological reactions. A reactor basin containing “activated sludge” consisting of wastewater solids and micro-organisms, receives wastewater which is mixed and aerated to oxidize the waste. This process allows for growth of more microorganisms and production of carbon dioxide and water. The solids are separated by a clarifier or membrane and returned to the aeration basin as RAS. The clarifier overflow or membrane permeate is the treated effluent. Solids are periodically removed as waste activated sludge (WAS) to maintain the desired mass of solids in the aeration basin. Design considerations include hydraulic retention time (HRT), solids retention time (SRT), mixing/aeration regime, reactor solids concentration, RAS rate, and WAS scheduling.

Some forms of activated sludge considered use the extended aeration activated sludge (ExAAS) process. ExAAS uses long solids retention time (SRT), low food to microorganism ratio (F/M) and high solids concentration to create a robust stable process that is easy to operate. The ExAAS process is available in a number of generic and proprietary designs. In some cases, the ExAAS process does not require subsequent digestion of the biosolids.

Nitrogen removal occurs by internally recycling oxidized (nitrified) wastewater (where ammonia and other nitrogen forms have been oxidized to nitrate) through an anoxic zone where denitrification occurs. An anoxic zone is nearly free of dissolved oxygen but is continuously mixed. Nitrogen is consumed by the specific types of bacteria predominant in this zone. resulting in these bacteria releasing nitrogen gas to the atmosphere. With proper arrangement of tanks and equipment, the nitrification/denitrification process can be used with most activated sludge processes.

Eight (8) different secondary processes have been initially considered for the new Wastewater Treatment Plant:

1. Sequencing Batch Reactor (SBR)
2. Complete Mix Activated Sludge (CMAS)
3. Extended Aeration Activated Sludge - Oxidation Ditch
4. Extended Aeration Activated Sludge - Pond Based (Biolac™)
5. Extended Aeration Activated Sludge - Aeromod Sequox®
6. Rotating Fixed Film/Activated Sludge - STM-Aerotor™
7. Membrane Bioreactor (MBR)
8. BioFiltro (earthworms)

Except for the BioFiltro process, the other 7 listed process alternatives differ in the reactor basin configuration, aeration system, flow regime, structure type, biological growth rate, method of solids separation and method of returning solids. The SBR is a batch process with intermittent filling and discharge of a set of parallel process tanks. The next six processes are continuous flow and also, except the MBR process, include secondary clarification. Rectangular secondary clarifiers were assumed as this typically allows common wall construction with secondary process basins. However, this assumption will be further explored during the final design process as circular clarifiers provide advantages in process operations. The MBR process does not require clarifiers because it uses membranes for solid/liquid separation. It, however requires finer preliminary screens. Each secondary treatment process considered is described below.

The BioFiltro process is unique and the particulars of this technology are discussed in Section 7.4.9.

6.4.2 Sequencing Batch Reactor

The Sequencing Batch Reactor (SBR) system is a fill-and-draw, batch activated-sludge treatment process. Typically the system consists of five steps in sequence, for each reactor tank:

FILL: Fill occurs in two steps, mixed fill (anoxic) and react fill (aerated). In react fill, aeration continues until organic biodegradation is complete and ammonia has been converted to nitrate.

SETTLE: All influent wastewater and air is stopped. Solids separation occurs leaving a clarified, treated effluent above the sludge blanket. The incoming wastewater is diverted to a parallel tank train.

DECANT: A floating, solids-excluding decanter draws water from beneath the surface so that floatables are not entrained. No turbulence is allowed to occur and the secondary treated supernatant is drawn off.

IDLE: This is an idle time period used for process adjustment.

NITROGEN REMOVAL: Biological removal of nitrogen occurs when the micro organisms do not have sufficient oxygen present. Under these conditions, a second colony of microorganisms develops, that utilizes nitrate in place of oxygen and releases nitrogen as a gas. This “denitrification” step occurs in the SBR after the react phase, and continues during the decant and idle portions of the sequence. The SBR is particularly effective in nitrogen reduction, because the duration of each of the process phases is independently adjustable.

The primary advantage of an SBR system is that separate stages of the treatment process take place at different times within the same tank, thus eliminating the need for separate primary and secondary clarifiers and associated return sludge pumping, and reducing the physical footprint and related equipment cost. However, the sequencing batch operation of this system makes process controls more complex. SBR systems rely on systems of mechanical floats, electronic level sensors, timers, automated valves, and start/stop of a number of blowers and pumps. All of these are controlled by sophisticated control logic residing in a master computer PLC. The need for operator training is higher for SBR facilities than conventional plants of similar size. This is due to the level of automated valves and equipment provided, and the need for maintenance of those devices. Also, due to changes in hardware and software and factory support, the SBR is likely to require a change in operating software or hardware at approximately 5 to 8 year intervals.

In addition to the complexity of the SBR system, because it is a batch flow system (not continuous), it would likely require a post equalization tank prior to tertiary treatment.

Because flow is not continuous, pipes and hydraulic structures must be oversized compared to continuous flow processes.

6.4.3 Complete Mix Activated Sludge (CMAS)

In a conventional activated sludge plant, the primary treated wastewater and acclimated micro-organisms (activated sludge) are mixed and aerated in a basin or tank. Primary clarifiers are often included to reduce the initial organic loading on the aeration basins, reducing the amount of air required which lowers electricity consumption. After a sufficient aeration period, the activated sludge solids are separated from the wastewater by gravity separation in a secondary clarifier. The clarified wastewater continues through the plant for further treatment or discharge. Most of the settled sludge is returned as RAS to the inlet of the aeration basin where it mixes with the raw wastewater entering the basin. The remaining sludge (WAS) is wasted to the sludge handling portion of the treatment facility. The recirculated ratio (RAS to raw wastewater flow) is adjustable, and is selected to produce the maximum removal of organic material from the wastewater. Recirculation varies from 30 to 100 percent of the raw wastewater flow, depending on treatment conditions and wastewater characteristics. A separate anoxic basin and recirculation pumps will also be required for nitrification and denitrification to occur. The HRT is lower for conventional CMAS than for ExAAS thus reducing the size of the basin. The use of primary clarifiers can reduce organic load, and thus CMAS would have lower energy cost than ExAAS. The CMAS system however is less stable and more challenging to operate than ExASS because it is more heavily loaded.

6.4.4 Extended Aeration Activated Sludge (ExAAS) Oxidation Ditch

Extended Aeration Activated Sludge - Oxidation Ditch. This process is widely used in smaller wastewater treatment plants (less than 5 mgd) because of its reliability, ease of operation, and consistent performance. No primary clarification is needed for an oxidation ditch provided that there is effective preliminary treatment and removal of screened solids. It can provide nitrification and denitrification, produces a stable sludge and provides a high quality effluent in terms of BOD and TSS. The effluent is suitable for tertiary treatment and/or direct disinfection. The predicted effluent quality is 10 to 20 mg/L BOD and TSS.

Oxidation ditch treatment facilities differ from SBR’s in two functional areas: biological unit volume, and clarification capability. The Oxidation Ditch uses an oval shaped reactor basin with a center partition wall. Wastewater that enters the basin is aerated while it circulates around the partition wall. A separate, mixed, anoxic zone or basin is used for nitrogen removal. The aeration system can use mechanical aerators (vertical or brush) or diffused air with mixers. “Extended aeration” treatment processes such as the oxidation ditch contain a larger mass of microorganisms (larger process tanks) to better accommodate both flow and loading fluctuations. The overall volume of liquid in the ditch is usually equal to about 18 to 36 hours of flow (hydraulic retention time), somewhat greater than an SBR. The ditch supports an active culture of microorganisms for contaminant removal, and discharges into a separate clarifier for settling. As with the CMAS, most of the settled solids are returned to the ditch as RAS and a small portion is wasted as WAS.

6.4.5 ExAAS Biolac™

The ExAAS - Biolac pond based system is an activated sludge process that uses lined ponds as the aeration basin. The large pond volume provides the capacity to handle organic loading without the use of primary clarifiers. The Biolac system is a proprietary extended aeration activated sludge process utilizing fine bubble diffused aeration in lined ponds. Secondary clarifiers are provided to separate and return activated sludge (RAS) to the aeration basin. The design criteria are similar to that of the oxidation ditch; however the basin configuration, hydraulic regime, and aeration/mixing system are much different. The hydraulic retention time is about 24 to 48 hours. The system operates at very low food to microorganism ratio (F/M) and operates with a long solids retention time (SRT), thus producing a very stable sludge. The secondary clarifiers can be constructed internal to the aeration basin or can be external.

Pad mounted blowers provide air to the system. The fine bubble membrane diffusers are attached to floating aeration chains, which naturally oscillate by the air released from the diffusers. The moving diffusers provide efficient mixing of the basin contents as well as high oxygen transfer at low energy usage. The air supply chains, with submerged diffusers, float in the pond, without contacting or harming the basin liner or eroding an unlined basin bottom. Each diffuser chain air supply can be individually controlled by an air valve, providing flexible adjustment in air supply. In normal operation, aerated and anoxic zones alternate along the length of the basin, allowing nitrification/denitrification to occur simultaneously in one basin.

The Biolac process provides long hydraulic and solids retention time, making it a relatively stable process with comparatively less sludge generation. However, the long sludge age can make the clarifying process more troublesome. The Biolac system is slightly more complex than a conventional oxidation ditch, but it is still relatively simple to operate and requires less operator training than more complex or operator intensive systems, such as the SBR.

6.4.6 ExAAS - Aeromod Sequox®

Aeromod Sequox is a proprietary activated sludge process consisting of an anoxic selector, two stage aeration, clarifier, and sludge digester, all integrated in one compartmented concrete basin. The Sequox process provides flexible operation in one simple structure, therefore reducing yard piping, electrical runs, and transfer pump station.

However, the layout of the concrete tank makes expansion more difficult. Additional treatment capacity for future phases must be built-in during initial construction, or a new plant would be required when additional capacity becomes necessary.

6.4.7 STM-Aerotor™

STM-Aerotor is a proprietary process which combines an activated sludge process and fixed film process in one tank. The fixed film media supplement the need of nitrification and denitrification. Mounted on a center shaft, hollow media capture air as the shaft turns, drawing it down into the bottom of the tank. The air is slowly released as coarse bubble aeration. Oxygen transfer efficiency compares favorably to coarse bubble aeration, but without the expense of running blowers or diffusers. Fixed film growth on the media is in an ideal environment for nutrient (nitrogen) removal, and stays optimally reactive due to the roiling waters of the tank.

Clarifiers following the STM aeration basin settle the solids and produce secondary effluent discharging to storage and disposal ponds. The STM-Aerotor process uses fixed film media in combination with suspended activated sludge to remove nitrogen in one tank. However, the process is sensitive to the dissolved oxygen level in the tank and needs additional attention from the operator in order to achieve simultaneous nitrification/denitrification.

While the STM-Aerotor can be designed to allow easy expansion by an addition of a treatment train, the STM-Aerotor process requires installation of additional Aerotors with complete set of motor, gear box, and chains to be expanded, making it more equipment intensive as it scales up to a larger capacity. The STM-Aerotor is best suited to small plants, usually less than 1 mgd capacity.

6.4.8 Membrane Bioreactor

The Membrane Bioreactor (MBR) process consists of a suspended growth biological reactor integrated with a membrane filter system. Essentially, the membrane system replaces the solids separation function of secondary clarifiers and tertiary filters in a tertiary activated sludge system. The membrane filter employed typically has pore sizes of about 0.1 micron, which ensures that no particulate matter is discharged in the effluent. The MBR process produces the highest quality effluent of all alternatives considered and does not require a separate tertiary filtration process. Disinfected tertiary recycled water can be produced directly by adding a disinfection process.

The process consists of an anoxic tank, pre-aeration tank, and membrane tanks where membrane cartridges are submerged. Membranes are available in either flat plate or tube technologies. Through the use of a suction pump, a vacuum is applied to a header connecting the membranes. Water is suctioned through the membranes, into the pump and then discharged to the disinfection processes. The concentrated mixed liquor in the membrane tank is recirculated to the anoxic tank or pre-aeration tanks as return sludge for denitrification. Because the membrane pores are small, the effluent generated is extremely clean, with turbidity often lower than 0.2 NTU. No subsequent filtration process is needed, and disinfection is sufficient to allow unrestricted use (disinfected tertiary) classification of effluent.

The membrane technology effectively overcomes the problems associated with poor settling of sludge in conventional activated sludge processes and can operate at a much higher biomass concentration, therefore reducing the tank volume and physical footprint dramatically. However, these bonuses come at a cost, with MBR having the highest capital and operations and maintenance costs. MBR systems also require additional and more advanced screening as primary treatment. MBR is the latest technology in activated sludge treatment, and therefore has fewer installations and less operational history than the other alternatives. It is the most advanced treatment process on the market, and therefore requires the highest operator training classification. The MBR process is often best considered when tertiary treated effluent is desired for the entire flow stream.

6.4.9 Biofiltro

The Biofiltro process uses beds of sawdust populated with earthworms to achieve secondary treatment and nitrogen removal. This process requires the construction of concrete basins that include an underlying bed of river rock covered with a layer of sawdust. The sawdust layer is seeded with earthworms. Wastewater is applied to the beds via a sprinkler system. The wastewater flows through the beds and out the bottom of the basin. The beds require the removal of worms and castings as part of the ongoing maintenance of the system, along with the replacement of the sawdust. This removal operation is provided by Biofiltro under their license and annual operations agreements with the facility owner.

Two of Biofiltro's pilot projects were observed, one at the City of Firebaugh treating domestic wastewater and a second at the Fresno State University Dairy treating dairy wastes. Both are very small scale pilot projects. Biofiltro has no large full scale treatment plants in the US treating domestic wastewater and has only one similar sized (to the City's wastewater treatment needs) facility. This facility is in Chile and treats waste from a peach processing plant, not municipal waste.

Headworks and screening requirements are similar to other alternatives discussed above. Details of the sprinkler systems used by Biofiltro indicate that additional filtration of the influent wastewater may be required to avoid clogging of the sprinkler system. According to information provided by Biofiltro, the system can provide secondary wastewater effluent with effluent nitrogen below 10 mg/L.

Biofiltro requires a license agreement for its technology and a minimum of a 10-year operations agreement. The operations agreement provides for removal of the worms and castings on an annual basis, replacement of the removed sawdust and some technical support for operation of the facility.

The Biofiltro system is land intensive requiring a minimum of 8 acres for the first phase (3.6 mgd AAMM) and 11 acres for the second phase (5.4 mgd AAMM). While the first phase might be accommodated on either the City or the NAWS sites, the second phase would require the acquisition of additional land at either of the sites.

6.5 Viable Secondary Wastewater Treatment Processes

Based on a preliminary analysis, STM-Aerotor, Aeromod Sequox, MBR treatment and Biofiltro processes were dropped from further consideration. The Biofiltro, STM-Aerotor and the Aeromod Sequox processes are proprietary. The MBR process is more expensive to build and operate than the other processes. Although the MBR provides a higher quality effluent (tertiary quality) the City's current disposal methods do not justify the added investment. The Biofiltro process is unproven in the US and for municipal wastewater treatment at this size facility. Costs and land requirements for the Biofiltro system (including filtering the influent wastewater to avoid clogging of the sprinkler system) are higher than other alternatives. The SBR, Complete Mix Activated Sludge, Oxidation Ditch, and Biolac treatment processes are considered proven, viable, competitive alternatives and will be further discussed and analyzed in this section.

6.5.1 Sequencing Batch Reactor

The SBR process has been described above. The components required for a flow of 3.6 mgd SBR process are summarized in **Table 6-3**.

Table 6-3. SBR Component Summary

Component	No. of Units[1]	Design Capacity per Unit
Aeration Basin and Equipment	4 (6)	0.90 mgd 63'x60'x22' deep HRT 20 hrs SRT 15 days
Primary & Secondary Clarifier	Not required	N/A
RAS/WAS Pump Station	Not required	N/A
RAS/WAS Pumps	Not required	N/A
Sludge Digester	2 (3)	100' diameter

Blower Building	1	2,500 SF

[1]Future criteria at an ADMM of 5.4 mgd are presented in parentheses.

As proposed for the City of Ridgecrest, the SBR in Phase 1 (3.6 mgd plant) will consist of four reactors, each with a capacity of 0.9 mgd. Two additional cells will be added for Phase 2. Effluent from the SBR will be discharged directly to percolation / evaporation ponds and/or storage. Each of the process alternatives will also include influent pump station and headworks, office/lab building, sludge dewatering, sludge thickening, anaerobic digestion, and effluent disposal, as described in Section 6-3.

The SBR process has a relatively small volume of cell mass available to accommodate fluctuations in flow and waste strength, it is a less stable process than the Oxidation Ditch or Biolac processes. The shorter detention time makes the process less forgiving or resistant to shock loadings. The SBR process is relatively new and more complex than the other processes. This process does not utilize separate clarifiers. Clarification is internal to the aeration basin, and is therefore not as effective at separating clear liquid from the return solids as those processes that provide separate clarifiers. The process has a good track record of producing low effluent BOD, TSS and Nitrogen. Because SBR is a batch process, it will require more operator attention than a continuous flow process. There are many suppliers of SBR equipment and thus the system can be competitively bid. Tertiary treatment is easily added on to an SBR process.

6.5.2 Complete Mix Activated Sludge

The conventional complete mix activated sludge (CMAS) process was described above. The components required for the 3.6 mgd CMAS process are summarized in **Table 6.4**.

Table 6-4. CMAS Component Summary

Component	No. of Units[1]	Design Capacity per Unit
Aeration Basin and Equipment	2 (3)	1.8 mgd 82’x55’x16’ deep HRT 10 hrs
Anoxic Basin	2 (3)	0.4 mgd 20’x45’x18’ deep
Primary Clarifier	2 (3)	1.8 mgd 55’ diameter

Secondary Clarifier	2 (3)	1.8 mgd 65' diameter
RAS/WAS Pump Station	1	
RAS/WAS Pumps	2 (3)	1650 gpm
Sludge Digester	2 (3)	75' diameter
Blower Building	1	1500 SF

[1]Future criteria at an ADMM of 5.4 mgd are presented in parentheses.

As proposed for the City of Ridgecrest, the CMAS in Phase 1 will consist of two aeration tanks, each with an AAD flow capacity of 1.8 mgd, two primary clarifiers, and two secondary clarifiers. A third aeration tank, primary clarifier, and secondary clarifier will be added for Phase 2. Effluent from the secondary clarifiers will be discharged directly to percolation evaporation ponds and/or storage.

The CMAS process has a relatively small volume of cell mass available to accommodate fluctuations in flow and waste strength, it is a less stable process than the Oxidation Ditch or Biolac processes. The shorter detention time also makes the process less forgiving or resistant to shock loadings. This process is easy to operate and is commonly used in California. This process utilizes separate clarifiers, which are generally more effective at separating clear liquid from the return solids than those processes that provide clarification internal to the aeration basin. It produces a stable sludge that requires less digestion than the SBR, but more so than the Biolac or oxidation ditch processes. The process produces secondary quality effluent, but a separate anoxic zone/tank is required for denitrification. Tertiary treatment is easily added to a CMAS process. There are many suppliers of CMAS equipment and thus the system can be competitively bid. The CMAS process requires less hydraulic retention time and less oxygen, and thus is often more cost-effective than ExAAS processes.

6.5.3 ExAAS- Oxidation Ditch

The Oxidation Ditch process was described above. For Ridgecrest, the appropriate Oxidation Ditch process contains the components summarized in **Table 6-5**.

Table 6-5. Oxidation Ditch Component Summary

Component	No. of Units[1]	Design Capacity per Unit

Aeration Basin (includes anoxic zone) and Equipment	2 (3)	1.8 mgd 149'x43'x15' deep HRT 24 hrs
Secondary Clarifier [2]	2 (3)	1.8 mgd 80 ft. diameter
RAS/WAS Pump Station	1	--
RAW/WAS Pumps	3 (4)	1400 gpm
Sludge Digester	2 (3)	80 ft. diameter
Equipment Building	1	1000 SF

[1]Future criteria at an ADMM of 5.4 mgd are presented in parentheses.

[2]Primary clarifiers not required to the ExAAS process

As proposed for the City of Ridgecrest, the Oxidation Ditch in Phase 1 will consist of two reactors, each with a capacity of 1.8 mgd, and two clarifiers. A third ditch and clarifier will be added for Phase 2. Effluent from the clarifiers will be discharged directly to percolation evaporation ponds and/or storage.

In contrast to the CMAS, the Oxidation Ditch process has a large volume of cell mass available to accommodate fluctuations in flow and waste strength, and is therefore a relatively stable process. The high solids content and long detention time make the process forgiving and resistant to shock loadings. It is easy to operate and is commonly used in California. This process utilizes separate clarifiers, which are generally more effective at separating clear liquid from the return solids than those processes that provide clarification internal to the aeration basin. It produces a very stable sludge that requires less digestion, depending on the method of disposal. The process has a good track record of producing low effluent BOD, TSS and Nitrogen. Tertiary treatment is easily added on to an oxidation ditch process. There are many suppliers of oxidation ditch equipment and thus the system can be competitively bid.

6.5.4 Biolac

The Biolac process was described above. For Ridgecrest, the appropriate Biolac process contains the following components summarized in **Table 6-6**.

Table 6-6. Biolac Component Summary

Component	No. of Units[1]	Design Capacity per Unit

Aeration Basin and Equipment	2 (3)	1.8 mgd 250'x170'x14' deep HRT 45 hrs
Secondary Clarifier [2]	2 (3)	1.8 mgd 85 ft. diameter
RAS/WAS Pump Station	1	--
RAW/WAS Pumps	2 (3)	1650 gpm
Sludge Digester	1 (2)	70 ft. diameter
Blower Building	1	1500 SF

[1]Future criteria at an ADMM of 5.4 mgd are presented in parentheses.

[2]Primary clarifiers not required to the ExAAS process

The Biolac system proposed for Ridgecrest will consist of two shotcrete lined basins with two external clarifiers. Two basins and clarifiers will be provided so that the plant can meet the necessary process component redundancy requirement for recycling wastewater. Process air will be provided by three positive displacement blowers. A blower building will be constructed adjacent to the basins. For the Phase 2 expansion, a third basin and clarifier will be added, with an additional blower.

The Biolac process combines the low cost of pond system and high efficiency of finebubble aeration, without the need for additional mixing. The Biolac process also provides long hydraulic and solids retention time, therefore resulting in a relatively stable process with lesser sludge generation. However, the long sludge age can cause problems in particle settleability, therefore producing a lower quality effluent. A key advantage of a Biolac or similar system is the ability to utilize pond type construction or to upgrade an existing pond. A disadvantage is that it requires a larger footprint than other processes. The system cost is lowest with an integral clarifier, however an independent clarifier is recommended for better process control and for the best quality effluent. The diffused air system is very efficient in oxygen transfer and thus utilizes less energy for aeration than an oxidation ditch.

6.6 Comparison of Secondary Wastewater Treatment Alternatives

A preliminary layout of each of the above treatment processes was performed as a tool in estimating the initial capital cost. In addition to the initial cost, the screening of alternatives considered the following factors:

1. Capital and operating cost estimates based on previous experience and vendor information.
2. Track record of process and number of installations in California.

3. Overall size and footprint required for the treatment facility.
4. Relative quality of secondary effluent produced.
5. Process reliability, stability, and ability to remove nitrogen.
6. Complexity, ease of operation and skill set needed for successful operation.
7. Scalability or modularity - the ability to easily expand the facility.
8. Operator familiarity or ability to find operators familiar with the treatment process.

The evaluation matrix used is shown in **Table 6-7**. The Oxidation Ditch ranked the highest followed by SBR. The factors considered in the evaluation parameters are described below.

Table 6-7. Evaluation Matrix for Secondary Biological Processes

Evaluation Parameter	Weight %	SBR	CMAS	Ox Ditch ExAAS	Biolac ExAAS
Lifecycle Cost	30%	9	8	10	7
Related Operation History	15%	7	9	9	9
Footprint	5%	9	7	5	2
Secondary Effluent Quality	10%	10	6	9	9
Process Stability	14%	7	7	9	10
Complexity of Process	10%	7	8	8	10
Modular Expandability	10%	10	8	8	3
Operator Familiarity	6%	8	10	9	9
	100%	8.4	7.9	8.9	7.7

Note: Scoring is from 1 to 10, with 10 being the highest ranked alternative.

6.6.1 Lifecycle Cost

The estimated capital and operating cost for the secondary treatment alternatives are shown in **Table 6-8** and **Table 6-9**. This cost table was developed for comparative purposes only and all costs of the wastewater treatment facility are not included. For example, costs for disposal are not included. The cost table should be considered relative costs. The Oxidation Ditch had the lowest life cycle cost as shown by the present worth analysis, followed by the SBR system, and with the Biolac and CMAS systems close behind.

6.6.2 Related Operation History

The City's ability to find, train and retain qualified operators for each process was considered. There are a number of CMAS and oxidation ditch plants in the surrounding area and in the state. Being a relatively new process, there are only about 25 installations of the Biolac process in the State of California. The SBR also has fewer installations.

6.6.3 Footprint

The smallest treatment process is the SBR followed by the CMAS. Both the Oxidation Ditch and the Biolac system are relatively large in physical footprint. A preliminary layout of the Biolac system showed that the Biolac system would not fit on the land available at the NAWS site with expansion to Phase 2 whereas the oxidation ditch would. If a proposed process would not fit on the available land at the NAWS site, it is a fatal flaw and that process cannot be further considered.

6.6.4 Secondary Effluent Quality

All the processes evaluated in this chapter are able to reliably deliver secondary effluent quality. The difference of the performance is related to the settleability of the particles in the subsequent clarifying process as well as nitrogen removal. The ExAAS processes (oxidation ditch and Biolac) are more challenging with respect to settleability of the particles because of the potential for over-oxidation and the formation of pin-point floc, while the SBR and CMAS processes produce a more settleable product. The SBR and Biolac processes both provide nitrification and denitrification within the typical process. The Oxidation Ditch and CMAS would also be designed to provide nitrogen removal by providing a slightly larger footprint in order to provide a dedicated anoxic zone.

6.6.5 Process Stability

The Oxidation Ditch and Biolac processes are considered to have good process stability due to the long sludge age and high inventory of biological solids. Both the Activated Sludge and the SBR processes are rated somewhat lower in stability. Due to their smaller volume, influx of contaminants will have a larger negative impact on the process stability.

6.6.6 Complexity of Process

Of the listed processes, the SBR is most mechanical, most automated, and perhaps the most difficult to troubleshoot. Because the SBR is a batch process, it requires more operator attention than the continuous flow processes. The remaining biological processes are all continuous flow and roughly similar in complexity, with the Biolac being the least complex. Continuous flow processes are considered to be less complex to operate than a batch flow process such as an SBR. Systems that do not require pumped flow for recirculation to an anoxic basin are also considered less complex. SBR, Biolac and oxidation ditch do not require pumped recirculation to an anoxic basin.

6.6.7 Expandability

All of the alternatives, except for Biolac, can be designed to allow easy expansion by an addition of a third train of treatment. The SBR and CMAS are more modular in design, however, and therefore received higher scores with respect to expandability. Both systems save structural costs by sharing walls. The Biolac process uses sloped basins that require more footprint.

6.6.8 Operator Familiarity

The ability to find and obtain operators familiar with the treatment process is an important consideration. The CMAS process is the most generic treatment alternative, with the other processes being some form of modification of that general process. The Oxidation Ditch and Biolac processes are commonly used in California, and are very similar processes to the CMAS in that they are continuous flow. There is not likely a significant pool of operators with experience with the SBR process. Additionally, the SBR process is a more complicated, batch system which would require more operator training.

Table 6-8. Capital and Operating Costs

Item No.	Item Description	Process SBR	CMAS	Oxidation Ditch	Biolac
1	Plant Land Use	$0	$0	$0	$0
2	Mobilization, bonds, insurance	$1,240,000	$1,240,000	$1,240,000	$1,240,000
3	Headworks/Primary Treatment	$2,022,500	$2,890,842	$2,022,500	$2,022,500
4	Aeration Basin	$2,054,000	$1,591,000	$1,813,000	$1,622,000
5	Aeration Equipment	$1,931,000	$1,074,000	$1,091,000	$1,754,000
6	Clarifier Structure	$0	$687,000	$921,000	$921,000
7	Clarifier Equipment	$0	$935,000	$935,000	$935,000
8	Flow Distribution Boxes/Valves	$0	$100,000	$100,000	$100,000
9	RAS/WAS Pump Station	$0	$350,000	$350,000	$350,000
10	Site and Yard Piping	$1,300,000	$1,500,000	$1,500,000	$1,800,000
11	Buildings	$1,250,000	$1,250,000	$1,000,000	$1,250,000
12	Sludge Digester	$2,914,000	$2,914,000	$2,914,000	$2,914,000
13	Sludge Dewatering Equipment	$1,325,000	$1,325,000	$1,325,000	$1,325,000
14	Effluent Pump Station	$200,000	$200,000	$200,000	$200,000
15	Effluent Distribution Pipeline	$18,000	$18,000	$18,000	$18,000

16	Site work/Fencing		$2,145,000	$2,212,000	$2,221,000	$3,287,000
17	Electrical and Controls		$3,000,000	$3,000,000	$3,000,000	$3,000,000
18	Emergency Power		$300,000	$300,000	$300,000	$300,000
19	Abandon/Demo Existing WWTP		$250,000	$250,000	$250,000	$250,000
Subtotal for Secondary Effluent			19,950,000	$21,837,000	$21,201,000	$23,289,000
	Engr/CM/Environment/Admin	30%	$5,985,000	$6,551,000	$6,360,000	$6,987,000
	Financing Cost	10%	$1,995,000	$2,184,000	$2,120,000	$2,329,000
	Contingency	20%	$3,990,000	$4,367,000	$4,240,000	$4,658,000
Total For Secondary Treatment			31,920,000	$34,939,000	$33,921,000	$37,263,000
	Annual O&M Cost		$1,014,600	$906,600	$888,100	$841,800
	O&M Present Worth, 2.5%	20 yrs	$15,817,000	$14,134,000	$13,845,000	$13,123,000
	Total Present Worth		$47,737,000	$49,073,000	$47,766,000	$50,386,000

Table 6-9. Annual Operations and Maintenance Costs

Cost Component		SBR	CMAS	Oxidation Ditch	Biolac
Power Cost	0.12/kW-hr	$280,000	$251,000	$376,000	$282,000
Equipment Maintenance		$60,600	$69,400	$25,300	$66,800
Biosolids Disposal	$300/Truck	$39,500	$38,100	$30,900	$34,000
Equipment Replacement	3%/yr	$39,500	$38,100	$ 30,900	$ 34,000
Labor	$85,000/op	$595,000	$510,000	$425,000	$425,000
Cost-Secondary		$1,014,600	$906,600	$888,100	$841,800
Operators for Secondary	# Operators	7	6	5	5

6.6.9 Recommended Secondary Wastewater Treatment. Alternative

The present worth analysis shows that the Oxidation Ditch process and SBR have the lowest present worth cost and the highest ranking in the secondary process evaluation matrix. Although Biolac has slightly lower operating costs, it was eliminated because it will not fit on the available NAWS site. Because the SBR is not a continuous flow process and it has higher on-going operations cost, the oxidation distch alternative is preferred even though capital costs are lower. Based on the comparison evaluation, the Oxidation Ditch process is recommended as the preferred secondary treatment process.

6.7 Candidate Tertiary Treatment Alternatives

This section reviews candidate processes for production of tertiary disinfected water for unrestricted reuse. The tertiary treatment processes would be located at the site of the secondary treatment facility, providing additional treatment to the secondary effluent. A portion of the secondary treated effluent, instead of being discharged to evaporation/percolation ponds or used for alfalfa irrigation, would go through the tertiary filters and disinfection prior to being recycled. Additionally the tertiary treated water could be applied to the China Lake Golf Course, providing a higher quality irrigation water without the operational challenges currently experienced and the current restrictions on application of chlorinated secondary effluent.

Three (3) tertiary filter systems were initially evaluated for the City of Ridgecrest WWTP upgrade and expansion. All are California Title 22 approved filter systems suitable for unrestricted reuse. The three alternatives include cloth media filters, continuous backwash sand filters, and Fuzzy Filter compressible media filter. Based on initial evaluation, continuous backwash sand filter was not considered to be a viable tertiary treatment alternative because the sand filter has significantly higher operation and maintenance costs than the other two filter alternatives, mainly due to higher power costs. The sand filter also requires a much larger footprint than the other filters considered. Aqua-Disk cloth media filter and Fuzzy Filter are proprietary products. Other cloth filters are now available. MBR was not considered as a primary alternative because this technology combines both secondary and tertiary treatment in one process train. Only a portion of the City's wastewater flow will require tertiary treatment (for unrestricted use) and therefore the MBR technology would not be necessary for the majority of the City's wastewater flows.

To meet Title 22 criteria for disinfected tertiary recycled water, all proposed filter systems will have coagulant feed prior to filtration. The coagulant feed systems are assumed to be the same for both alternatives. Both filter systems will feed directly to the selected disinfection process.

6.8 Viable Tertiary Treatment Processes

Tertiary treatment of effluent consists of two discrete processes: filtration and disinfection. Each process can be provided using several different types of equipment. Two common methods for each are described below.

6.8.1 Cloth Filter

The cloth media filter is a gravity fed filter capable of on-line automatic backwashing without interrupting the filtration. The standard unit can hold 12 vertically oriented disks with average hydraulic capacity up to 3 mgd for one unit. Disks can be added for future expanded flow. The unit is fully automated with a PLC control system.

Several different vendors make cloth media disk filters; the most common in California is made by Aqua-Disk. The Aqua-Disk cloth media filter has the highest capital equipment cost but has more installations than Fuzzy Filter. Operations and maintenance costs are similar to the Fuzzy Filter.

The Aqua-Disk cloth filters are modular and take up a very small footprint, making the filters easily expandable and scalable. The filters can easily be added to the secondary process as needed.

6.8.2 Fuzzy Filter

The Schreiber Fuzzy Filter is a proprietary, upflow, compressible spherical media filter capable of high hydraulic and solids loading rates. The low density and high porosity of the media can capture and contain more solids than conventional sand media. The total porosity of the filter bed can be altered by mechanically compressing the media.

Fuzzy Filter uses air scouring and effluent water to clean the media without the need for stopping influent feed. The high filtration rate of 30 gpm/sf makes the filter the most compact in terms of footprint for both alternatives.

The Fuzzy Filter has the lowest cost for both capital and operations and maintenance.

Similar to the Aqua-Disk, the Fuzzy Filter is modular, easily expanded and scalable for plant expansion.

6.9 Comparison of Tertiary Treatment Alternatives

A detailed evaluation of the tertiary filter alternatives listed above include: a preliminary capital and operation cost estimate; weighing of the advantages and disadvantages of each alternative; and scoring and ranking of the processes accordingly. The evaluation matrix used is shown in **Table 6-10** and criteria are discussed below.

Table 6-10. Evaluation Matrix for Tertiary Filtration

Item Description	Weight %	Cloth Filter	Fuzzy Filter
Lifecycle Cost	30%	10	9
Related Operation History	15%	10	9
Footprint	5%	9	10
Effluent Quality	15%	9	9
Process Stability	20%	10	10
Complexity of process	8%	10	9
Expandability	7%	10	10
	100%	9.80	9.32

Note: Scoring is from 1 to 10, with 10 to be the best alternative.

6.9.1 Lifecycle Cost

Operational cost for any type of filter consists of media replacement and energy costs due to pressure loss through the unit. For either option considered, media replacement costs are negligible. The Cloth Filter has a slightly lower capital cost, yielding the lowest 20 year lifecycle cost.

This installed capacity will be dependent on the acreage of landscape irrigation or other demands that can be incorporated into the recycled water program. The costs were developed assuming 1.8 MGD of capacity in the tertiary treatment process, which includes 0.9 MGD of demand for City landscaping during peak summer use and an additional 0.9 MGD for the China Lake Golf Course during peak summer use. Capital and operations costs for the tertiary treatment alternatives are summarized in **Table 611**.

6.9.2 Related Operation History

Within the San Joaquin Valley and Mojave Desert areas, there is very limited experience with tertiary filters because most wastewater is only treated to a secondary level. Consequently, additional operational history experience should be researched during the design phase. The Fuzzy Filter system has fewer installations than the cloth filter, but there is an installation nearby in Malaga (San Joaquin Valley). The Fuzzy Filter at the Malaga wastewater treatment plant has been in operation for about nine (9) years. The cloth filter is much more widely used and has a good track record of performance.

6.9.3 Footprint

Fuzzy Filter has the smallest footprint, while Aqua-Disk is slightly larger. However, the footprint for the filters is small compared to the biological process, and for the same reason as with the biological processes, footprint is given a weight of only 5 percent.

6.9.4 Process Reliability / Effluent Quality

Both technologies have more than 10 years of operation history and are approved by the Division of Drinking Water for Title 22 application.

6.9.5 Process Stability

Given the proven track record for both systems, it is apparent that both systems have control and alarm systems sufficient to provide long term, reliable service.

6.9.6 Complexity of Process

The Fuzzy Filter has one gear motor to compress the media and two blowers for backwash. The Aqua-Disk filter has a gear motor to rotate the disks during backwash, and one small backwash pump, and is mechanically less complicated.

6.9.7 Expandability

Expansion of cloth filters for future flow is achieved by adding four more disks to each filter; no additional filter tanks would need to be purchased. Fuzzy Filter is undergoing a test for the Division of Drinking Water to increase the filtration rate from 30 gpm/sf to 42 gpm/sf. Both the Aqua-Disk and Fuzzy Filters are considered equally expandable.

Table 6-11. Capital and Operating Costs - Tertiary Filtration, 1.6 MGD

Tertiary Treatment (1.6 MGD)		AquaDisk	Fuzzy Filter
Tertiary Filter		1,050,000	1,194,000
Engr/CM/Environment/Admin	32%	336,000	382,000
Contingency	20%	210,000	239,000
Total for Tertiary Process		1,596,000	1,814,000
Annual O&M Cost		144,000	143,000
O&M Present Worth, 2.5%	20 yrs	2,249,000	2,230,000
Total Present Worth		3,844,900	4,047,455

6.9.8 Recommended Tertiary Treatment Alternative

Based on this evaluation, the Cloth Filter is indicated to be the slightly preferred alternative for Ridgecrest. Tertiary filtration processes and experience are rapidly evolving. It is recommended that, should tertiary treatment be selected, additional research and evaluation of the candidate processes be completed during the design phase.

6.10 Disinfection Alternatives Considered

A disinfection process will immediately follow filtration. Two candidate methods of disinfection will be discussed in this section, chlorine disinfection and Ultraviolet (UV) disinfection. An evaluation matrix is shown in **Table 6-12** for the same criteria discussed in Section 6.9. Although Ozone generation and use can provide adequate disinfection, this alternative has not been considered due to its very high initial and operational costs.

6.10.1 Chlorine Disinfection

Chlorine disinfection consists of the injection of liquid sodium hypochlorite into the effluent stream. After injection, the effluent flows through a chlorine contact basin to provide contact time. Chlorine offers the advantage of continued residual disinfection after the initial injection when excess chlorine is injected above the immediate chorine demand. The addition of chlorine will add salt to the effluent and increase the electrical conductivity (EC) of the effluent. The salt addition from chlorine disinfection will have minimal impact on the total EC. The potential for impact to groundwater from disinfection byproducts (DBPs) is also a possible concern with chlorine disinfection. However, with recycled water irrigation at agronomic rates, very little water will reach the groundwater table. Chlorine disinfection is a well proven, well understood and reliable form of disinfection. It is less costly to install, and O&M costs would also be less costly than the energy intensive UV system.

6.10.2 UV Disinfection

UV disinfection is a relatively new process that is rapidly gaining acceptance, particularly when there is a discharge to a water body or when the impact of disinfection by-products is of concern. UV disinfection utilizes ultraviolet (UV) light to inactivate pathogenic organisms. UV disinfection works by passing water over (or around) submerged UV lights. Unlike chlorine disinfection, it does not require additional contact time. It works best with filtered effluent because its effectiveness is dependent on light transmittance through very clear water. UV disinfection has the advantage of not adding chemicals to the water and thus does not produce DBPs nor added salt compounds in the effluent. It is especially advantageous for discharge to surface water canals or streams because it does not require a second de-chlorination step to protect aquatic life from the toxicity of chlorine compounds. However, UV disinfection is more costly and energy intensive than chlorine disinfection. Another disadvantage is that there is no residual disinfecting power.

Therefore, depending on the type of re-use, chlorine may need to be added following UV to obtain residual disinfection.

Table 6-12. Evaluation Matrix for Disinfection

Item Description	Weight %	Chlorine Disinfection	UV Disinfection
Lifecycle Cost	30%	10	6
Related Operation History	15%	10	10
Footprint	5%	5	10
Effluent Quality	15%	8	10
Process Stability	20%	10	10
Complexity of process	8%	8	8
Expandability	7%	8	10
	100%	9.15	8.64

Note: Scoring is from 1 to 10, with 10 to be the best alternative.

Capital and operations costs for the disinfection alternatives are summarized in **Table 6-13**.

Table 6-13. Capital and Operating Costs - Disinfection, 1.6 MGD

Disinfection (1.6 MGD)		Chlorine	UV
Disinfection		$629,000	$1,225,000
Engr/CM/Environment/Admin	31%	195,000	380,000
Contingency	20%	126,000	245,000
Total for Tertiary Process		950,000	1,850,000
Annual O&M Cost		23,000	59,000
O&M Present Worth, 2.5%	20 yrs	351,000	916,000

Total Present Worth	1,301,000	2,766,000

6.10.3 Recommended Alternative

Based on cost considerations alone, chlorine disinfection is recommended. Chlorine disinfection has both lower capital and operating costs.

6.11 Staffing Requirements

6.11.1 Selected Secondary Treatment Process

The Oxidation Ditch process is highly mechanized processes, with capacity for a great deal of automated control. Operator training requirements, established by the RWQCB in Title 23, will require an operator certified as a treatment Grade IV to be in charge of the treatment process.

In addition to the need for a highly trained certified operator, the operation of the mechanical equipment may require staffing of the facility for more than one shift per day while operating at a design flow of 3.6 mgd. To allow for staffing of the facility during weekend, and alternating shifts, a total of 6 licensed operators will be needed for the facility, with additional maintenance and support staff. A total staff of 8 is recommended.

This operator requirement is a minimum, and is separate from the City's need for staff to perform repairs and oversee wastewater operations and repairs outside the plant boundary- including sewer maintenance workers.

6.11.2 Selected Tertiary Treatment Process

The addition of a tertiary treatment process to the new facility will increase the need for operator qualifications. The RWQCB will require an operator certified as a treatment Grade IV to oversee the operations of the facility when tertiary filtration facilities (either type of filter) and disinfection facilities are added. The addition of tertiary facilities will not change the overall staffing level needed for the facility.

7 EFFLUENT DISPOSAL

7.1 Introduction

This section presents the findings of alternative methods of effluent disposal for the City's wastewater treatment facility. As discussed in Section 4, effluent disposal currently occurs by three mechanisms: evaporation, percolation, and evapotranspiration. At the City site, percolation and evaporation occur through the disposal ponds. Additionally, effluent is recycled to irrigate alfalfa (evapo-transpiration). At the NAWS disposal pond site, facultative ponds 1 through 7 are lined with bentonite clay, and provide minimal percolation, but will provide evaporation. Ponds 9 and 10 provide for both percolation and evaporation. Currently, ponds 8 and 11 cannot be put in service due to excess seepage from the ponds. Figure 4-13 shows the configuration of the existing ponds. Effluent pumped to the City site or used for golf course irrigation is taken from Pond 3.

Currently Ponds 1-7 act as facultative lagoons providing secondary treatment of the City's wastewater. When the plant expansion and upgrade is completed, treatment will occur in the selected secondary process and the ponds will serve as evaporation ponds for wastewater disposal and storage for golf course irrigation. At that time Ponds 8 and 11 should be available for wastewater disposal and storage.

Ponds 9 and 10 provide disposal by evaporation and percolation. These ponds are required to be maintained wet at all times through an agreement between the City and NAWS in order to provide seepage flow to Lark Seep, a nearby habitat of an endangered species of fish, the Mohave Tui chub. The China Lake Golf Course also utilizes recycled effluent for irrigation. The City is under agreement with NAWS to provide 750 AF/yr to the US Navy for golf course irrigation or other uses. However, flow data provided by the City shows that the golf course only uses approximately 500 AF/yr.

The total existing capacity from all of the disposal methods currently in service is estimated to be about 2.54 mgd (AAD flow), based on a 25-year storm event, and assuming all 11 acres of ponds at the City site are utilized. The actual capacity may be greater because of variability in percolation rates at different locations. This, however, is well below the permitted maximum day flow capacity of 3.6 mgd, and insufficient for any increase in flows. If new treatment technology is installed providing secondary treated effluent to the ponds, Ponds 8 and 11 could be used for effluent disposal. The total existing disposal capacity including ponds 8 and 11 (assuming they would provide evaporation and percolation) would be approximately 2.82 mgd (AAD flow).

7.2 Percolation Rates

A single percolation test was performed on one of the existing City site percolation/ evaporation ponds over a two week period during December 2010. The pond was partially filled at the beginning of the percolation test. After filling, the initial level was measured, and level was subsequently measured daily during the two-week testing period. The total evaporation for the same time period subtracted from the total decline in pond level yielded the total percolation. The percolation test showed about 0.1 inch per day of percolation. Percolation rates for ponds 9 and 10 are unknown and have not been directly measured.

Based on the measured percolation rate of 0.1 inches per day, with the existing disposal areas discussed in Section 4, the effluent disposal capacity would only be 2.3 mgd. By observation and past history, the City is able to dispose of at least 2.6 mgd without problem. Using a water balance calculation with flow of 2.6 mgd, normal year rainfall and evaporation, the indicated percolation rate is much higher, approximately 0.23 inches per day. This calculation assumed that there is minimal surplus capacity at 2.6 mgd with the following conditions; ponds 8 and 11 at the NAWS site are not in use and approximately 7.2 acres of the approximate 10.5 acres of disposal ponds on the City site are currently being used. Therefore the tested percolation rate of 0.1 inches per day has been determined to be not representative of the overall average percolation rate. Water balance calculations are included in Appendix B, Water Balance Calculations. A percolation rate of 0.23 inches per day will be assumed in determining future disposal land area requirements.

A *Draft Preliminary Soil Investigation Report, Ridgecrest Wastewater Treatment Plant, Ridgecrest, California*, prepared by BSK & Associates in January 2011, indicated infiltration rates (referred to herein as percolation rates) throughout the City site ranging from 1.1 to 2.2 inches per day. These percolation rates are much greater than those calculated or achieved during the recent percolation test, and are not considered to be realistic long term achievable values of actual percolation from the disposal ponds.

A *Preliminary Slow Rate Infiltration Study, Inactive Sewage Treatment Facility, Ridgecrest, California*, prepared by Converse Consultants Southwest, Inc. in April 1991, indicated that the design percolation rate based on hydraulic conductivity for the majority of the City site should fall within the range of 133 to 333 feet per year (4.4-11.0 inches/day), and their recommended design percolation rate for the northern portion of the City site should fall within the range of 83 to 200 feet per year (2.7-6.6 inches/day). These are much higher rates than those calculated or achieved through the percolation test; they are also higher than those estimated in the soil study prepared by BSK. *Preliminary Slow Rate Infiltration Study* also indicates that sodic soils are prevalent at this site. Sodic soils are characterized by a high sodium adsorption ratio (SAR). With sodic soils, high sodium levels may, over time, cause low soil permeability and soil “sealing” against infiltration/percolation. However, Converse Consultants suggest that sodic conditions may be corrected with the addition of soluble calcium (gypsum) to the soils and subsequent leaching to remove displaced sodium. The calculated percolation rate discussed above and used in this report may therefore be improved with the addition of soil amendments, and required disposal pond areas would therefore be reduced. The relative benefits of adding gypsum versus costs have not been evaluated in this report. Addition of gypsum will therefore not be considered further in the disposal analysis.

The percolation test, calculation, and studies above indicate a wide variance in potential percolation rates for the area. Based on knowledge of the disposal facilities, usage, and sufficiency at current flows, the calculated percolation rate of 0.23 inches per day is determined to be the most appropriate value, and will therefore be used in this report. Percolation rates should, however, continue to be evaluated as more disposal ponds are constructed in order to validate or dispute the percolation rate used herein. It is recommended that additional analysis of percolation rates be done in conjunction with the final design phase.

7.3 Future Disposal Capacity

Future wastewater effluent in Ridgecrest will be disposed in much the same manner as at present- either by evaporation, percolation, or storage and recycling. Current disposal facilities were described in Section 4. The existing disposal facilities have sufficient capacity to dispose of current flows, but essentially no surplus capacity is available. If secondary treatment facilities are added and the ponds are used only for storage, percolation and evaporation of previously treated secondary effluent, Ponds 8 and 11 may be put back into service, increasing the disposal capacity to 2.82 mgd (AAD flow).

While the design capacity of the proposed WWTP secondary treatment facility is based on average day maximum month (ADMM) flows, the disposal facilities will be designed based on annual average daily (AAD) flows. At the planning horizon, it will therefore be necessary to dispose of 5.4 mgd (AAD) of effluent (based on an ADMM design flow of 5.9 mgd). Phase 1 will require capacity for disposal/recycling of 3.6 mgd AAD. New disposal facilities will need to be constructed incrementally, as needed to accommodate effluent flows. The following analysis presents an evaluation of methods, locations, and capacities to accommodate future flows.

7.4 Effluent Disposal Options

The current disposal capacity is approximately 2.54 mgd, based on a 25-year rainfall recurrence interval. The permitted capacity of the existing WWTP is 3.6 mgd maximum day flow. Ultimately, the disposal capacity should match the permitted capacity of the WWTP, but even more important, additional disposal capacity is needed to ensure capacity remains sufficient during wet seasons (greater than a 25-year rainfall year) and to accommodate any near-term population growth.

Assuming the present disposal facilities will be utilized to the present capacity of 2.88 mgd, (with Pond 8 and 11) additional disposal capacity of 0.7 mgd must be developed to dispose of the added effluent in the near term, to the Phase 1 WWTP capacity of 3.6 mgd. This disposal capacity will be sufficient until about year 2029 given estimated growth rates in the community. Available methods for disposal of this added flow are described in the following sections.

7.4.1 Discharge to Surface Water – (stream or lake)

This option has been removed from consideration for several reasons:

- Suitable water bodies do not exist near Ridgecrest.

- RWQCB policies strongly encourage the reuse of water.
- Permitting and monitoring of surface water discharges is much more complicated and expensive than alternative choices.
- Environmental constraints on this type of discharge are formidable.
- NPDES discharge is highest risk to City and exposes City to possible minimum mandatory penalties for violations.

7.4.2 Percolation and Evaporation Ponds

Soils in the area percolate about 0.23 inches of water daily as calculated based on the current flows. Based on rainfall of 0.8 feet per year, a percolation rate of 0.23 inches per day (7.0 feet per year), and an evaporation rate of approximately 9.0 feet per year, approximately 16 feet of effluent per year per acre can be disposed through use of percolation and evaporation ponds. Because of the high evaporation rate, about 60 percent of the water is lost to the atmosphere and does not benefit the overall region's water balance. Therefore, if other more direct methods of re-use are available economically, the reliance upon evaporation should be discouraged.

Depending on the nature of soils and underlying geology, construction of additional evaporation / percolation ponds for effluent disposal is often an economical choice, and offers several other benefits to the City:

- Aquifer recharge provides a long term benefit to the entire community.
- Ease of permitting, testing, and reporting.
- Ease of operation.

Percolation ponds located at the City site would be preferred over ponds located at the NAWS site because the underlying groundwater is of better quality.

7.4.3 Storage and Irrigation

The City's capacity to irrigate will vary seasonally. A portion of effluent destined for irrigation must be stored during winter months for application during the irrigation season. Irrigation of a forage crop such as alfalfa may utilize approximately 7.4 feet per year of recycled effluent.

It is also possible that the City may elect to treat effluent to tertiary standards in the future and use recycled effluent on landscaped areas within the City. Irrigation of landscaped areas may utilize approximately 7.2 feet of recycled effluent per year. Irrigation of landscaping and crop irrigation is consistent with State policy that encourages beneficial re-use of water,

An issue of concern is the current China Lake Golf Course irrigation practices. The irrigation water is currently taken from the facultative pond 3.. Originally, the water was to be filtered then disinfected prior to use on the golf course. Filtration has been discontinued due to plugging of the filters by algae from the ponds. While filtration is not technically required for a restricted use golf course (Title 22 requirements), disinfection must maintain coliform levels below 23 MPN. Effluent provided by the new secondary treatment process will likely be better quality than the current treatment processes, but some algae growth may occur in the storage ponds. It is assumed that the golf course will continue to be responsible for disinfecting the effluent as it is applied to the golf course. The Navy and the City may want to consider an alternate arrangement where the City provides tertiary recycled wastewater to the Navy at an agreed price.

Four broad issues for irrigation disposal of effluent must be considered: irrigated acreage, effluent storage, water quality constraints, and suitability of water quality.

1. Irrigated acreage is assumed to vary with annual flow. Adding more irrigated acres as flows increase is relatively simple, and incurs costs only as needed.
2. Storage ponds might be unlined, serving also as percolation/evaporation ponds or lined so that a larger fraction of the water can be effectively irrigated. Liners add capital cost of the storage pond. In addition, liners increase the pond storage volume required and reduce the percolation fraction. For these reasons, the use of liners will increase the need for disposal acreage.
3. Water quality required for irrigation is subject to regulation by the Division of Drinking Water, as detailed under Title 22 of the State Code. Different crops (including turf) and their corresponding irrigation water quality requirements are discussed in **Section 5.6.**
4. Suitability of water quality for irrigation of specific crops based on chlorides, sodium, TDS, nitrates, boron, etc., must be considered. Some crops are much more tolerant than others to high levels of salts and other nutrients, and a nutrient mass balance should be conducted if irrigation is selected as a disposal method.

To identify a desirable effluent disposal system, several alternatives have been developed that combine elements of evaporation, percolation and irrigation in different proportions. A preliminary estimate of required acreage was then developed for each.

7.5 Effluent Disposal Alternatives Considered

To help quantify the required disposal alternatives, three different methods of disposal were developed for conceptual layout and costing:

1. Added percolation / evaporation ponds (un-disinfected, secondary treatment).
2. Irrigation of alfalfa (un-disinfected, secondary treatment).
3. Irrigation of public landscape areas (disinfected, tertiary treatment).

Each of the alternatives was defined to meet the following criteria:

- Capacity sufficient to dispose of an annual average daily flow of 3.6 mgd and 5.4 mgd (Phase 2) during the 25-year rainfall recurrence.
- A percolation rate of 0.23 inch per day for unlined ponds, and zero for lined ponds.
- Irrigation of the China Lake Golf Course at an annual average rate of 497 acre feet per year.

- Irrigation of landscape (parks and schools) at an annual average rate of 86 inches per year. This will require Title 22 tertiary disinfected recycled water for unrestricted access.
- Irrigation of alfalfa at an annual average rate of 89 inches per year. This will require un-disinfected secondary treatment.
- Storage of wintertime flows for later irrigation during the growing season.
- The City will continue to provide un-disinfected, secondary effluent to the China Lake Golf Course. NAWS will be responsible for disinfection prior to application to the golf course.
- The City site is the preferred location for new percolation/evaporation ponds.
- Existing landscape areas would be identified and available for recycling of tertiary effluent, as well as approximately 70 acres of golf course presently irrigated at the NAWS.
- Although Ponds 1-7, 8 and 11 are currently non-percolation ponds (per the existing WDR's), converting these ponds from facultative (treatment) lagoons to secondary effluent storage ponds may allow these ponds to be converted to evaporation/percolation ponds. Removal of the existing lining and accumulated sludge would be required, but this change could be more economical than acquiring land and constructing new ponds.

7.6 Evaluation of Alternative Disposal Methods

7.6.1 Water Balances

Water balances for various disposal scenarios were developed. See **Appendix B** for the water balance calculations. Three sets of scenarios were developed as described below. All flows presented below are average annual daily flows (AAD). For all scenarios it is assumed that effluent being delivered to the China Lake Golf Course is maintained at the current level.

7.6.2 No Upgrades or Expansion of Treatment Capacity

Two evaluations were performed under this scenario. The alternatives under this set of evaluations were:

- No change in treatment processes or capacity. Ponds 8 & 11 remain out of service Disposal capacity is 2.54 mgd
- No change in pond size and volume, or golf course and alfalfa disposal capacities. Ponds 8 & 11 assumed to be lined and in service. Disposal capacity is 2.82 mgd.

7.6.3 Treatment Capacity Increased to 3.6 MGD (Phase 1)

Three evaluations were performed under this scenario. These assume no percolation from Ponds 1-7, 8 and 11, and that Ponds 9 and 10 are in service and percolate. The alternatives under this set of evaluations were:

- Add alfalfa irrigation area. Pond capacities remain the same with Ponds 9 & 10 assumed to percolate and evaporate. This requires the addition of 145 acres of alfalfa and no additional evaporation/percolation ponds.
- Add additional evaporation/percolation ponds with Ponds 9 & 10 assumed to percolate and evaporate. This requires the addition of 56 wet acres of new evaporation/percolation ponds.
- Add landscape irrigation. Pond capacities remain the same with Ponds 9 & 10 assumed to percolate and evaporate. This requires 129 acres of landscape irrigation and no acres of additional evaporation/percolation ponds.

7.6.4 Treatment Capacity Increased to 5.4 MGD (Phase 2)

Three evaluations were performed under this scenario. These assume no percolation from Ponds 1-7, 8 and 11, and that Ponds 9 and 10 are in service and percolate. The alternatives under this set of evaluations were:

- Add alfalfa irrigation area. Pond capacities remain the same with Ponds 9 & 10 assumed to percolate and evaporate. This requires the addition of 367 acres of alfalfa and 19 wet acres of additional storage ponds.
- Add additional evaporation/percolation ponds with Ponds 9 & 10 assumed to percolate and evaporate. This requires the addition of 187 wet acres of evaporation/percolation ponds.
- Add landscape irrigation. Pond capacities remain the same with Ponds 9 & 10 assumed to percolate and evaporate. This requires 342 acres of landscape irrigation and 64 acres of additional evaporation/percolation ponds.

7.6.5 Treatment Capacity Increased to 3.6 MGD, All Ponds - 0.23 Inches/Day Percolation

This evaluation assumes that all existing ponds have a percolation rate of 0.23 inches/day. By modifying all ponds to percolate, no other disposal improvements are required to provide disposal capacity for 3.6 mgd. This would require the removal of both the accumulated sludge in the ponds and the existing bentonite liner in the ponds. There is, however, no guarantee that removing the bentonite clay layer and accumulated biosolids, that the desired 0.23 in/day percolation rate can be achieved.

7.6.6 Treatment Capacity Increased to 5.4 MGD, All Ponds - 0.23 Inches/Day Percolation

Three evaluations were performed under this scenario. These assume 0.23 inches/day of percolation from Ponds 1-7, 9 and 10, as well as Ponds 8 and 11.The alternatives under this set of evaluations were:

- Add alfalfa irrigation area. This requires the addition of 271 acres of alfalfa and 61 wet acres of additional evaporation/percolation ponds.
- Add additional evaporation/percolation ponds. This requires the addition of 109 wet acres of evaporation/percolation ponds.

- Add landscape irrigation. This requires 276 acres of landscape irrigation and 18 wet acres of additional evaporation/percolation ponds.

7.6.7 Summary

Table 7-1 presents a summary of the flows, percolation and acreages developed for the various options described above.

Table 7-1. Water Balance Summary

With Only Ponds 8, 11 and New Ponds Percolation @ 0.23 in/day

Disposal Method	Phase I – 3.6 MGD	Phase 2 – 5.4 MGD
Alfalfa Irrigation	145 acres	367 acres plus 19 acres new ponds
Evaporation/Percolation Ponds	56 wet acres	187 wet acres
Landscape Irrigation	129 acres	342 acres plus 64 acres of new ponds

With All Ponds Percolation @ 0.23 in/day

Disposal Method	Phase I – 3.6 MGD	Phase 2 – 5.4 MGD
Alfalfa Irrigation	0	271 acres plus 61 acres new ponds
Evaporation/Percolation Ponds	0	109 wet acres
Landscape Irrigation	0	276 acres plus 18 acres new ponds

In order to expand its effluent disposal capacity to 3.6 mgd (ADMM), the City has several options. The most cost effective option is to add 56 acres of new evaporation/percolation ponds on the City owned land (City site).

Due to the ongoing drought and concerns about local groundwater, the City has expressed interest in disposal options that would either recharge local groundwater supplies or replace existing demands on groundwater supplies. Replacing existing irrigation demands that use groundwater supplies with effluent from the WWTP would have the greatest benefit. As there is little existing irrigated farmland near the City or the NAWS sites, irrigation of parks and schools grounds is the other available alternative. Irrigation of these grounds would require the addition of tertiary treatment of the wastewater effluent and disinfection for Title 22 unrestricted reuse, equalization storage and a dedicated purple pipe distribution system from the location of tertiary treatment to the application areas. Approximately 77 gross acres of potential irrigation areas were identified within the City as practical and are shown on **Figure 7-1**.

It appears that only 40 to 45 acres of these sites are currently irrigated compared with a need for 129 acres to provide the additional disposal for 3.6 mgd WWTP capacity. Additional park and recreation areas exist on the NAWS base, but agreements with NAWS to use these areas for effluent disposal would be required. Assuming approximately 129 irrigated acres could be identified, this alternative would require the addition of approximately 0.7 mgd of tertiary treatment capacity at the wastewater plant and a recycled water pump station and pipeline to the disposal areas. Should the China Lake Golf Course add approximately 0.9 MGD to the tertiary water demand, the total treatment capacity would be approximately 1.6 mgd. Siting the WWTP at the City site would require a dedicated pipeline to transport the tertiary recycled water to the China Lake Golf Course.

The capital costs associated with the three options (alfalfa, evaporation/percolation ponds and landscape irrigation) are summarized in **Table 7.2**.

It is assumed that the value of the alfalfa product offsets the operating cost of farming. The cost associated with landscape irrigation does not include the additional tertiary treatment process and disinfection discussed in Section 6.9. The landscape irrigation cost assumes that the WWTP will be located at the NAWS site; otherwise the additional cost of a recycled water pipeline to the China Lake Golf Course would add over $1,000,000 to the capital cost.

Table 7-2. Cost Estimate for Proposed Effluent Disposal Options

Item		Alfalfa Irrigation	Percolation Ponds	Landscape Irrigation
Disposal		$592,000	$2,540,000	$2,600,000
Engr/CM/Environment/Admin	32%	$189,000	$812,000	$832,000
Contingency	20%	$118,000	$508,000	$520,000
Total for Disposal		$899,000	$3,860,000	$3,952,000
Annual O&M Cost		$0	$16,000	$60,000
O&M Present Worth, 2.5%	20 yrs	$0	$249,000	$935,000
Total Present Worth		$899,000	$4,109,000	$4,887,000

7.7 Recommended Disposal Option

Based on the current set of assumptions, there is not enough disposal capacity to accommodate the current permitted flow, and current flows are approaching the capacity of the current disposal area. The City currently owns land near the City site, including approximately 80 useable acres within Kern County and 47 additional acres within San Bernardino County (**Figure 7-2**). In addition, there are parks and schools throughout the City and the NAWS that could potentially utilize recycled water..

It is recommended that all existing disposal methods and locations continue to be utilized, including maintaining and upgrading existing disposal ponds 8 and 11 to put them back into service as evaporation/percolation ponds. It is recommended that the additional disposal capacity required be satisfied with percolation and evaporation ponds. The long term sustainable rate of percolation from the various sites will be a key factor in determining the acreage needed for this disposal method. The calculated current percolation rate discussed above (0.23 inches per day) is to date the best prediction of the actual long term percolation rate. As such, 56 acres of additional disposal ponds will be required to meet a capacity of 4.0 mgd ADMM (3.6 mgd on an AAD basis). The required 56 acres of disposal ponds is based on a “wet area”. The total gross land requirement is approximately 95 acres, which includes berms, setbacks and roads.

Future disposal capacity beyond 3.6 mgd (AAD)) may include construction of more ponds, or irrigation of public landscape areas should the City elect to treat a portion of the flow to tertiary treatment standards. Demand for recycled water for irrigation purposes is highly seasonal with the majority of the use in the summer months. Storage and percolation ponds will still be necessary for the storage and percolation of effluent when the irrigation demand is low.

It is recommended that the City continue to pursue recycled water for landscape irrigation as a future priority method of disposal. The City should begin working with school districts, parks, and the U.S. Navy to identify parcels that could utilize recycled water for landscape irrigation.

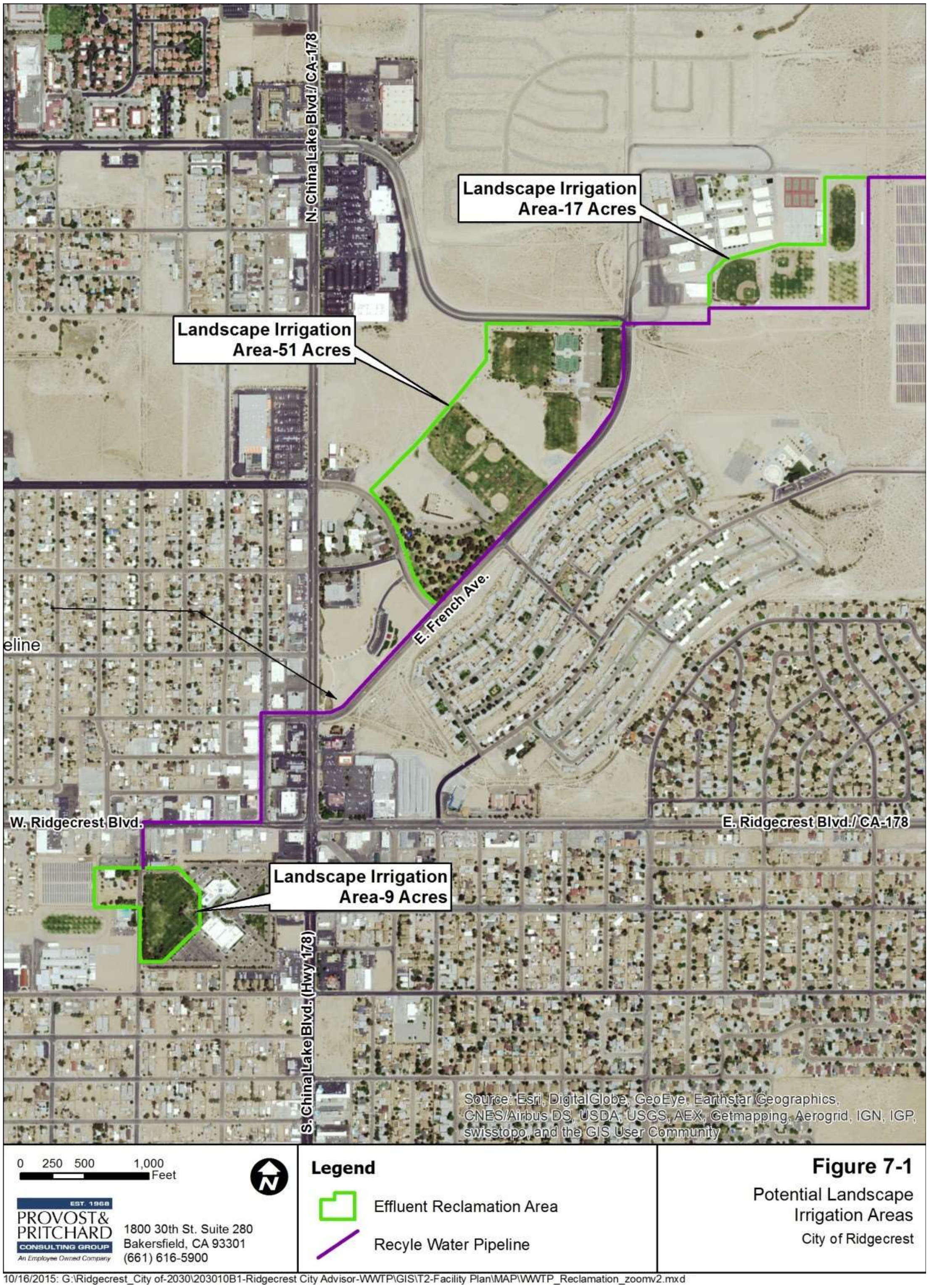

Figure 7-1. Potential Landscape Irrigation Areas

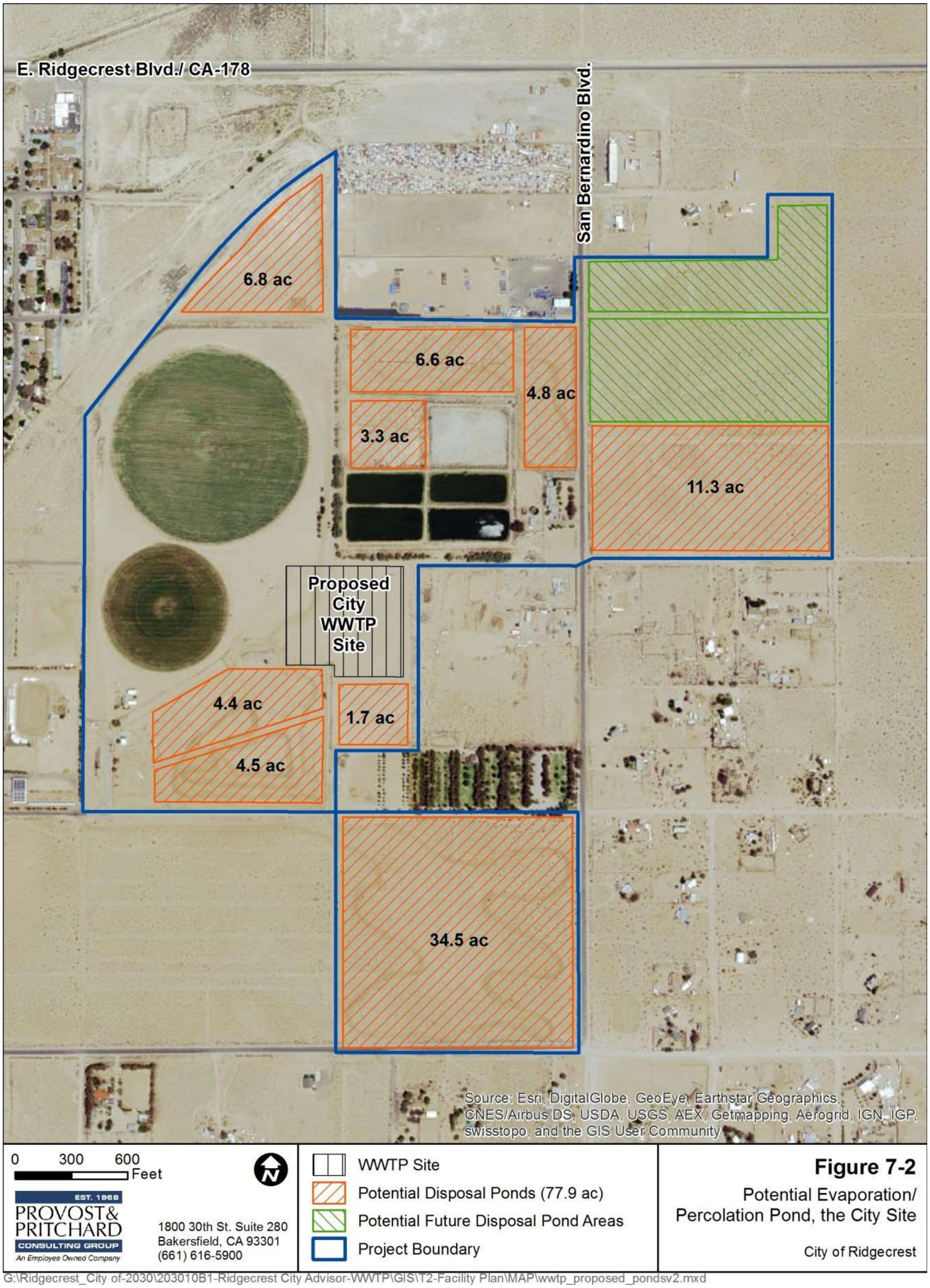

Figure 7-2. Potential Evaporation/Percolation Ponds at the City Site.

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8 BIOSOLIDS DISPOSAL ALTERNATIVES

As discussed in Section 3, the quantity and quality of wastewater biosolids (sludge) produced in Ridgecrest will vary somewhat with several primary factors:

- The population served;
- The characteristics of the wastewater treated
- The biological wastewater treatment process used; and
- The nature of sludge treatment processes.

While Section 3.7 discusses the historic and projected biosolids production, this section will focus on the disposal or use of that biosolids in the long term.

At the City's existing WWTP, solids are generated only from sedimentation in the primary clarifiers. These solids are characterized as primary sludge because they are not generated from a biological treatment process. Biosolids that are generated in the secondary treatment process remains in the facultative lagoons and is not removed or disposed off site. Because a biological treatment process will be utilized, the volume of biosolids will significantly increase and will require dewatering and disposal.

Several methods are used in the United States to dispose of biosolids generated by wastewater treatment. Most common in the western United States include disposal into landfills, use of a dedicated parcel of ground for land disposal, or land application. Land application requires additional processing of the waste to make it suitable for use in agriculture, such as digestion, composting, or both.

A number of public landfills exist within 100 miles of the City, including seven landfills operated by Kern County and an additional five operated by San Bernardino County. Considering the cost of truck transport, those nearest Ridgecrest would be least costly, including the Kern County landfills in Ridgecrest, Boron, and Mojave/ Rosamond, and the San Bernardino County landfill in Barstow. Landfill disposal generates several substantial operating costs: hauling, the value of landfill volume consumed, plus the lost value of the product itself as a soil amendment. Use of biosolids in an agricultural setting as biosolids is a preferable method. It should be noted that Kern County bans land disposal of biosolids, but biosolids can be applied as a soil amendment within City limits.

The practice of composting wastewater biosolids is becoming more common in the western US as well. Where practiced, the composting process uses large quantities of "bulking agents", neutral organic products such as wood chips, to mix with the biosolids. The resulting mixture is piled, aerated, and mixed to promote bacterial growth in the pile, and corresponding reduction of the organic matter in the biosolids. The resulting compost is a marketable product in locations where crops and landscaping generate sufficient demand. Efficient composting is best practiced where a ready source of bulking agents exists, and where markets are developed for sale and use of the compost; neither of which exists near Ridgecrest.

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When properly used directly for agricultural purposes, biosolids provides a beneficial soil amendment to improve soil texture, plus low levels of plant nutrients. The City of Ridgecrest presently land applies digested wastewater biosolids onto City owned and irrigated farm cropland. Prior to disposal the biosolids are anaerobically digested, dried, and stockpiled for more than three months. The resulting biosolids product is tested for deleterious substances and trace metals prior to spreading on the field. This method of biosolids disposal is approved by the regulatory agencies, relatively low in cost, and recognizes the value of the biosolids as a soil amendment.

Since other methods of biosolids disposal have higher costs and less benefit to the City, it is recommended that future disposal of biosolids continue in the same manner. However, the existing City owned farmed land will not be sufficient as biosolids production increases with the addition of mechanical secondary treatment. The remaining biosolids that cannot be utilized as a soil amendment will need to be hauled to a landfill for disposal.

9 WWTP SITE EVALUATION

9.1 Introduction

This section presents a discussion of WWTP site options for the City of Ridgecrest.

When the City first started the facility planning process in 2010, the City was considering constructing a new second wastewater treatment plant at the City site. The new plant would provide for future growth and the existing plant at the NAWS site would be upgraded and continue to operate at a reduced base flow. After some preliminary analysis, it was apparent that it would be in the City's best interest to own and operate a single WWTF. In general, one treatment plant is preferred over two treatment plants for various reasons:

- The RWQCB prefers to avoid proliferation of WWTP's.
- Permit monitoring and reporting is simpler for a single plant.
- Overall construction and operations costs for a single plant are lower than for two smaller plants due to the economy of scale.
- Fewer (larger) process components are used at a single plant; the overall potential for failures and outages is reduced.
- Utility service is more efficient for a single plant; this applies to telecommunications, electrical and standby power, and needed deliveries such as chemicals.
- It is less expensive to operate and maintain one plant due to lower costs of travel, maintenance of equipment, and communications.
- Minimum staffing levels dictate that two plants will necessarily have a higher operating labor cost than a single plant.

Two treatment plant sites may be advantageous under some of the following circumstances:

- The existing site is landlocked and cannot be easily expanded.
- The service area topography dictates that two plants are needed to avoid costly pumping.
- Effluent disposal systems are different and are separated geographically.
- Operating conditions at one site are restricted and a new site offers fewer limitations and greater control.

Considering the above factors, the first three items do not generally apply to the situation in Ridgecrest. The last item may apply in that the City of Ridgecrest WWTP is located on land owned and controlled by the U.S. Navy. This imposes restrictions where the City does not have full control of the site and operations. National security concerns may limit full access and control by the City. From a technical standpoint, without other overriding considerations, it is recommended that the City continue to operate one WWTP for both the City of Ridgecrest and NAWS.

9.2 WWTP Site Alternatives

Sites considered for the new WWTP are the China Lake NAWS site adjacent to the existing WWTP and the City site. No other sites were considered because to our knowledge, none are available that meet the needs of the City. A new site should have sufficient land; it should be located topographically down-gradient from the City, it would preferably be located adjacent to the existing trunk sewer, and it should be isolated from sensitive neighbors. The City site is illustrated in **Figure 9-1**. The City site is the original location of the City of Ridgecrest WWTP until the time that treatment was consolidated at the NAWS site. The NAWS site location is illustrated in **Figure 9-2**. This report considers various factors that may affect the site selection, including construction costs, operational costs, environmental factors, and non-monetary factors.

Disposal Alternatives

For either treatment plant location, both the NAWS and City sites will be utilized for effluent disposal. All existing effluent disposal methods and facilities are assumed to be maintained and fully utilized. Regardless of the treatment site selected, the existing 20inch effluent force main would continue to be used for effluent transfer to the other disposal site. Detailed discussion of disposal alternatives is included in Chapter 7.

Disposal capacity must be expanded and will be provided on the City site. Several additional disposal ponds will be necessary to dispose of effluent at a design AAD flow rate of 3.6 mgd. Based on the City's General Plan, the City site is located within the "Military Influence Area" (MIA). The impact of this classification on the construction of new disposal ponds has not been determined.

As sites are identified that could potentially utilize recycled water for irrigation, the City should consider implementing tertiary treatment and extending a purple pipe distribution system to those sites. Space for tertiary treatment can be provided at both sites. Because the points of use for tertiary recycled water are not known at this time, this factor has a minor impact on the location of the future treatment plant. However, if recycled water is used primarily at the NAWS site, the future treatment plant would be better located at the NAWS site. If recycled water is to be used exclusively in the City services area, the future treatment plant would be better located at the City site.

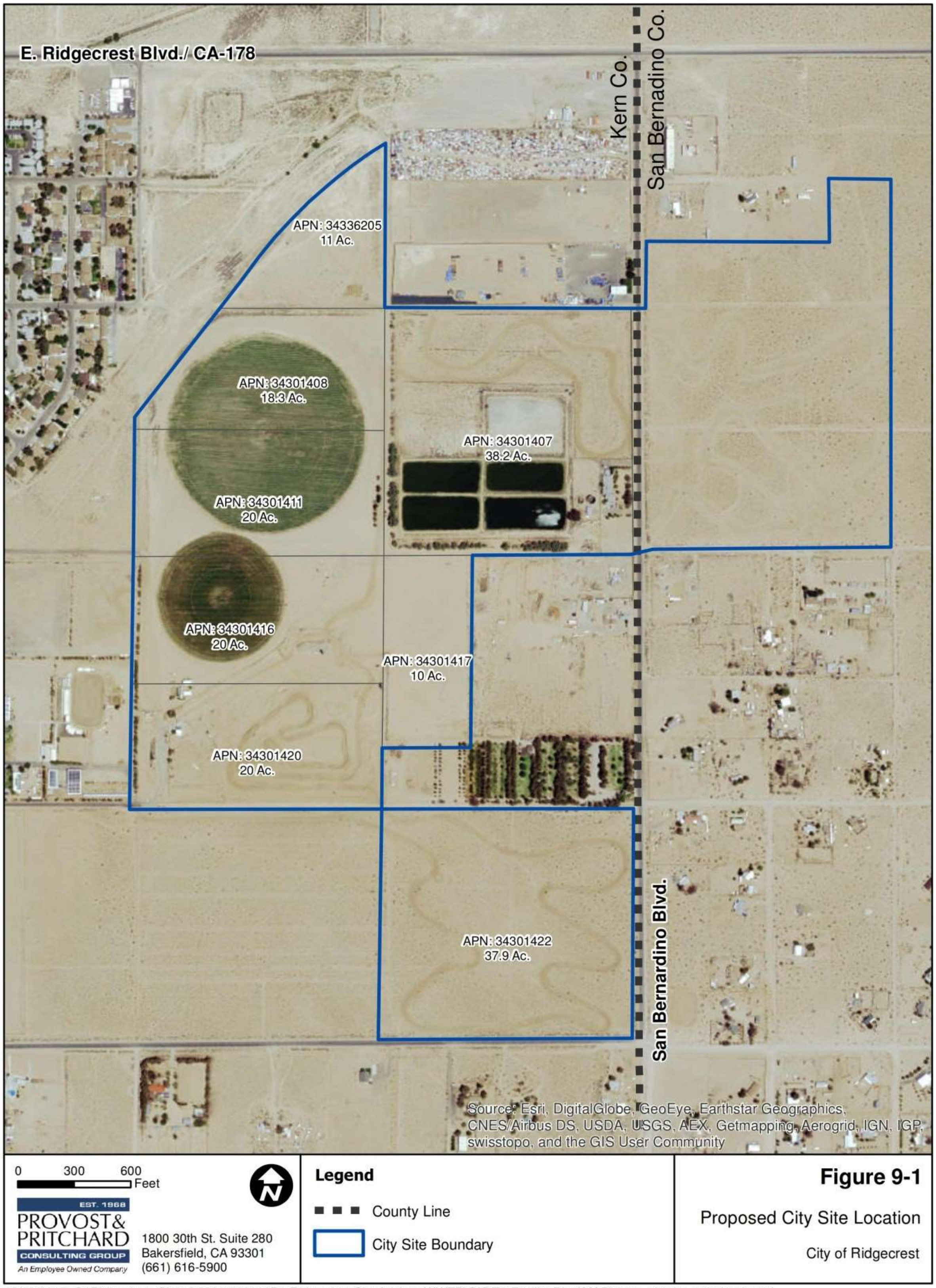

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Figure 9-1 - City Site Location

Figure 9-2. - NAWS Site Location

9.3 Assumptions

The evaluation of the alternate WWTP sites is based on several assumptions as follows:

- Existing WWTP will be completely abandoned after startup of the new WWTP.
- Facultative ponds will be converted to storage/evaporation ponds and existing percolation/evaporation ponds will remain in service. It may be possible to convert these ponds to percolate also.
- WWTP design capacity of 3.6 mgd AAD (expandable to 5.4 mgd AAD).
- Piping between plant sites are sized based on peak hour flow (PHF) at ultimate plant capacity.

9.4 Cost Comparison

9.4.1 Treatment Facilities

The ultimate 5.4 mgd plant will be constructed and implemented in phases. Phase 1 will have a AAD flow design capacity of 3.6 mgd, with 1.8 mgd added in Phase 2. Most capital costs of a new treatment facility at either site would be similar and would include:

- Headworks: Influent lift station, screening, grit removal and flow measurement. A septage receiving station will also be included.
- Secondary treatment facilities: Oxidation ditches with mechanical aerators and nitrification/denitrification, RAS/WAS pump station, mixed liquor distribution structure, secondary clarifiers.
- Effluent Handing: Pumps and piping to both disposal areas.
- Solids Handling: Aerobic sludge digester, mechanical dewatering equipment,
- Site Improvements: Influent sewer to headworks, paving, fencing, electrical, standby generator, utilities.
- Administration/lab building, maintenance building, SCADA controls, metering.
- Abandoning and demolishing portions of the existing WWTP.
- Provisions will be made for the incremental addition of tertiary recycling treatment facilities.

The capital cost of the first phase, 4.0 mgd primary and secondary treatment facilities (including abandonment/demolition of existing facilities) is anticipated to be approximately the same at either site. However, contractors may escalate their construction prices if the WWTP is located on the NAWS site because of the need to have equipment, materials and manpower enter the secured site on a daily basis. These additional costs are anticipated to be in the range of 3 to 5 percent. This analysis assumes an additional 5 percent construction cost to account for added labor, contractor risk, contingency and administrative costs related to NAWS site access. The actual premium for construction at the NAWS site, if any, will most likely be closely related to bidding conditions at the time. The capital costs for the new disposal ponds are summarized in **Table 9-1**.

Table 9-1. Pond Cost

Item		Ponds
City Site – 56 wet acres of ponds		$2,540,000
Engr/CM/Environment/Admin	32%	876,000
Contingency	20%	381,000
Total		$3,797,000

Disposal costs are assumed to be the same for both site alternatives. Capital cost will be required to add additional percolation / evaporation ponds at the City site for either of the two WWTP location alternatives. All options assume that secondary effluent would continue to be delivered to the City's alfalfa fields and the China Lake Golf Course would receive disinfected tertiary effluent.

Sludge digestion at either site would include aerobic digestion and would be similar in cost,

Biosolids dewatering at either site will include mechanical dewatering equipment with the hauling of a portion of the dewatered biosolids to the City site for use as a soil amendment with the majority of the biosolids being disposed of offsite at a composting facility or a landfill.

9.4.2 Collection System

Although the cost of a new treatment facility is roughly equal at either site, there are differences in the required sewer system that must also be considered in the comparison. If the WWTP is located at the City site, a new sewer force main and lift station are needed to deliver raw wastewater from China Lake NAWS and the northern portion of the City (Sewer Service Area #1) to the new WWTP at the City site. The City site elevation is approximately 70 feet higher than the NAWS site. A minimum of three high head solids handling pumps with motors of approximately 50 to 60 hp will be required. Annual operating costs for pumping will be significant. The force main would be sized based on peak hour flow, and would need to accommodate flow from the portion of the City generally north of Drummond Avenue as well as accommodate flows from the NAWS service area. This combined flow is projected to be approximately 1.8 mgd on an average annual day basis by 2050. Based on a peaking factor of 2.0, the peak hour flow from Sewer Service Area #1 is 3.5 mgd (2,400 gpm). In order to accommodate this flow, a 16-inch sewer force main approximately 4 miles in length would need to be constructed from the NAWS site to the new WWTP   The force main would follow the alignment of the existing 20-inch effluent force main.. This lift station at the NAWS site will be a substantial installation requiring City staff onsite supervision on a daily basis. This report assumes that the lift station will pump raw sewage, further evaluation during design may indicate that screening and grit removal prior to pumping would be preferred. Removing solids at the NAWS site would require additional City staff operations at this location.

If the City site is selected, the existing trunk sewed now flowing to the NAWS site must be intercepted at Lumill Street and California Avenue and redirected to the City site. Approximately 1,600 feet of 27 inch sewer will be constructed to direct flow to the new influent lift station at the City site.

Other cost differences may include additional odor control facilities at the discharge point of the NAWS raw wastewater force main where it discharges to the sewer or headworks at the City site. Other differing costs may also include differences in yard piping, fencing, pavement and site improvements but these differences are minor when compared to the additional cost of the raw wastewater pimp station and force main described above. If the City site is selected and a tertiary component is developed to produce disinfected tertiary recycled effluent a new purple pipe will need to be constructed to deliver recycled water to the NAWS site and golf course

The approximate capital, operation and present worth costs for these additions are shown in **Table 9-2**. The approximate locations of the lift station and sewer force main are shown **Figure 9-3.**

Table 9-2. Cost Estimate for Sewer Service Area #1 Lift Station and Force Main

Item		
Lift Station and Force Main		$3,691,600
Engr/CM/Environment/Admin	30%	$1,273,600
Contingency	15%	$553,700
Total for Disposal		$5,518,900
Annual O&M Cost		$78,000
O&M Present Worth, 2.5%	20 yrs	$1,216,000
Total Present Worth		$6,734,900

9.4.3 Operating Factors

The City site alternative would have additional operating expenses related to sewage pumping. Operation of a new WWTP at the City site may be less expensive due to travel and time expended related to daily access of the NAWS site. This will be a relatively minimal savings, which will not greatly influence the choice of site. The major cost items, labor and electric energy are approximately equal for both plant sites

The existing NAWS disposal ponds as well as existing and new disposal ponds at the City site would be needed for either WWTP location. A new WWTP at the City site would require staff to still monitor the disposal ponds within the NAWS in addition to monitoring the sewage lift station. The choice of WWTP site is therefore independent of effluent disposal.

9.4.4 Preferred Site based solely on Cost

The capital cost differences related to the site alternatives are summarized in **Table 9-3**.

Table 9-3. Capital Costs by Site

	Alternative 1 - City Site 4.0 MGD		Alternative 2 - NAWS Site 4.0 MGD	
	Description	Cost	Description	Cost
Collection System	New lift station at NAWS and new force main to pump raw sewage to new site.	$5,518,900	Sewer System Improvements	$ -
Primary and Secondary Treatment Processes	New Primary and Secondary Treatment Process.	40,720,600	New Primary and Secondary Treatment Process (Includes additional bid cost for NAWS Site Access)	40,465,800
Effluent Disposal	New Percolation Ponds.	3,640,300	New Percolation Ponds.	3,797,300
Total		$49,880,000		$44,263,000

The annual operation costs are shown in **Table 9-4**. The main differences are related to sewage pumping and access into a secured site. Operating cost difference is estimated to be approximately 6 percent in favor of the City site.

Table 9-4. Operational Costs by Site

	Alternative 1 - City Site 3.6 MGD		Alternative 2 - NAWS site 3.6 MGD	
General	Labor, utilities and materials	$715,000	Labor, utilities and materials	$715,000
Sewage Pumping	Maintenance and Power Cost	30,000		-
Effluent Pumping			Maintenance and Power Cost	25,000
Biosolids Hauling	Truck cost and Driver	30,000	Truck cost and Driver	60,000
Security measures	Limited visit by staff or visitors to NAWS site	10,000	More staff and visitors to NAWS site	30,000
Total Annual Cost		$785,000		$830,000

9.4.5 Future Tertiary Treatment by Site

The capital cost of tertiary treatment as discussed in Section 6.9 and disinfection in Section 6.10 are shown in **Table 9.5**. Pipelines from the wastewater plant locations to the potential City landscape areas are very similar, whereas the greatest difference in cost is transmission from the City site to NAWS for golf course and other potential irrigation areas, which are not reflected in the costs below.

Table 9.5. Tertiary Treatment

Tertiary Treatment		$3,127,500
Disinfection		590,300
City Landscaping Transmission		1,800,000
Total		$5,517,800
Engr/CM/Envrionment/Admin	30%	1,903,600
Contingency	15%	827,700
Total		$8,249,800

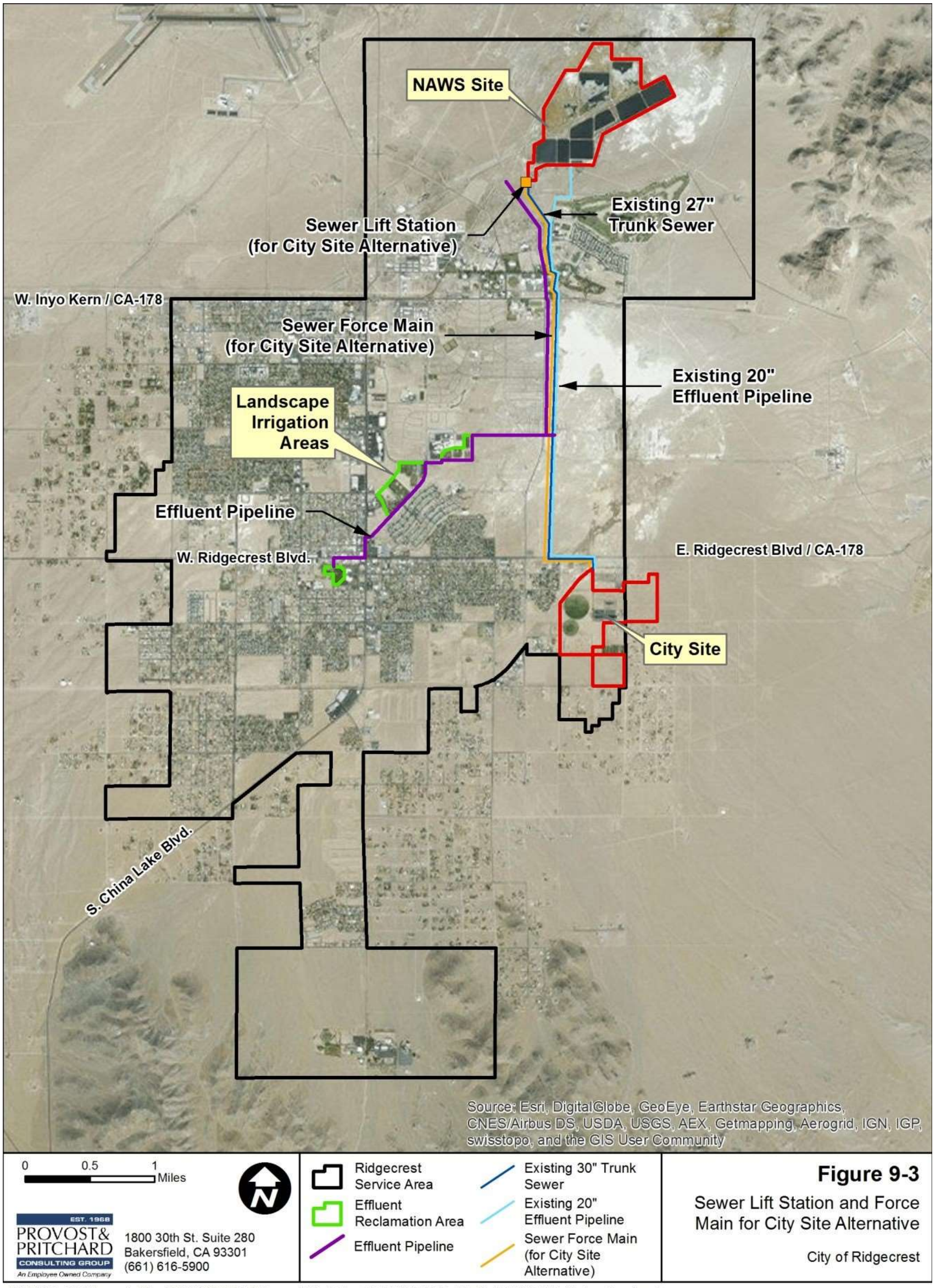


Figure 9-3 - Sewer Lift Station and Force Main for City Site Alternative

9.4.6 Comparison of other Factors

There are many non-monetary factors that should be considered in making a site selection. What these factors are, their importance, and their ranking are largely subjective. The factors are not equal in importance and thus must be weighted according to their perceived importance, as determined by the City. Ultimately it is up to the City to determine the importance of each and make the final site selection. In some cases, similar to an environmental evaluation, there may be overriding considerations that trump cost and other considerations.

One factor regarding the existing site is its location on NAWS land under agreement with the Navy. City operators must access the WWTP daily through Navy security gates. Visitor access and deliveries must be coordinated in advance. While this has been manageable for the City, it would be preferable to operate the WWTP without dealing with secured access and other restrictions related to the NAWS location.

Both site alternatives will require continued access to the NAWS site. If the City site is selected for the WWTP, City operators would still need to access NAWS on a daily basis to operate the sewage lift station for Sewer Service Area #1 as well as monitor the effluent storage ponds. On the other hand, having the WWTP remain at the NAWS site would provide the simplest transition to the new WWTP as the sewers are already in the ground, and the entire flow is by gravity to the NAWS site. Additionally there are no residential areas in the vicinity of the NAWS site, therefore from a public acceptability standpoint the chance of odor and vector nuisance conditions is much less.

Another important factor may be control and autonomy. The City WWTP is located on the NAWS site under an easement that has a limited term that is subject to conditions imposed by the Navy. It is impossible to predict what future conditions may be imposed. If the WWTP were located on land outside the base and on City owned land, the City could presumably maintain complete control and autonomy over the WWTP and its operations. While this is an important factor, it could be mitigated during negotiations for the revised agreement with NAWS.

Table 9-6 includes the non-economic evaluation factors with preliminary weighting and scores by site. Based on a preliminary analysis of non-economic factors, a new WWTP on the NAWS site is preferred over a new plant at the City site. Environmental factors will be assessed in the CEQA/NEPA environmental review.

Recommendations

Cost differences between alternative sites are significant, with the NAWS site lower in construction and operating cost. The NAWS site also has significant technical advantages because it is at a lower elevation and can receive all flow by gravity sewer. It is also more remote (isolated from residents) and more secure than the City site. However, there are several non-economic factors and environmental factors that should be considered in the site selection process. Based on the weighting factors and scores illustrated in **Table 9-4**, the NAWS site is preferred. Because this is a subjective evaluation criteria, others may score and weight the factors differently. The environmental analysis is not yet complete and it may reveal environmental impacts that should also be considered. It is therefore recommended that the City delay the final decision on a site alternative until the environmental analysis is complete.

Limitations

Estimates of costs discussed in this section are preliminary and to be used for relative site comparison purposes only. These estimates are not intended to be absolute but rather relative concept level opinions of construction or operations and maintenance costs. As the project is further defined and a more complete design is developed, more reliable and detailed estimates will be prepared.

Table 9-6. Non-Economic Factors for Evaluation of Site Alternatives

Item	Factors to consider	Weighting	NAWS Site	City Site
Accessibility	Is it easy for operators to gain entry? Is it necessary to pass through security to gain entry? Can contractors and visitors easily gain entry? How much time is lost securing permission for entry? What is the time and hassle factor for City employees to obtain entry for visitors and contractors? Will accessibility change in the future for better or worse?	5%	6	8
Proximity	Is the site close to the City and its other public works operations? How much travel time is expended getting to and from other city operations?	5%	7	10
Proximity to potential recycle water users	Can a purple pipe distribution system be economically constructed to serve recycled water to future users? Is the WWTP site well centered?	10%	9	7
Expandability	Can WWTP facilities be easily expanded on the site? Will future security provisions limit the ease of expansion?	5%	7	10
Flexibility	Can WWTP operations be easily adapted to unforeseen future conditions? Can the site easily adapt to changing regulatory requirements?	5%	7	10
Autonomy	Can the City construct and operate the facility without undue interference with outside agencies? Can the City control its own destiny?	15%	6	10
Regulatory Oversight	Can the City meet current and future regulatory requirements?	5%	8	8
Security	Is the site secure from vandalism, theft and outside threats?	5%	10	8
Odor & Nuisance Conditions	Will neighbors be subject to possible odor, noise and other nuisance conditions? Does the WWTP have adequate distance to buffer nuisance conditions?	15%	10	8
Public Acceptability	Will the public accept the new WWTP at this site? Is there a strong “NIMBY” sentiment? Can the WWTP possible nuisance conditions be mitigated such that the WWTP is acceptable to the public?	15%	10	7

Connectibility	Ease of connecting to existing system. Is the site positioned well topographically for gravity sewer service? Is it close to existing sewers?	15%	10	6
	Total weighted score	100%	8.55	8.05

Subjective scoring: 10 highest (best for City) score, 1 lowest score.

10 RECOMMENDED FACILITIES

10.1 Recommended Project

Based on the previous several sections, the recommended project will include a 4.0 mgd oxidation ditch secondary treatment plant located on the NAWS site, with effluent disposal to existing facilities, as well as new percolation and evaporation ponds located at the City site. It will also include mechanical biosolids dewatering and disposal as a soil amendment at the City alfalfa fields, but with the majority of the biosolids being disposed of off-site in an approved landfill,.

10.1.1 Treatment

The first phase WWTP will consist of a 4.0 mgd secondary treatment facility with biological nitrogen removal. The secondary treatment process recommended for the City of Ridgecrest is an oxidation ditch process consisting and two oxidation ditches and two circular clarifiers. A third oxidation ditch and clarifier will be added for a Phase 2 expansion to 5.4 mgd at some point in the future. Effluent from the clarifiers will be discharged directly to percolation and evaporation ponds located on both the NAWS site and City site. Provisions will be made for the construction of tertiary treatment facilities to provide up to 1.8 mgd of recycled water to b used for golf course irrigation and landscape irrigation. The WWTP will also include an influent pump station and headworks, office/lab building, maintenance building, aerobic digestion, mechanical biosolids dewatering, and effluent disposal. The proposed WWTP facilities are shown on **Figure 10-1** (NAWS site). **Figure 10-2** shows the proposed WWTP facilities for the City site alternative.

Once the new WWTP is constructed, the existing WWTP will be abandoned, with the majority of existing facilities being demolished.

The recommended disinfection method for the tertiary plant is chlorine disinfection. Chlorine disinfection consists of the injection of liquid sodium hypochlorite into the effluent stream. After injection, the effluent will flow through a chlorine contact basin to provide contact time. Chlorine offers the advantage of continued residual disinfection after the initial injection because chlorine residual remains in the water. Following chlorination the recycled water will be pumped into a 1.8 MG storage tank. Recycled water can flow to the China Lake Golf Course by gravity for golf course irrigation. A booster pump system will be provided to pump recycled water into a “purple pipe” recycled water distribution system.

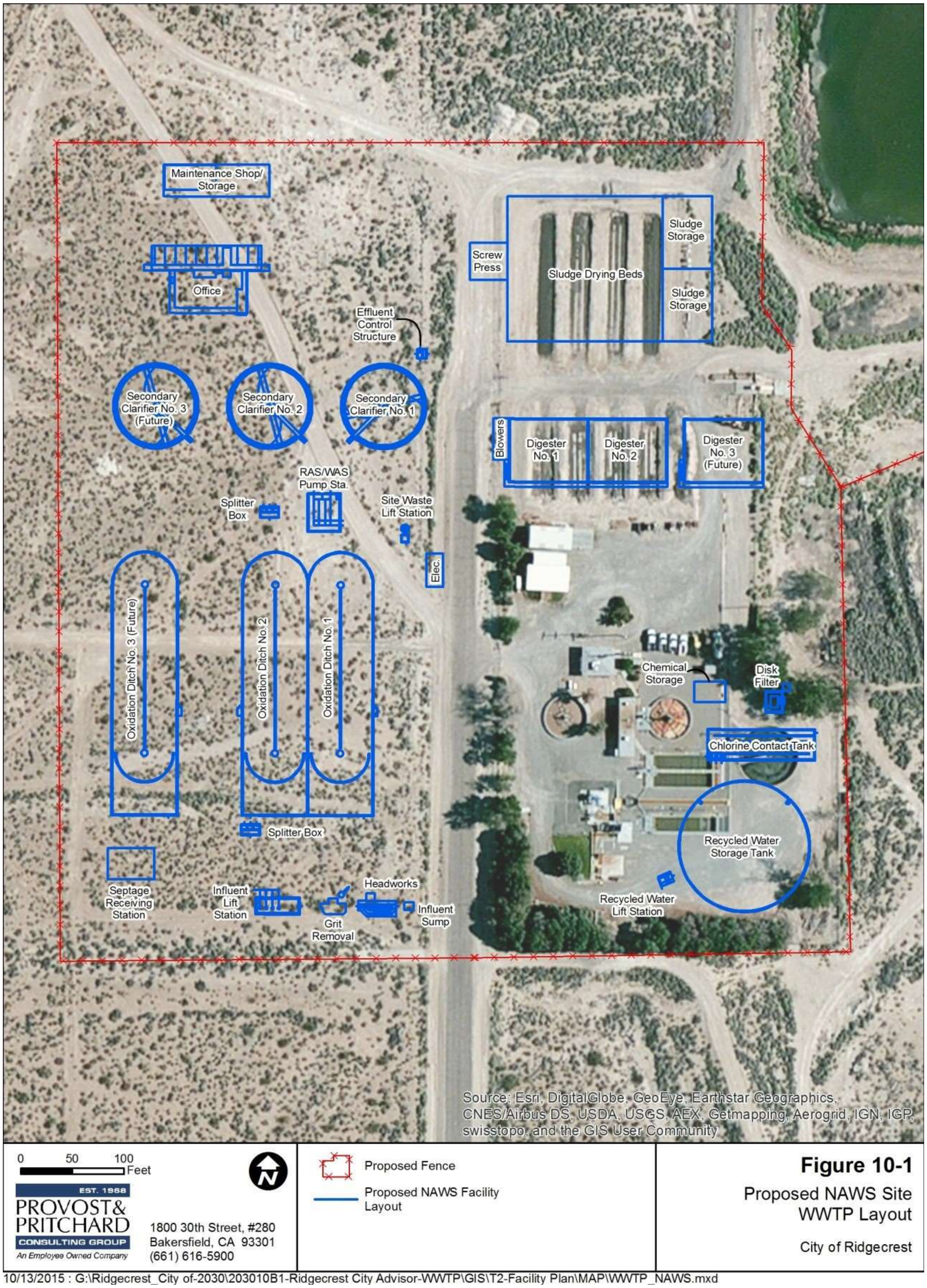


Figure 10-1. - Proposed NAWS Site WWTP Layout

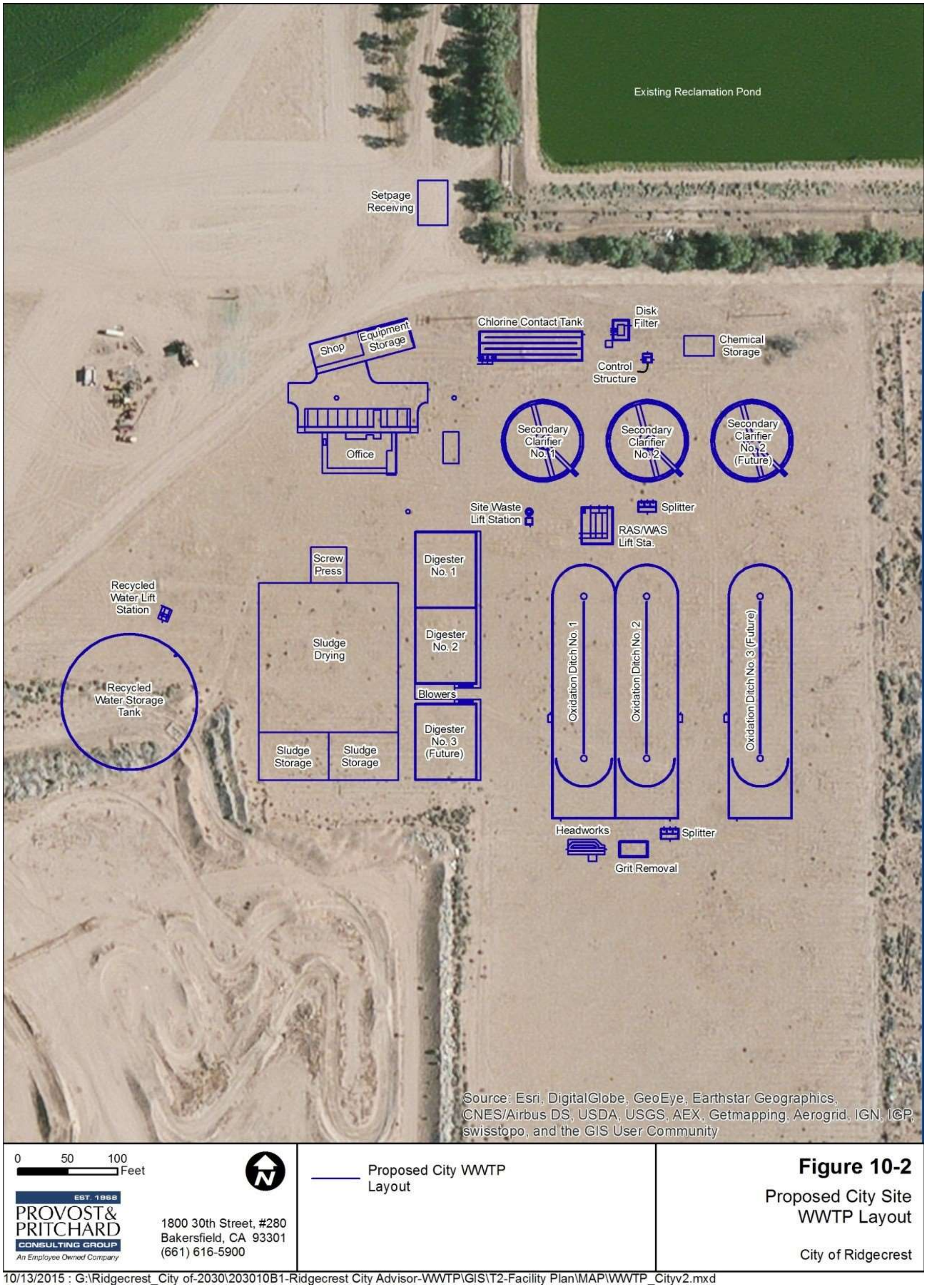


Figure 10-2. Proposed City Site WWTP Layout

10.1.2 Effluent Disposal

Existing effluent disposal capacity is limited, and is nearly insufficient for the current flows. It is therefore recommended that new effluent disposal ponds be constructed immediately on the City site. It is recommended that all existing disposal methods and locations continue to be utilized, including maintaining and upgrading existing disposal ponds 8 and 11 in order to put them back into service. As discussed in Section 7, at the City site approximately 56 wet acres of additional disposal ponds will be required to meet the proposed capacity of 4.0 mgd (ADMM), or 3.6 mgd on an annual average daily basis.

Approximately 1.6 mgd of demand for tertiary treated recycled water has been identified within the City of Ridgecrest and the China Lake Golf Course. This capability would entail the additional treatment facilities at the NAWS WWTP, with stand alone recycled water distribution facilities. To distribute the effluent would require recycled water storage, booster pump stations, and pipelines to the China Lake Golf Course and the City landscaped areas.

Future disposal capacity beyond 3.6 mgd may include construction of more ponds, or irrigate public landscape areas if the City elects to treat a portion of the flow to tertiary standards as discussed above.

10.1.3 Biosolids Disposal

The proposed WWTP includes mechanical dewatering of the biosolids, with dried biosolids being subsequently stockpiled in the same manner as currently used. These biosolids are recommended to continue to be used as a soil amendment to the extent possible on the City site. As biosolids production will increase beyond what the available crop land is able to accept, it is recommended that the excess biosolids be hauled to a landfill or composting facility for disposal.

10.2 Project Costs

10.2.1 Capital Costs

As shown in **Table 10-1**, the total capital construction cost for the proposed 4.0 mgd secondary WWTP, including abandonment and partial demolition of the existing WWTP, is approximately $29.6 million. At the proposed NAWS site, there is anticipated to be an additional increase in construction costs due to site access. There are also costs associated with engineering, construction management, environmental permitting, financing costs, as well as a contingency, totaling about $14.6 million. There will be additional capital cost associated with construction of the proposed disposal ponds. Capital cost for approximately 56 wet acres of percolation and evaporation ponds is anticipated to be approximately $2.5 million with costs associated with engineering, construction management, environmental permitting, financing costs, as well as a contingency, totaling about $1.2 million. The total capital cost for the recommended Phase 1 WWTP on the NAWS site is projected to be approximately $44.3 million.

Additional capital costs will be required if tertiary treatment is included. Added capital cost for tertiary treatment including recycled water distribution for 1.6 mgd is projected to be approximately $5.4 million with costs associated with engineering, construction management, environmental permitting, financing costs, as well as a contingency, totaling about $2.8 million.

Table 10-1. Summary of Project Costs

Item	Cost
Secondary WWTP	$29,600,000
Disposal Facilities	$2,540,000
Engr/CM/Environmental	$10,200,000
Contingency (15%)	$4,400,000
Total	$44,300,000

10.2.2 Operations and Maintenance Costs

Annual operations and maintenance costs for the proposed 4.0 mgd secondary WWTP is projected to be approximately $875,000.

10.2.3 Cost Impact to Users

The cost impacts to users will be the subject of a financial study to be completed in a subsequent Project phase.

10.3 Environmental Impacts

Environmental impacts will be addressed during the CEQA process. At the City's request, CEQA documentation is being prepared for both the NAWS site and the City site.

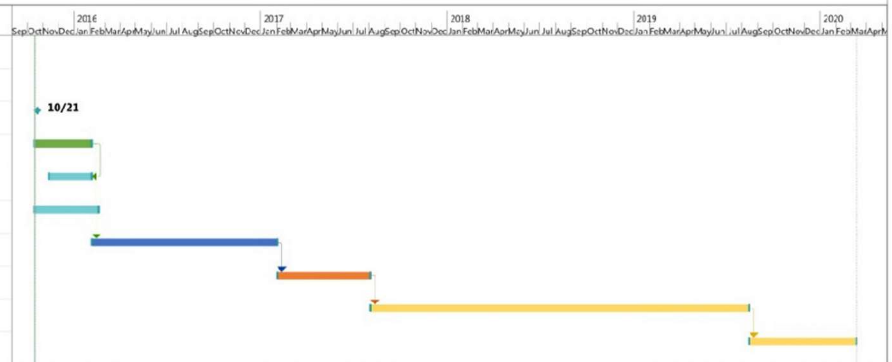
10.4 Schedule

Figure 10-3 presents the proposed schedule for construction of the new WWTP. The City has elected to utilize a design build process for construction of the new WWTP and this is reflected in the proposed schedule.

City of Ridgecrest, CA
Wastewater Treatment Plant Facility Plan
Implementation Schedule

Provost & Pritchard Consulting Group
October 2015

ID	Task Name	Duration	Start	Finish	Predecessors
1					
2					
3	30% Design Presented to Council	0 days	Wed 10/21/15	Wed 10/21/15	
4	CEQA/NEPA Documentation	80 days	Fri 10/16/15	Thu 2/4/16	
5	Report of Waste Discharge	60 days	Fri 11/13/15	Thu 2/4/16	4FF
6	SRF Application	90 days	Fri 10/16/15	Thu 2/18/16	
7	Final Design	260 days	Fri 2/5/16	Thu 2/2/17	4
8	Bidding & Award	130 days	Fri 2/3/17	Thu 8/3/17	7
9	Construction	530 days	Fri 8/4/17	Thu 8/15/19	8
10	Start-up & Final Completion	150 days	Fri 8/16/19	Thu 3/12/20	9



Schedule - Facility Plan.mpp
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Figure 10-3. Preliminary Project Schedule

11 IMPLEMENTATION

Because the City of Ridgecrest desires to accommodate growth as it may occur, and because the existing wastewater treatment facility is nearing the end of its useful life, it is now time for the City to begin a structured process to replace its wastewater treatment plant. The following provides a general overview of that process; including several approaches recommended to allow the needed infrastructure to be implemented smoothly. The following implementation plan is divided into several phases for convenience. Under each phase, a series of tasks are listed. The tasks listed within each phase is not necessarily chronological, but merely a checklist of activities that will need to be completed before moving to the next phase.

11.3 Negotiations with NAWS

The recommendation that the WWTP be constructed adjacent to the existing NAWS site is contingent upon the negotiation of a new agreement with the Navy. In order to finance the project, a long-term agreement or license with the Navy will be required. Otherwise, the City may not be able to secure favorable finance terms. Negotiations of the conditions and terms of agreement for use of NAWS land should continue. As the City investigates funding for the project, requirements relating to the ownership/lease of the NAWS site should be discussed with the potential funding agencies and included in negotiations with NAWS.

11.4 Planning and Permitting Phase

- City review and acceptance of the concepts and approach described in this Facility Plan.
- Complete the necessary CEQA investigations for the project described, and prepare draft and final environmental documents.
- Formal City approval of the Facility Plan and the related CEQA documents.
- Authorize and conduct the necessary applications for SRF and other funding mechanisms, as appropriate.
- Authorize and conduct the necessary rate studies for cost of service adjustments.
- Confirm the method of procurement to be used for contractor selection; design/build (D/B) process or design/bid/build (DBB).
- Complete 30% design of the facility.
- Establish a firm schedule for all remaining activities.
- Prepare a Report of Waste Discharge for the initial phase WWTP and submit an application to the RWQCB for Waste Discharge Requirements for the new WWTP. Submit a Title 22 report to the Division of Drinking Water for recycled water use on the alfalfa fields and the China Lake Golf Course.

11.5 Contractor Selection Phase (Design Build Option)

- Advertise the upcoming project to potential teams, and accept statements of qualifications from interested design/build teams. Screen and rank the qualification statements received, and select teams to develop subsequent cost proposals.
- Formally request proposals from qualified teams, and accept, review, evaluate, and rank the proposals received. Select a preferred team for the design/construction process.
- Award the project with approval of funding agency and process necessary contracting documents for the selected team.

11.6 Design and Permitting Phase

- Work closely with the selected D/B team to complete an acceptable design on the selected site.
- Work with regulators to secure the necessary permits for treatment and disposal operations.

11.7 Construction Phase

- Authorize the D/B team to proceed with purchases and construction.
- Monitor the construction as it progresses; process changes and pay requests as needed.
- Inspect and accept the new facilities as they become available.

11.8 Startup and Operations Phase

- Oversee testing and startup of the new facility.
- Divert wastewater to the new facility, startup the unit processes, and begin operation of the plant.
- Monitor plant operations and performance during initial operations.
- Abandon / demolish existing WWTP facilities.

11.9 Future Activities Phase

- Perform periodic reviews of user charges, to assure that sufficient funds will be available for expansion of the new facility.
- Construct an expansion to the new facility.

SECTION TWELVE

12 REFERENCES

BSK Associates, *Draft Preliminary Soil Investigation Report, Ridgecrest Wastewater Treatment Plant,* dated January 7, 2011.

Carollo Engineers, *City of Ridgecrest Wastewater Treatment Plant Final Project Report,* dated September 2008.

Converse Consultants Southwest, Inc., *Preliminary Slow Rate Infiltration Study, Inactive Sewage Treatment Facility, Ridgecrest, California*, dated April 12, 1991.

Tetra Tech EM Inc., *Winter 2010 Semiannual Groundwater Monitoring Report, City of Ridgecrest Wastewater Treatment Facility*, dated March 10, 2010.

Master Sewer Plan for the City of Ridgecrest, 1981

Revised Master Sewer Plan for the City of Ridgecrest, 1987

Master Plan for Wastewater Treatment Facilities, 1994

City of Ridgecrest Wastewater Treatment Plant Expansion, 1998

City of Ridgecrest – Sewer Study Letter, 2002

Groundwater Management in the Indian Wells Valley Basin – Ridgecrest California, 2003

Indian Wells Valley Water District – 2005 Urban Water Management Plan, 2005

City of Ridgecrest Wastewater Treatment Plant Final Project Report, 2008 (“Carollo Report”)

Winter 2010 Groundwater Monitoring Report, City of Ridgecrest Wastewater Treatment Facility, Ridgecrest, California,Tetra Tech, 2010.

Winter 2014 Groundwater Monitoring Report, City of Ridgecrest Wastewater Treatment Facility, Ridgecrest, California, BSK Associates, February 6, 2014

Indian Wells Valley Water District 2013 Consumer Confidence Report

Appendix A - Existing Waste Discharge Requirements

Appendix B - Water Balance Calculations