Homer Water and Sewer Master Plan

Final

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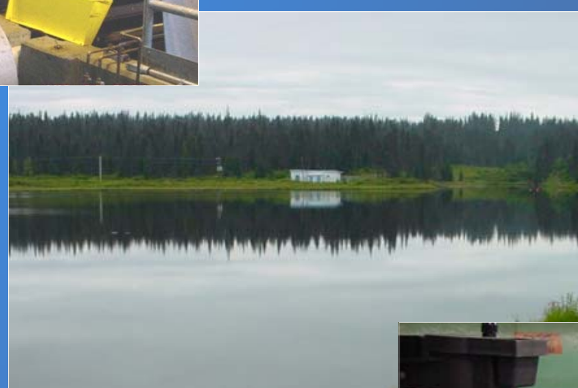

Prepared for:

CITY OF HOMER ALASKA
MARCH 31, 1964

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TABLE OF CONTENTS

TABLES

TABLES (continued)

CHARTS

DRAWINGS

APPENDICES

ACRONYMS AND ABBREVIATIONS

'	minutes
°	degrees
°F	Degrees Fahrenheit
AC	asbestos cement
ADEC	Alaska Department of Environmental Conservation
ADNR	Alaska Department of Natural Resources
ADOT&PF	Alaska Department of Transportation and Public Facilities
AWWA	American Water Works Association
BOD	biochemical oxygen demand
CI	cast iron
CIP	capital improvement project
City	City of Homer
CT	chlorine concentration and time of detention
DCED	Alaska Department of Community and Economic Development
DI	ductile iron
DPB	disinfection byproducts
FAA	Federal Aviation Administration
GIS	Global Information System
gpcd	gallons per capita per day
gpd	gallons per day
gpm	gallons per minute
HDPE	high-density polyethylene
HDPW	Homer Department of Public Works
HEA	Homer Electric Association
hp	horsepower
I/I	inflow/infiltration
ID	inner diameter
ISO	Insurance Services Office
KPB	Kenai Peninsula Borough
LF	linear foot/feet
LID	Local Improvement District

ACRONYMS AND ABBREVIATIONS (continued)

MG	million gallons
mg/L	milligrams per liter
MSL	mean seal level
NPDES	National Pollutant Discharge Elimination System
O&M	operation and maintenance
PRV	pressure reducing valve
psi	pounds per square inch
PVC	polyvinyl chloride
R&R	repair and replacement
RM	Richter Magnitude
ROW	right-of-way
RUS	Rural Utilities Service
SCADA	Supervisory Control and Data Acquisition
SHPO	State Historic Preservation office
SID	Sewer Improvement District
STEP	Septic Tank Effluent Pump
TTHM	total trihalomethanes
USACE	U.S. Army Corps of Engineers
VGES	variable grade effluent system
WID	Water Improvement District
WSMP	Water and Sewer Master Plan
WST	water storage tank
WTP	water treatment plant
WWTP	wastewater treatment plant

EXECUTIVE SUMMARY

This Water and Sewer Master Plan (WSMP) was prepared for the City of Homer (City), Alaska, to provide guidance for future improvements and expansions to the community's water and wastewater facilities. This document provides an evaluation of the construction costs, treatment plant operation and maintenance (O&M) costs, technical requirements, system management impacts, estimated revenues and expenses to operate an expanded system, and other related considerations for these proposed improvements and expansions. The WSMP covers a 20-year planning period (2006 through 2025). The WSMP includes an evaluation of population projections and resulting water and wastewater flows for the City, a summary of the recommended improvements, and a discussion of the capital and O&M costs for these improvements.

Recommendations proposed in this WSMP include phased construction of water and wastewater facilities. These improvements have been phased, or defined in manageable parts, to allow for a reasonable funding stream based on priority. This WSMP is the first stage of a three-stage process to ultimately provide needed improvements to the community utility systems. The second stage is preparation of construction documents. The third stage is the construction of facilities.

It is recommended that the City of Homer (City) review this WSMP every 5 years and update this WSMP every 10 to 15 years, assuming population growth rates remain relatively consistent with this WSMP, and expansion of the system occurs within the boundaries of the study area. This WSMP may need to be updated more frequently if major population or economic changes occur within the City.

CONCEPTUAL LAYOUT

A conceptual layout of proposed water and sewer system improvements is presented on Sheets 1 through 38, which appear at the end of the text of the WSMP.

CONSTRUCTION COSTS

The final recommendations, as prioritized by the City, along with the estimated design/construction costs are presented below.

Recommended Water Improvements

Costs for recommended water system improvements over the next 20 years are shown in Table ES-1 and summarized in Section 9.2 of this WSMP.

Table ES-1 Cost Estimates for Proposed Water System Improvements

Phase	Recommended Improvement	Design/Construction Costs ($)
1.0	Water Supply	3,400,000 – 4,500,000
2.0	Water Treatment Plant	6,600,000 – 8,300,000
3.0	Water Storage (4 MG total)	6,900,000
4.0	Water Distribution System Replacement and Repairs	3,885,000
5.0	Water Distribution System Expansions	31,411,000
	Total	**51,346,000** (average)

Notes:

MG = million gallon

Recommended Wastewater Improvements

Costs for recommended wastewater improvements over the next 20 years are shown in Table ES-2 and summarized in Section 9.3 of this WSMP.

Table ES-2 Cost Estimates for Proposed Wastewater System Improvements

Phase	Recommended Improvement	Design/ Construction Costs ($)
6.0	Wastewater Treatment Plant	32,418,000
7.0	Sewer Collection System Rehabilitation and Repair	1,792,000
8.0	Collection System	36,467,000
	Total	**70,677,000**

1.0 INTRODUCTION

1.1 PURPOSE

During the course of preparing this WSMP, options and recommended improvements were developed with the assistance of Homer Department of Public Works (HDPW) staff, and presented and discussed with the City Council and general public in a series of public hearings and council meetings. The final recommendations and priorities of construction are discussed in Section 9, and Appendices A through C. This WSMP was submitted to the Alaska Department of Environmental Conservation (ADEC) to ensure that the recommended plan is an available reference for future planning and construction. This WSMP is intended to conform to applicable ADEC and local guidelines and requirements, and is an update to master plans for the water supply and sewage collection systems prepared in 1983, and the 1977 comprehensive water plan. In the past 23 years, Homer has experienced significant growth and changes that have impacted the water and sewer systems. The recommendations in this WSMP are intended to serve as a guide for the City to plan for phased construction of needed future water and wastewater facilities to serve Homer; and to identify funding requirements for these improvements. This WSMP was written to conform to current City Comprehensive Plan and draft zoning designations.

Technical information and evaluations of future expansion and modification scenarios for the water treatment plant (WTP) and wastewater treatment plant (WWTP) are provided in Appendices A and B, respectively. An analysis of the Bridge Creek Reservoir and future water supply options is provided in Appendix C. A proposed Capital Improvement Program (CIP) schedule is provided in Appendix D. The City Council resolution adopting this WSMP is included as Appendix E. Records of public meetings conducted under this project are included as Appendix F.

1.2 SCOPE

The scope of work for the Homer WSMP includes the following criteria:

- Define and review the existing water and wastewater infrastructure.
- Obtain updated land use information for the study area.
- Prepare current mapping of the City to include water and sewer systems, land status, topography, flood plain, and aerial photography.
- Review the USKH 2003 Infiltration and Inflow (I/I) Study to determine impacts on flows to the WWTP.
- Prepare models of the existing piped water distribution and sewer collection facilities in a Global Information System (GIS)-based platform to allow for evaluation of future utility expansion scenarios.
- Prepare base maps of the study area include the existing system and recommended future improvements and expansions. The base maps will be provided to the City in both AutoCAD and GIS format for future use and regular updates.

- Identify deficiencies and needed replacement/upgrade projects for the existing water distribution and sewage collection systems, including pipe replacements, lift stations, pressure reducing valves (PRVs), and water storage tanks (WSTs).
- Analyze alternatives for future improvements and expansion of service for a 20-year planning period (2006 through 2025).
- Provide recommendations for improvements to the existing water distribution and sewer collection systems to continue serving the customer base and to accommodate growth.
- Provide recommendations for future modifications/upgrades to the WTP and WWTP.
- Provide recommendations for future water supply sources.
- Provide a basis for funding and implementation by providing preliminary cost estimates for proposed improvements. This includes the development of detailed CIPs and costs, including design, permitting, and construction costs for 2006 to 2010. For the period 2011 to 2025, this WSMP provides recommendations for improvements in general, 5-year program "blocks."
- Prepare draft 65%, draft 95%, and final copies of the WSMP. Involve the City Council, interested community members, and Public Works Personnel in developing final recommendations and priorities for improvements to the City's water and sewer utilities.

1.3 AUTHORIZATION

The City issued a notice of award to Bristol Environmental & Engineering Services Corporation on October 7, 2003, to prepare this WSMP. Project funding authorization was provided by City Ordinance 04-08, Homer City Council Public Hearing and Second Reading March 8, 2004. ADEC provided grant/loan monies in the amount of $300,000 for developing this WSMP.

Important public meetings and City Council work sessions were held on March 28, 2005, and on January 23, 2006. The developed alternatives, general information, and proposed phasing were discussed in detail. The improvements and expansions to the water and sewer systems recommended in this WSMP reflect the consensus and direction by the City Council and concerned residents. The priorities and phasing directions also reflect input by HDPW staff.

1.4 STUDY AREA

The current City limits encompass the boundaries of the study area for this project (refer to Sheet G1). This study area includes areas currently served by the City water and sewer systems, as well as adjoining subdivisions within the City limits now served with private water wells, septic systems, and haul systems (both water and wastewater). Kachemak City was briefly evaluated for potential water service.

1.5 PLANNING PERIOD

A 20-year planning period is used for estimating future water demands and wastewater flows. Future per capita water and wastewater flows were predicted based on historical flow measurements taken by the City. Water storage and piping is sized for domestic demands and fire flows. Wastewater piping and lift stations are sized for peak wastewater flows, based upon population projections established in this WSMP, historic water usage, and estimates of I/I.

1.6 LIMITATIONS

This WSMP serves to provide guidance to the City and involved public agencies in the development of future water and sewer improvements, and management of a sustainable utility infrastructure. This WSMP cannot anticipate future changes to laws and regulations, and it cannot anticipate unforeseen changes to the City’s population and/or land uses. The City should periodically review this WSMP relative to important changes to state and federal laws, and significant changes in the community’s infrastructure and growth patterns.

1.7 NEED FOR PLANNED IMPROVEMENTS

Without the improvements proposed in this WSMP, the basic sanitation needs of Homer could be compromised and the ability to provide basic water and sewer service to parts of town not served would be limited. Capacity problems facing the current water and sewer utilities will only increase as the current infrastructure ages and the population base increases. The water improvements are needed to ensure an adequate water supply that provides safe and reliable drinking water, as well as ISO recommended fire flows for the community. The wastewater improvements are needed to ensure the public heath and reduce the potential for contamination of receiving waters and other community resources. With the implementation of the proposed sanitation improvements, many potential community health problems can be eliminated or appreciably reduced.

2.0 PROJECT PLANNING AREA

2.1 HISTORY

The City of Homer was named for Homer Pennock, a gold mining promoter who arrived in the area in 1896 and built living quarters for his crew on the Homer Spit. The operation's plans were to mine the beach sands for gold along Cook Inlet from Homer to Ninilchik. In 1899, Cook Inlet Coal Fields Company developed a coal mine at Homer's Bluff Point, a town and dock on the Homer Spit, and a 7-mile-long railroad that carried coal to the end of the Homer Spit. Various coal mining operations continued until World War I. (ADF&G, 2000)

Initial construction on the City's water system first began in 1965 and on the sewer system in 1970. A discussion of historical improvements to the water and sewer systems are presented in Sections 4 and 5, respectively.

A water system feasibility report was completed in 1971 by Clair A. Hill and Associates. CH2M Hill prepared comprehensive water and sewer plans in 1977 as well as a water reservoir study in 1980. In 1983, CH2M Hill completed comprehensive master plan update for the sewer systems, and Olympic Associates completed a Water Improvement Study for the City.

2.2 LOCATION

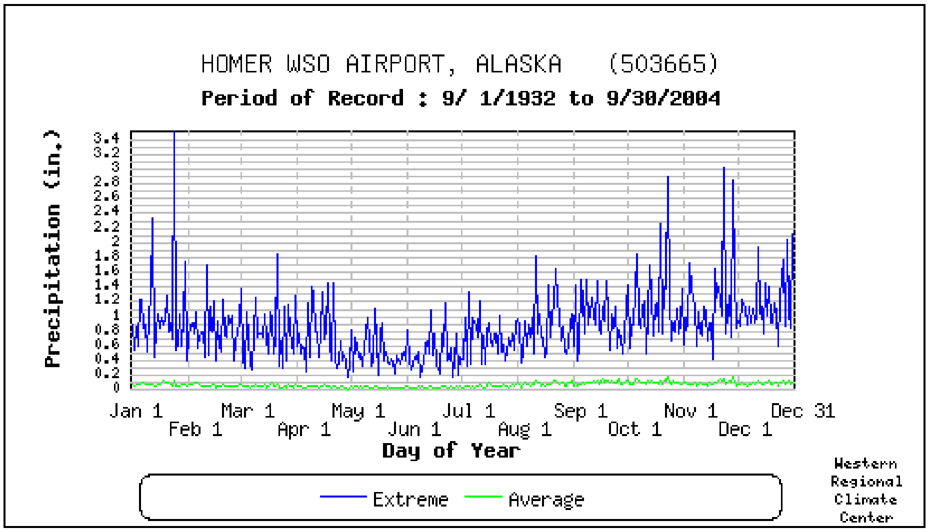
Homer is located on the northern shore of Kachemak Bay, on the southwestern end of the Kenai Peninsula. The City lies 227 miles south of Anchorage at approximately 59 degrees (°) 39 minutes (') North Latitude, -151° 33' West Longitude (Section 19, Township 006 South, Range 013 West, Seward Meridian). The City of Homer is located in the Homer Recording District. The City encompasses 25 square miles of land and 10.5 square miles of water (ADCED, 2004).

2.3 CLIMATE

Homer has a maritime climate. Average summer temperatures range from 45 to 65 degrees Fahrenheit (°F). Average winter temperatures range from 14 to 27 °F. Annual precipitation is 24 inches, with an average of 55 inches of snowfall during the winter (ADCED, 2004).

Storms can significantly influence the amount of I/I that enters the wastewater collection system. I/I in the collection system has exceeded 1 million gallons (MG) over one day. Storm events as high as 3.5 inches (January 24, 1939) have been recorded. On October 23, 2002, a 2.88-inch storm contributed approximately 1.2 MG of inflow (1.74 MG total flow) to the WWTP. A similar storm on November 23, 2002, contributed 1 MG (1.46 MG total flow). The maximum flowrates at the WWTP for both storms were recorded the day after the event. Chart 2.1 provides a rainfall summary for the Homer Airport.

Chart 2.1 Homer Airport Precipitation Data (Western Regional Climate Center)



2.4 Topography

Homer is located on a bench that is bounded by Diamond Ridge to the north, and Kachemak Bay to the south. The topography rises from sea level to (within the City limits) approximately 1,134 feet above mean sea level (MSL). A steep ridge is located between approximately 400 to 1,100 feet above MSL. From the approximately 800- to 1,100-foot elevation, the ridge rises gradually again.

The Homer Spit is approximately 4.5 miles long, northwest to southeast, and approximately 200 to 1,800 feet wide southwest to northeast. The Homer Spit is approximately 20 to 30 feet above MSL.

Planimetric mapping and photography for the city is shown on the water/sewer plan layouts (Sheets 1 through 30). This mapping and photography was obtained from Aeromap (August 10, 2003).

2.5 Geology and Soil Conditions

Homer lies within the Kenai Lowlands, which are subdivided into seven areas based on topography and geology. The Homer area is comprised of two of these geomorphological areas: Caribou Hills and Homer Bench. The portion of Homer consisting of the steep, 500- to 800-foot-high bluffs and the area atop the bluffs are part of the Caribou Hills, and the portion from the toe of the bluffs to Kachemak Bay are the Homer Bench.

The Caribou Hills are a broad, glaciated upland that rises 2,000 to 3,000 feet above Kachemak Bay and that is mantled with glacial soil deposits (chiefly glacial moraine deposits). Soils in the near surface are typically a thin layer of organic deposits overlaying mixtures of silt, sand, and gravel. Groundwater is typically found within a few to several feet of the ground surface. Peat and organic silt deposits occur locally in bogs and natural lakes. Soil conditions along the face of the bluff generally consist of a thin organic mat overlying a residual soil layer that is a few to several feet thick. Surface water and springs have eroded deep gullies in the face of the bluff; this bluff is highly erodable, especially when the surface is disturbed.

The Homer Bench is a lowland of glacial origin that was excavated by subsequent glacial advances. The surface is largely covered by loose deposits at the toe of the adjacent bluff. Further from the toe, the surface is covered by combined stream deposits formed by drainages that descend from bluff. Soil types include mixtures of silt, sand, and gravel overlying clay. Peat and organic silt deposits occur locally in bogs and naturally in-filled lakes. Groundwater is typically found within a few feet of the ground surface. The Homer Spit is a part of the Homer bench, and likely formed originally as a terminal moraine. The Spit has since been significantly reworked by marine processes.

Wetlands within the U.S. Army Corps of Engineers (USACE) jurisdiction are prevalent throughout certain areas of the City. A wetlands inventory is currently being conducted by the City Planning Department. During the development of previously planned water and sewer expansion scenarios, the City included comments on known wetland areas. The planned expansions incorporate these comments. Utility expansions were redefined in instances where the presence of wetlands could significantly impede construction, or where wetlands habitat would be adversely affected by the improvements. (Karlstrom, 1964; Quadra, 1984; ASF, 1987)

2.6 FLOOD, EROSION, AND SEISMIC HAZARDS

Lower elevations along the coast in Homer are susceptible to flooding, most dramatically from tsunamis. A 26-foot high tsunami struck Homer following the 1964 Good Friday Earthquake. Tectonic movement resulting from this dramatic event lowered the elevation of the Homer Spit approximately 4 feet. A tsunami also occurred in 1883 following the collapse of the dome of Mount Saint Augustine, a nearby island volcano in Kachemak Bay. Appendix G shows the predicted inundation areas for a 7.5 Richter Magnitude (RM) earthquake, as well as for a 9.2 RM earthquake. Flooding from storm surges may also affect low-lying coastal facilities.

The Kenai Formation, which underlies Homer, is not well hardened, and erodes on the upper bluffs in Homer where it is exposed. The coastal bluff is also subject to erosion from precipitation, frost action, wave action, and marine storms. Erosion of soils in Homer also occurs under typical erosion conditions of cohesionless soil, sloping ground surface, and surface water runoff.

Homer is located in Seismic Zone IV, where large ground motion may be expected. Structures may be damaged as a result of ground motion induced by an earthquake, or as a result of ground failure that may occur as a result of shaking.

2.7 WATER RESOURCES

There are limited developed water resources within the City's boundaries and the surrounding area. Available surface water supplies are generally small, surrounded by significant development, and/or have the potential for saline intrusion. There are no major surface water sources in the near vicinity of Homer that can be currently used as a significant source of drinking water without the construction of a surface water impoundment. Saline intrusion is not a concern for the Bridge Creek, Twitter Creek, Fritz Creek, and Diamond Creek watersheds due to the high topography of these drainage basins. Groundwater sources in the Homer area tend to be of very poor quality (high mineral content, sulfides, etc.) and low volume.

The Bridge Creek Reservoir, the City's current water supply, is approximately 35 acres in size. The volume of the reservoir is variable, depending on consumption, precipitation, temperature, etc. The reservoir is in a watershed management area, and has a moderate risk for contamination from animal waste, nearby residential development, and an adjacent roadway.

The elevation of the watershed is approximately 930 feet above MSL. The water from the reservoir is corrosive; however, the water is conditioned at the WTP with pH adjustment and inhibitors to be in full compliance with the Lead and Copper Rule.

2.8 ENDANGERED SPECIES AND CRITICAL HABITAT

Endangered and threatened species of Alaska include: the Aleutian shield fern, the short-tailed albatross, Eskimo curlew, spectacled eider, Steller's eider, leatherback sea turtle, Steller's sea lion, bowhead whale, finback whale, and humpback whale. Additionally, the northern sea otter is a proposed threatened species, and the kittlitz's murrelet is a candidate species in Alaska (USF&W, 2005). No habitat for the Aleutian shield fern, Eskimo curlew, spectacled eider, and finback whales exist near Homer or Kachemak Bay. The remaining birds and marine mammals are generally present along the shores of Homer and in Kachemak Bay. Two areas that may have high concentrations of these marine mammals are across from Homer near Halibut Cove and along the northeastern shores of Kachemak Bay.

Steller's sea lion critical habitat (rookery or haulout) is located in the Homer vicinity. A rookery is located approximately 40 miles southeast of Homer and four haulouts are located between 30 and 70 miles south and east of Homer. The rookery or haulouts would not be affected by capital improvements recommended by this WSMP. Critical habitat for Steller's sea lions is defined as within 3,000 feet inland, 3,000 feet vertically above, and 20 nautical miles seaward from each major rookery and haulout. The City of Homer is more than 20 nautical miles from the nearest rookery or haulout, and is well outside the boundary of critical Steller's sea lion habitat (NOAA, 2005).

The Anchor River and some of its tributaries (Twitter, Bridge, and South Beaver Creeks and two unnamed streams) are considered anadromous fish streams. The streams are located 3 to 10 miles west and north of Homer (ADF&G, 2005).

2.9 HISTORICAL AND ARCHAEOLOGICAL SITE CONDITIONS

All future water and sewer projects will be coordinated with the State Office of History and Archaeology in order to identify any potential impacts to historical and/or archeological sites. The State Historic Preservation Office (SHPO) must give an archaeological clearance for any state or federally funded project, which usually occurs when a project is going into final design and permitting.

2.10 Community Development Projects

Large (public) buildings that were recently constructed include a library, and a pet welfare center. A new hockey rink was recently constructed and is expected to have an average water demand of approximately 6,000 gallons per day (gpd). [equivalent to the water demand for the Tesoro Sports Centre in Anchorage (AWWU, 2004)]. Future planned development would generally be expected in the main part of town, along the Spit, and East Road, where much of the existing commercial development has already occurred. A new Fred Meyer and Town Center are currently being planned. Future contributions to the water and sewer systems are expected to derive primarily from residential construction. No major road rehabilitation or construction projects are currently planned within the City, other than those improvements for new subdivision developments.

2.11 Land Status

Lands impacted by the recommendations in this WSMP are primarily owned or controlled by the City, the Kenai Peninsula Borough (KPB), the State of Alaska, or public agencies. In some instances, future easements through private or public owned property will be required for the improvements recommended in this WSMP.

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3.0 SOCIOECONOMICS

3.1 POPULATION PROJECTIONS

Homer's population has grown from 325 in 1940 to 3,946 in 2000, according to U.S. Census data (Table 3.1). In 2002, a land annexation added an additional 419 residents to the City. The current (2006) population is estimated at 5,630 individuals. The annual growth rates (per decade) for the past 60 years vary from approximately -1.4 to 15.1 percent.

Table 3.1 City of Homer Historical Population and Average Annual Growth Rates

Year	Population	Period	Annual Growth Rate[1]
1940	325	1941-1950	-0.6
1950	307	1951-1960	15.1
1960	1,247	1961-1970	-1.4
1970	1,083	1971-1980	7.4
1980	2,209	1981-1990	5.2
1990	3,660	1991-2000	0.8
2000	3,946	not applicable	not applicable
Average Growth Rate 1941-2000			4.4
Average Growth Rate 1981-2000			3.0
Maximum Growth Rate (per decade) 1941-2000			7.4
Assumed Annual Growth 2006-2015			4.5
Assumed Annual Growth 2016-2025			3.0

Note: [1]Growth rates were calculated from U.S. Census Data provided by ADCED, 2004.

Table 3.2 presents a summary of future population projections for 2006 through 2025. It is assumed that future population growth will occur at a growth rate similar to recent historic growth rates. After evaluating the current status of private development within the City, and an expansion of the water and sewer system to residents not served, an annual growth rate of 4.5 percent was assumed for 2006 through 2015. An annual growth rate of 3.0 percent was assumed for 2016 through 2025. Based upon these projections, the population in 20 years (2025) is estimated to be 11,244 people.

The KPB has projected a growth rate for the Kenai Peninsula at approximately 2 to 3 percent through 2010 (Camp, July 2004). At a 2 percent annual growth rate, Homer would have a population of approximately 7,500 residents by 2025.

Table 3.2 Estimated Population Projections for Homer

Year	Estimated Population	Year	Estimated Population
2006	5,630	2016	8,618
2007	5,883	2017	8,876
2008	6,148	2018	9,142
2009	6,425	2019	9,417
2010	6,714	2020	9,699
2011	7,016	2021	9,990
2012	7,332	2022	10,290
2013	7,661	2023	10,598
2014	8,006	2024	10,916
2015	8,367	2025	11,244

3.2 ECONOMY/FINANCIAL PROFILE

Homer's economy is based primarily on fishing and tourism. Approximately 540 residents hold commercial fishing permits. Every summer the town is flooded with seasonal workers who maintain employment in the tourism, sport fishing, and commercial fishing industries. There is also some limited logging in the Homer area. The 1999 Homer Comprehensive Plan Update addresses the future development of Homer's economy.

3.3 TRANSPORTATION

Homer can be reached by air, sea, and ground based transportation. The City has an airport, seaplane base, and a heliport. The airport consists of a 6,700-foot long by 150-foot wide asphalt runway. Regular flights are available from Anchorage and Kenai. There is also regular small plane service to surrounding communities in Kachemak Bay. The City has a small boat harbor and docking facility that is large enough to accommodate ferries and cruise ships. Homer is on the Alaska Marine Highway System (ADCED, 2004).

3.4 LAND USE

The 2000 U.S. Census Sheets reported 1,873 housing units within the City. Of these, 78 percent were occupied year-round, 7 percent were vacant because of seasonal use, and 15 percent were vacant all year. The average household size was 2.4 individuals (ADCED, 2004).

The developed land in Homer is classified as being used for commercial, industrial, or residential purposes. Industrial facilities include fish processing, marine support, shipping, and timber processing. A specific categorization of commercial, residential, or industrial uses by acreage or density is not currently available. KPB and City mapping were used to estimate existing and future land use patterns. Existing property and subdivision boundaries are detailed in the City water and sewer layouts in Sheets 1 through 30.

The Homer Planning Department has a zoning map (2004) of the City, which is provided in Appendix H. The airport and related facilities are owned and maintained by the Alaska Department of Transportation and Public Facilities (ADOT&PF) and the City. Public schools in Homer are operated by the Kenai Peninsula School District, and include Homer High School (456 students), Homer Middle School (228 students), Paul Banks Elementary School (222 students), and West Homer Elementary School (296 students). Three small charter schools also operate within the City.

3.5 UTILITIES

Homer's drinking water is derived from a reservoir at Bridge Creek, which was built between 1974 and 1976. This surface water supply is the community's sole source of drinking water; there is no backup water supply. Water is treated, stored in tanks (total storage capacity is approximately 2,650,000 gallons, including the "A" frame and Homer Spit water tanks), and distributed. The sewer system is a combination of gravity and force mains; Kachemak City has a variable grade effluent system (VGES) which is connected to the Homer collection system. The WWTP has a maximum average flow capacity of 880,000 gpd. Approximately 64 percent of the occupied homes in the City are served by the water system. Approximately 53 percent of the occupied homes are served by the sewer system. The remaining homeowners haul water or have wells, and use on-site sewer systems.

Homer Electric Association (HEA) operates the Bradley Lake Hydroelectric Plant at the end of Kachemak Bay. HEA is a part-owner of the Alaska Electric Generation and Transmission Cooperative, which operates a gas turbine plant in Soldotna. HEA also purchases electricity from Chugach Electric. The utility currently charges residential customers a base charge of $11, plus $0.101 to $0.111 per kilowatt-hour (usage dependent). Commercial and industrial users are charged separate rates. Telephone service is provided by Alaska Communications Systems.

The community's current water and sewer rates (July 2004 City of Homer Fee Schedule) are provided in Appendix I.

3.6 POLITICAL JURISDICTION

Homer is a first-class city, and was incorporated on March 31, 1964. The City government consists of an elected mayor and a six-member City Council that provides policy direction. City operations are the responsibility of the City Manager. Water and sewer facilities are the responsibility of the HDPW. Other departments/divisions include the: Police, Fire Department, Personnel, Library, Planning, Port and Harbor, Finance, and City Clerk.

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4.0 EXISTING WATER SYSTEM

4.1 SUPPLY FACILITIES FOR THE WTP

Homer's water system (water system ID AK2240456) is classified as a Class A public water system. Homer's drinking water is supplied by a reservoir at Bridge Creek, which is the sole source of the community's public drinking water supply. The watershed area is not fenced and is accessible by road. The Bridge Creek Reservoir is approximately 930 feet above MSL and is approximately 35 acres in size. The dam was constructed from 1974 to 1976 using native silty clay and coarse sandy silt. The dam is 24 feet high and 260 feet wide. The reservoir has an average depth of about 10 feet, and a maximum storage capacity of approximately 450 acre feet (or approximately 150 MG).

The Bridge Creek Reservoir raw water pump station was built between 1973 and 1974, and was expanded in 1983. The pump station has three 75-horsepower (hp) pumps that have a capacity of 700 gallons per minute (gpm) each. Raw water from the pump house travels through dual 8-inch cast iron (CI) and 10-inch ductile iron (DI) transmission mains to the WTP. The pump station is in relatively good condition, and no significant improvements or modifications are anticipated at present.

The water supply is potentially susceptible to contamination from several sources, though such contamination is not currently thought to be a problem. Potential pollutant sources include residential septic and fuel tanks, roads, and vehicles. The intake for the pump house is at the northwestern end of the reservoir. The intake level can be adjusted (i.e., suction line for the raw water pumps). The reservoir surface elevation can drop several feet below the normal overflow level during the summer.

The City retains the water rights for the surface water, with a total allowed capacity of 900,000 gpd (Alaska Department of Natural Resources [ADNR] permit LAS 18902 – refer to Appendix J). This permit expires on December 1, 2006, and should be renewed. The City should apply for expanded capacity at renewal, because summer average flowrates are estimated to approach 935,000 gpd by 2009. By 2013, the average yearly demand will approach the current 900,000 gpd allowance (See Section 6.2). There are no groundwater supply wells or other supply facilities that are currently operated by the City.

The City established the Bridge Creek Watershed Protection District under Chapter 21.59 of the City Code. Zoning under this regulation restricts development in the Bridge Creek Watershed. Restrictions include a minimum lot size of 5 acres, and a maximum impervious surface area of 7 percent on new lots. Part of the watershed is located outside of City boundaries.

The supply lines from the Bridge Creek pump station to the WTP require periodic "pigging" to remove interior build-up of solids and algae. The lines were last pigged in 2004. The pig has been jammed in the past due to a buildup of organic material and soil in the inner line. The City typically schedules pigging the line when flows from the Bridge Creek pump station drop significantly.

4.2 WATER USAGE

4.2.1 Water User Classifications

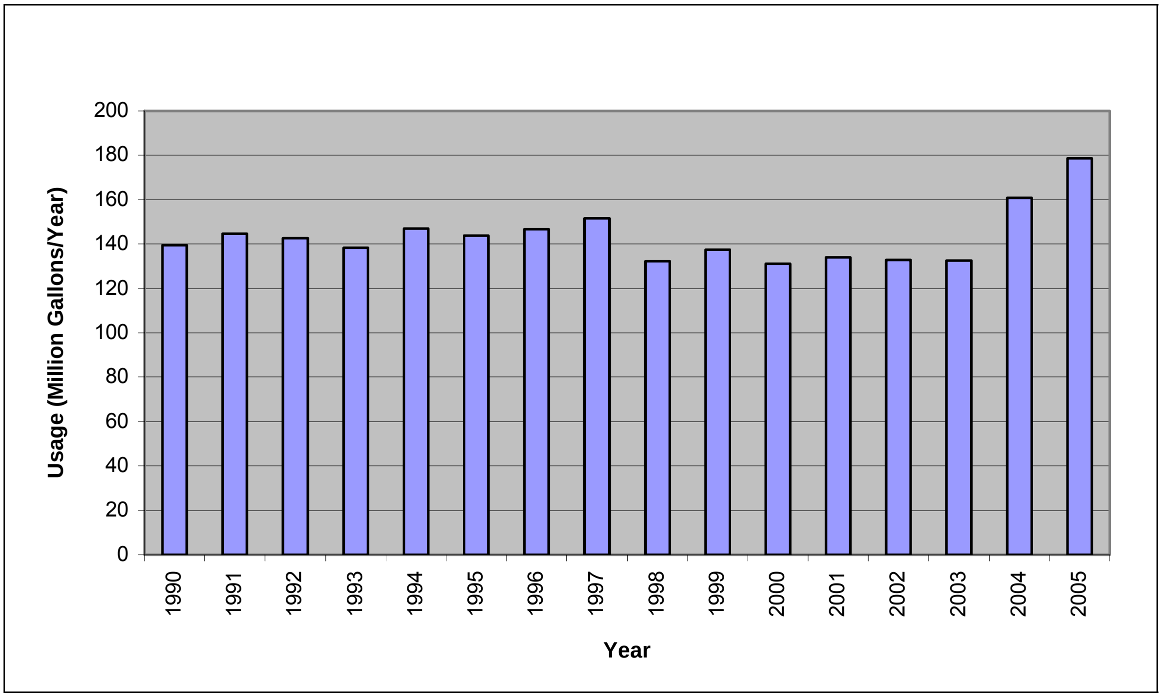
Water service in the Homer area is classified into five separate categories: 1) residential, 2) commercial, 3) industrial, 4) bulk water, or 5) vessel service. The majority of the use is for residential and commercial purposes. Water service for vessels is provided at the small boat harbor and two municipal docks.

4.2.2 Water Usage

Approximately 1,430 customers are currently served by the water distribution system. The per capita usage rate for the City is estimated at approximately 82 gallons per capita per day (gpcd). The per capita flowrate contribution of the wastewater system does not mirror the water system per capita demand. In many communities, wastewater flow can typically be as little as 40 percent of the average water consumption. This usage rate includes water usage attributed to industrial, commercial, and marine demands, but does not include water haulers, leakage, or bleeding. The average production at the WTP for 2005 ranged from approximately 0.26 to 1.9 MG per day. In 2005, the average annual production was approximately 540,000 gpd. The average summer production during 2005 averaged 700,000 gpd, and winter production was approximately 471,000 gpd.

Since the population fluctuates fairly significantly from winter to the summer, major seasonal demand changes do occur in the water distribution and sewer collection systems. Water demand essentially doubles during the summer (June to August), compared to demands during the height of winter (December and January). Chart 4.1 shows the average yearly metered water usage rates from 1990 through 2005. Chart 4.2 shows these rates as an overall percentage of annual water use by month from 1990 through 2005. Homer's (former) largest fish processing plant, Icicle Sea Foods, was destroyed by a fire in 1998; the closure of the plant and a substantial increase in metered fees accounts for the usage drop after 1997. The average water consumption for 2004 and 2005 was significantly higher than for prior years.

Chart 4.1 Water Consumption[1] by Year (1990-2005)



Notes:
[1]Total amount consumed by water customers, as measured by individual water meters.

Chart 4.2 Average Percentage of Yearly Water Usage by Month (1990-2005)

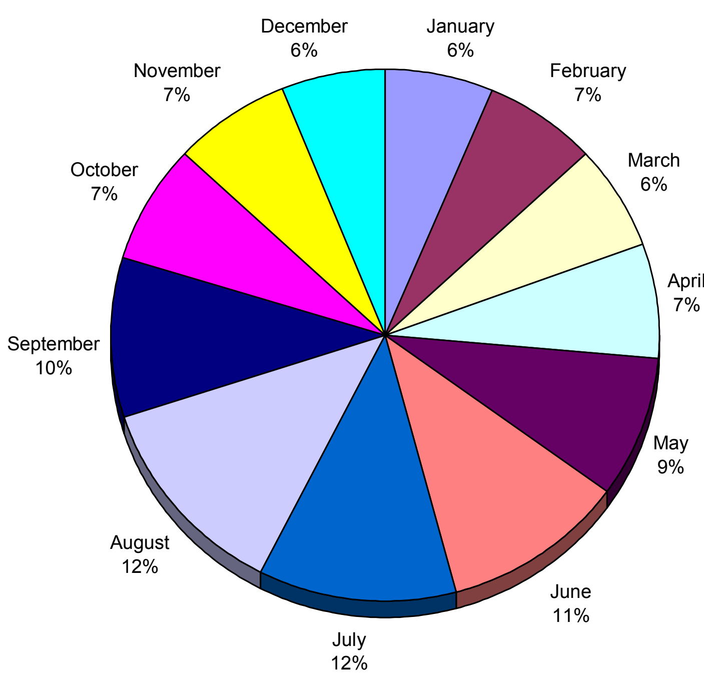
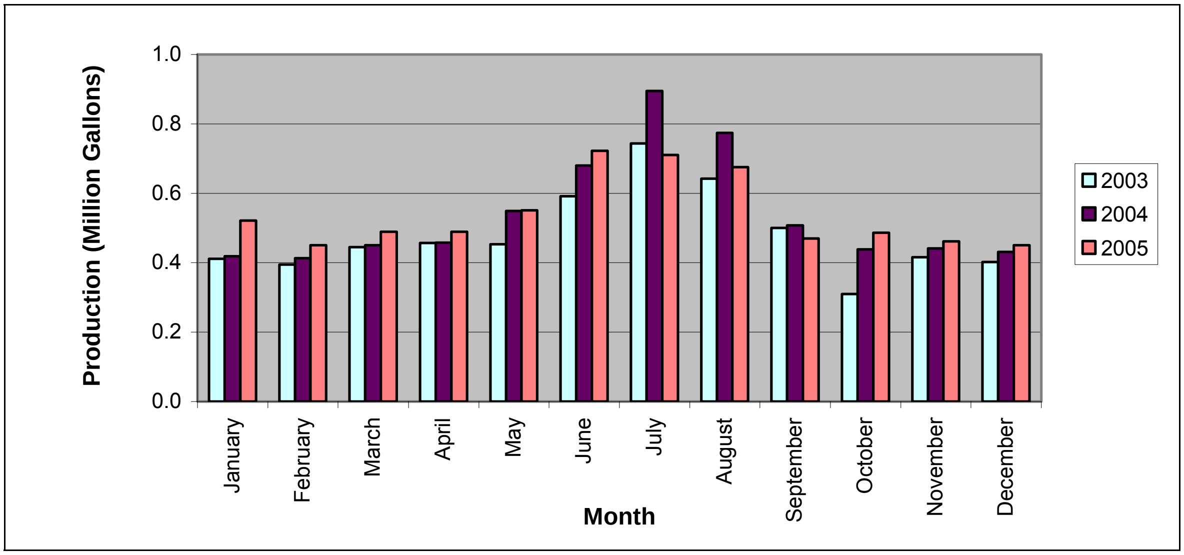


Chart 4.3 shows the average total water production for 2003 through 2005 (per month).

Chart 4.3 Water Production[1] per Month (2003-2005)

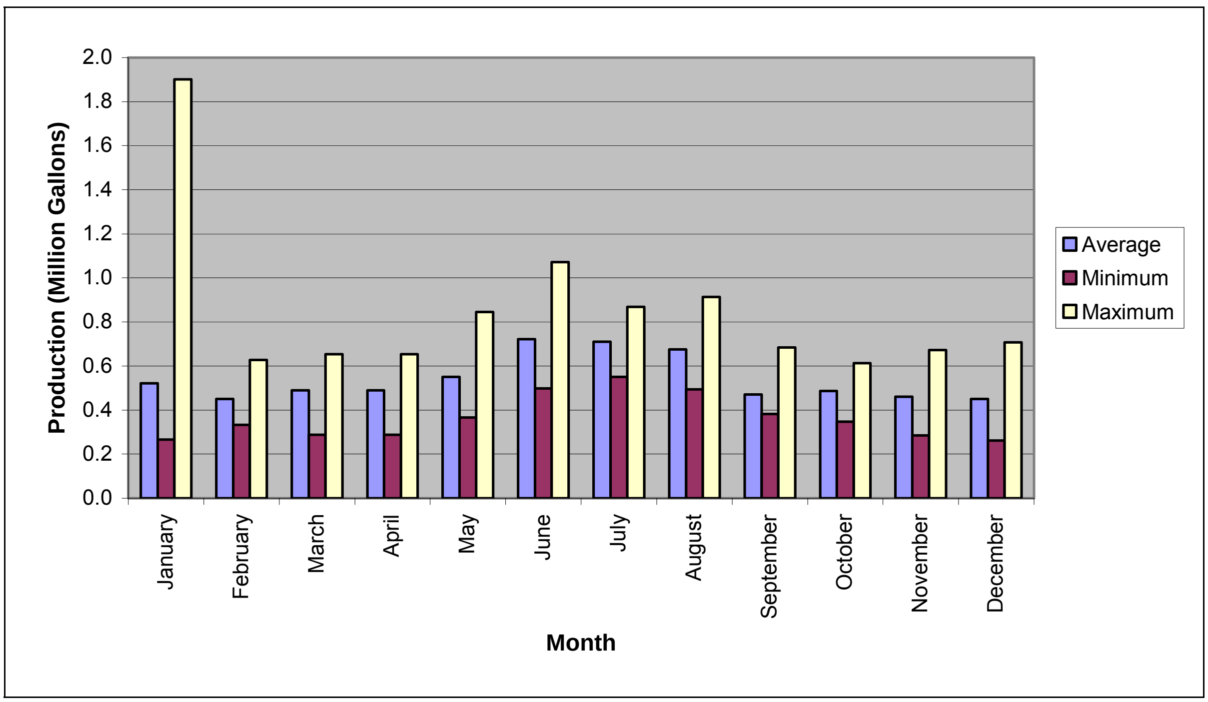


Notes:

[1]Metered amount of water *produced* at the WTP. Note that metered *consumption* values will differ because of water loss due to leakage and bleeding.

Average, maximum, and minimum monthly production records for the WTP for 2005 are presented on Chart 4.4.

Chart 4.4 Daily Water Production Rates by Month (2005)



It is not known if the production spike for January 2005 is a meter error, data error, or some other anomaly. The maximum daily production for 2003 and 2004 was 1,110,000 and 1,070,000 gpd, respectively.

Generally, demand is highest from June through September, with July and August being the months with the highest demands (Chart 4.3). Water production for summer 2005 averaged 21.5 MG per month, or approximately 700,000 gpd (Chart 4.4). Approximately 30 percent of the flow through the WTP is unaccounted for from the meter summaries. The majority of this flow is believed to be lost through "bleeding" of dead end lines at the end of East Road and the Homer Spit. The City bleeds these lines to maintain chlorine residual at dead ends and to prevent stagnation in the Spit Reservoir. There are few large demand water users at the end of the Spit (near the Spit Reservoir).

The WST at the end of the Homer Spit (Spit Reservoir) was built to provide fire protection to businesses and facilities on the Spit. The pumps at the Spit Reservoir can be manually activated during fire emergencies and high demands. The fish dock (at the end of the spit) is equipped with cold storage facilities, an ice manufacturing system, and a vacuum fish-loading system.

There are no known major water main leaks. Water mains are regularly checked and maintained for leaks, using flowrate comparisons against meter readings. Leak testers have been hired by the City in the past to look for leaks on the Spit.

4.3 WATER TREATMENT PLANT (WTP) / WATER QUALITY

Refer to Appendix A for a detailed description of the WTP and the quality of the water supply.

Disinfection compliance is not a problem for the Homer water system. The City recently installed a new chlorination disinfection system. Residuals are tested and monitored regularly. The City maintains a watershed protection program for the Bridge Creek watershed, and complies with current levels of concentration, and time of contact, or "CT" requirements of the Surface Water Treatment Rule. The City periodically bleeds lines at the end of East Road and the Spit to maintain disinfection residual. Backwash requirements for the WTP, bleeding to maintain disinfection residual, and to a lesser extent main leakage accounts for a substantial percentage of the usage for water generated at the WTP. The average residual leaving the storage tanks at the WTP is 0.6 to 0.8 milligrams per liter.

4.4 WATER STORAGE

The City has approximately 2,650,000 gallons of water storage available in five WSTs. Three of the WSTs are located near the WTP: (CT Tank, Clearwell / Upper Reservoir (also called the "wood stave tank"), and 1-MG Reservoir. The "A" frame tank is the original WST that was constructed for the 1965/1966 water distribution system, and is located above the hospital in the west area of town. The Spit Reservoir is used to provide emergency fire protection and additional storage for the Homer Spit. A summary of the City WSTs is presented in Table 4.1.

Table 4.1 Water Storage Tanks

Tank	Location	Nominal Volume (gallons)	Estimated Available Volume (gallons)	Type	Floor Elevation (feet)[1]	Stored Water Height (feet)	Diameter (feet)	Year Built
"A" Frame Reservoir	Near "A" Frame PRV	250,000	250,000	Steel	362	35	35	1966
Clearwell/ Upper Reservoir	Water Treatment Plant	500,000	500,000[2]	Wood Stave	1,098	35	62	1974-1976
Spit Reservoir	End of Spit	850,000[2]	750,000	Steel	25	28[2]	72[2]	1980/ 1981
CT Tank	Water Treatment Plant	150,000	150,000	Welded Steel	1,098	24.5	36	1998
1-MG Tank	Water Treatment Plant	1,000,000	1,000,000[3]	Welded Steel	1,098	21	98	2003

Notes:
[1]Floor elevations provided by the Homer Department of Public Works.
[2] Estimated or assumed value
[3]Volume available at overflow
MG = million gallon
CT = chlorine concentration and time of detention
PRV = pressure relief valve

The WSTs serve as reservoirs for water demands and fire emergencies, as well as providing contact time for the chlorine. Baffling is provided in the CT Tank to provide additional contact time for disinfection. No significant freezing problems have been reported at any of the WSTs.

The City tries to operate with the WSTs continuously full; however, water demands can drop capacities below maximum storage limits. The lowest height that the 1-MG Tank and Clearwell/Upper Reservoir have dropped to (since the installation of the 1-MG Tank) is approximately 14 feet above the floor of the 1-MG Tank. This dropped the estimated storage at the three WSTs near the WTP to an estimated 1.1 MG (out of 1.65 MG available). The CT Tank is always full.

The WSTs are currently visually inspected annually by the City crews, and cleaned on as as-needed basis; the stringent filtration system at the WTP removes virtually all sediment that would typically accumulate in the tanks. The last formal inspection was in 2000, when the last cleaning was performed. The current integrity of the WST interiors is unknown. It is recommended that all the WSTs undergo comprehensive cleanings and inspections at least once every 5 years. This includes draining, cleaning, and inspecting each WST, followed by disinfecting (before filling). These inspections should include assessing coating and/or corrosion problems, structural deficiencies, integrity of anodes, and any noted water quality problems. Any coating failures due to ice formation should also be noted and corrected. Inspections should be performed by a qualified engineer or a National Association of Corrosion Engineers-qualified coatings inspector.

If problems with the paint coatings are noted in future inspections, the interior of the WST should be recoated with an American Water Works Association (AWWA)-approved potable water coating system. The functionality of any cathodic protection systems in the WSTs should also be tested and verified, and the systems replaced if necessary. Incremental repairs have been made to the coating systems in recent years. The Spit Reservoir was cleaned and recoated in 2005.

The Spit Reservoir has a booster pump station, which was also constructed in 1983. The pump station is equipped with a 3,200-gpm fire demand pump, and a 700-gpm domestic/industrial demand pump. Water flows continuously through the Spit Reservoir to a bleeder line at the end of the Spit in order to reduce the age of the water in the tank and maintain disinfection residual.

In the event of a failure at the Bridge Creek Pump Station, the City currently has sufficient volume for approximately 2 to 5 days of storage depending on the time of year. This assumes a maximum daily demand of 1,200,000 gpd, and an average winter daily demand of approximately 500,000 gpd. If a worst-case fire demand were to occur during this time period (3,500 gpm, 3-hour fire flow = 630,000 gallons), the City would have approximately 1.6 to 4 days of storage. On average, the City has approximately 5 days of reserve storage during the summer, or approximately 4 days if a worst-case fire demand occurs.

4.5 PIPELINES

There are approximately 40 miles of pipe in the water distribution system, not including service piping and abandoned mains. Water distribution pipelines in Homer consist of CI, DI, polyvinyl chloride (PVC), and high-density polyethylene (HDPE) pipe, with main sizes ranging from 4- to 16-inch inner diameter (ID). An inventory of the known active water distribution mains in Homer is presented in Table 4.2. This inventory does not include service connections and pipes immediately associated with the WTP and WSTs. Existing water distribution lines are depicted on Sheets 1 through 15.

Table 4.2 Estimated Linear Feet of Active Water Main

Pipe Material	Linear Feet of Water Main[1]					
	4-inch	6-inch	8-inch	10-inch	12-inch	16-inch
Cast Iron	-	4,500	14,000	29,700	-	-
Ductile Iron	1,200	18,300	41,200	16,900	9,000	300
HDPE	-	2,300	10,400	5,300	11,500	5,500
PVC	800	8,900	10,000	5,000	16,800	-

Note: [1]Does not include service connections.

HDPE = high-density polyethylene
PVC = polyvinyl chloride

Table 4.3 provides an estimated *typical* lifespan of water and sewer piping materials. Although numerous studies have been conducted to determine pipe life expectancies, the actual lifespan will vary depending on soil conditions, installation methods, seismic conditions, etc.

Table 4.3 Piping Material Life Expectancies

Piping Material	Estimated Installation Date (Mains)	Estimated Life Expectancy (years)
Asbestos Cement	1964 – 1975	20 – 30
Cast Iron	1964 – 1975	50+
Ductile Iron	1975 – 1998	50+
Polyvinyl Chloride	1970 – 1990	30+
High-density polyethylene	1992 – present	50+

References:
1) Water Distribution Systems Handbook, Larry Mays, McGraw Hill, 2000
2) Foundation for Water Research, Dec 1988: http://www.fwr.org/pipeline/dwi0131.htm

Based on meter readings, an average of approximately 69 percent of the flow to town is through the East Hill Road Main; 31-percent is through the West Trunk Main. The 8-inch CI West Trunk Main serving the "A-frame" reservoir and vicinity allows for a maximum of approximately 1,360 gpm with the current pressure control valves at the Hilltop, Midhill, and "A-frame" PRV stations. Flows above 1,300 gpm, especially running through the 4-inch PRV at the Hilltop station create excessive velocities in the line. With the new Midhill and Hilltop PRV modifications, this line will be restricted to a maximum of 1,300 gpm during high demand flow conditions. The Ridgeline PRV station currently has problems believed to be due to air pockets that form upstream of the station during high demand conditions.

Water lines in the Homer area are buried a minimum of 7 feet below grade. Piping is typically installed using native bedding material in conformance with the City of Homer Standard Construction Specifications. The use of native material as backfill typically prevents widespread groundwater flow following buried piping. Dead-end lines are typically flushed through hydrants on an annual basis to remove sediment.

The following is a general summary of major water improvements completed within the City:

- 1965 and 1966 – the original 8-inch transmission line to the A-frame PRV station (former WTP) and A-frame WST was constructed.
- 1966 – service was provided to Main Street, and included extensions along Bayview Avenue, Hohe Street, Fairview Avenue, Bartlett Street, and Pioneer Avenue.
- 1967 – a 10-inch line was extended on Homer East Road to the Homer Elementary School.
- 1968 – a 10-inch transmission line was constructed from Pioneer Avenue to the Homer Spit.
- 1974 to 1976 – Additions to the water distribution system, including extensions along East Hill Road, East Road, Main Street, Bayview Avenue, and Kachemak Way.
- 1975 – a 12-inch PVC main was constructed to the airport.
- 1974 – The Bridge Creek Reservoir Dam was expanded.
- 1983 – The Spit Reservoir and pumphouse were constructed.
- 1998 – The CT Tank was constructed.
- 2003 – The 1-MG Tank was constructed.
- 1980s to present – substantial system expansions provided under the Accelerated Water and Sewer Program.

Significant exterior corrosion has occurred in the past on CI lines installed at the Spit. The City has replaced the majority of the CI piping on the Spit with HDPE piping, with the exception of approximately 4,800 linear feet (LF) of CI piping at the end of the Spit (on Homer Spit Road, Fish Dock Road, and Ice Dock Road). The condition of the remaining CI piping on the Spit is unknown.

4.6 PRESSURE REDUCING VALVES

Homer has a total of 19 operating PRVs within the water distribution system, excluding those PRVs that were installed to provide service to the boat harbor. The locations of these PRV stations are shown on Sheet 15. Three new PRV stations were installed in 2005. PR18 (Bartlett Street), PR25 (Barcus), and PR26 (Sterling Highway). A summary of the PRV stations is presented in Table 4.4.

Table 4.4 Existing PRV Stations

PRV Name	PRV Location	Elevation (feet)[1]	Valve Sizes (inches)	Inlet Setting (psi)	Outlet Settings (psi)[2]
PR01-Hilltop [old]	Hilltop [old]	799	2, 4	75	25
PR02-Midhill [old]	Midhill [old]	576	2, 8	112	21
PR03-"A"-frame	Dehel Street	369	2, 6	112	49
PR04-Efflers	Diamond Willow Circle	912	1-½, 3, 6	86	25
PR05-Switchback	East Hill Road	717	1-½, 3, 6	96	26
PR06-Barnett	East Hill Road	430	1-½, 3, 8	145	45
PR07-Hoedel's	East Hill Road	299	1-½, 3, 6	101	21
PR08-Kachemak	Kachemak Way	263	2, 6	97	36
PR09-Main/Danview	Main and Danview	266	2, 6	60	34
PR10-Bus Garage	Ohlson Lane	81	2, 6	115	54
PR11-Jeep Sales	Main Street	134	unknown	94	33
PR12-Lucky Shot	Lucky Shot Street	105	2, 4	104	44
PR13-HEA	Lake Street	115	1-½, 3, 6	97	36
PR14-Lakeside	Ben Walters Lane	112	2, 6	95	41
PR15-Bear Creek	Early Spring Street	94	2, 6	110	50
PR21-Ridgeline	Ridgeline Off Fireweed	851	1-½, 3, 6	113	50
PR18-Bartlett	Bartlett Street	219	6, 2	110	55
PR25-Barcus	Eric Lane / West Hill Road intersection	279	8, 3, 1-½ TBD	87	30
PR26-Sterling Highway	Sterling Highway, between Watson Place and Saltwater Drive	280	8, 3, 1-½ TBD	84	26

Notes: [1]Elevation of inlet pipe to PRV station, based upon survey datum (NAD 83/88) provided by the City of Homer Department of Public Works.

[2]PRV outlet pressures are flow dependent. Outlet settings for individual valves are shown where verified.

HEA = Homer Electric Association

PRV = pressure reducing valve

psi = pounds per square inch

The Hilltop (PR01) and Midhill (PR02) PRV stations are currently scheduled for replacement in 2006. A summary of the proposed new PRV stations is presented in Table 4.5.

Table 4.5 New PRV Stations (2006)

PRV Name	PRV Location	Approximate Elevation (feet)	Expected Valve Sizes (inches)	Expected Inlet Setting (psi)	Expected Outlet Setting (psi)
PR01-Hilltop	Hilltop (Replacement)	790	8, 3, 1-½	76	20
PR02-Midhill	Midhill (Replacement)	566	8, 3, 1-½	117	21

Notes:

psi = pounds per square inch
PRV = pressure reducing valve
CT = chlorine concentration and time of detention
TBD = to be determined

4.7 SERVICE PRESSURES / FLOWRATES

There are 16 pressure zones within the current Homer water distribution system. Approximate static pressure ranges for each zone are summarized in Table 4.6. Exit pressures from individual PRVs can fluctuate by as much as 5 to 10 pounds per square inch (psi) depending on the flow through the PRV.

The proposed "expanded" water system (year 2025 estimate) is shown on Sheets 1 through 15. A pressure zone map of the water system (prior to the installation of PRV stations PR18, PR25, and PR26) is shown on Sheet 36.

Table 4.6 Current Water System Pressure Zones

Pressure Zone	Upstream PRV Station(s)/Facility	Downstream PRV Station(s)/Facility	Approximate Maximum Static Pressure (psi)	Approximate Minimum Static Pressure (psi)
1	Water Treament Plant	PR21-Ridgeline PR04-Efflers	113	5
2	PR01-Hilltop	PR02-Midhill	112	25
3	PR02-Midhill	PR03-“A”-frame	112	21
4	PR03-“A”-frame	PR08-Kachemak PR09-Main/Danview “A”-frame Tank PR-18 Bartlett PR-25 Barcus PR26-Sterling Hwy	110	47
5	“A”-frame Tank	PR09-Main/Danview	56	22
6	PR04-Efflers	PR05-Switchback	96	25
7	PR05-Switchback	PR06-Barnett	145	26
8	PR06-Barnett	PR07-Hoedel’s	101	45
9	PR07-Hoedel’s PR08-Kachemak PR09-Main/Danview PR-18 Bartlett PR-25 Barcus PR26-Sterling Hwy	PR10-Bus Garage PR11-Jeep Sales PR12-Lucky Shot PR13-HEA PR14-Lakeside	135	21
10	PR10-Bus Garage	-	76	54
11	PR12-Lucky Shot	-	80	37
12	PR13-HEA	-	70	26
13	PR14-Lakeside	-	63	38
14	PR15-Bear Creek	-	60	50
15	PR21-Ridgeline	PR01-Hilltop	75	50
16	PR11-Jeep Sales	-	56	33

Notes:

HEA = Homer Electric Association
PRV = pressure reducing valve
psi = pounds per square inch

The replacements of the Hilltop and Midhill PRVs will provide for the pressure zones described in Table 4.7 (Zones 2 and 3).

Table 4.7 Proposed New Water System Pressure Zones

Pressure Zone	Upstream PRV Station(s)/Facility	Downstream PRV Station(s)/Facility	Approximate Maximum Static Pressure (psi)	Approximate Minimum Static Pressure (psi)
2	Hilltop (new)	Midhill (new)	117	20
3	Midhill (new)	A-frame [PRV]	106	21

Notes:

HEA = Homer Electric Association
PRV = pressure reducing valve
psi = pounds per square inch

The downstream pressure zones for five existing (PR10, PR11, PR12, PR13, and PR14) PRV stations and one additional planned PRV station (East End Road) are expected to be combined as one pressure zone.

Existing meter vaults and smaller PRV stations serving the ferry terminal and boat harbor are presented in Table 4.8. The City has listed these stations under the same naming designation as the main line PRV stations.

Table 4.8 Meter Vaults and Harbor PRV Stations

Vault Name	Location	Station Type
PR16 -TS	Ocean Drive (Meter Vault For Spit Flows)	Meter Vault
PR17-Fish Dock	Fish Dock Road	Meter Vault
PR19 -Harbor	Harbor-Ramp 4, 4166 Homer Spit Road	PRV
PR20- Harbor	Harbor-Ramp 2, by Harbor Office	PRV
PR22- Harbor	Harbor-Ramp 6, 30 Acres	PRV
PR23-Harbor	Ferry Terminal Parking Lot	PRV
PR24-East End Road	East Hill Road and East End Road	Meter Vault

Note: PRV = Pressure Reducing Valve

4.8 FIRE PROTECTION

4.8.1 Fire Protection Requirements

A primary criterion for the sizing and placement of water distribution systems is fire protection. The following minimum criteria were incorporated into the design of the Homer water distribution system:

- The system will provide a minimum fire flow protection of 1,000 gpm sustainable for

2 hours for residential and commercial lots. The following fire flow criteria were established for other areas in the distribution system:

- Large facilities such as the schools, hospital, airport, and visitor center: 3,500 gpm for 3 hours
- Boat Harbor: 2,000 gpm for 3 hours (the Spit pump station provides additional pressure)

- The minimum static pressure within the system will be 30 psi. The maximum static pressure will be 80 psi. Areas with service connections greater than 80 psi will need individual PRV stations installed at service connections. The minimum residual pressure during a fire will be 20 psi throughout the system.
- Minimum water storage requirements are 630,000 gallons (fire demand of 3,500 gpm for 3 hours) + reserve storage for 3 days of average domestic demand during the summer, currently estimated at 0.80 MG per day (a total of 3.0 MG for 3 days of reserve storage, plus a worst-case fire flow). By 2025, the average July domestic demand will be 1.86 MG per day (a total of 6.2 MG for 3 days of reserve storage, plus a worst-case fire flow).

Refer to Section 4.8.3 for information regarding spacing requirements for fire hydrants, and Section 7.5 for proposed additional hydrant installations.

Fire flows generally conform to National Fire Protection Agency minimum recommended fire flow requirements. Water storage volumes could decrease substantially after a fire, and cause an emergency shortage if a very large, abnormal fire emergency were to occur. It is recommended that the available storage volume be increased. Refer to Section 9.2.3.

In the event of a transmission line breakage, sufficient storage is available to supply the City with water for approximately 3.3 days of average summer demands and 2.1 days of maximum summer demands, assuming the WSTs are full to begin with. Available storage would, of course, be dramatically reduced if a fire occurred during this type of emergency.

4.8.2 Fire Department

Homer maintains a combination paid/volunteer fire department with approximately 24 fire fighters, and 16 support staff. Approximately seven employees are full time. The Fire Department personnel are state certified at the Firefighter I, II, or Fire Officer I levels. The Fire Department is accredited by the State of Alaska to instruct at the Firefighter I and II levels. The department maintains two tanker trucks, two pumper trucks, one rescue truck, two medic trucks, two utility trucks, and one brush truck. Firefighting is severely hampered in areas outside the water distribution system.

4.8.3 Hydrants

There are currently 298 fire hydrants connected to the City water distribution system. All hydrants have a 5.25-inch main valve, with a pumper port and two hose outlets. Hydrants are standard AWWA 502, dry-barrel units as manufactured by Waterous, Mueller, or Darling.

Hydrants are labeled with a unique identification number that is permanently attached to each hydrant. All hydrants are currently in service, and there are no known operational or maintenance problems. The City flushes hydrants annually to remove sediments from hydrants and dead end lines, and maintains a stock of repair parts.

The recommended maximum spacing between hydrants is 500 feet, with a maximum distance (hose length) between hydrants and service points of 250 feet. For large public buildings such as the schools and the visitor center, four hydrants are recommended with a maximum spacing of no more than 350 feet. For the boat harbor, hydrants should have a maximum spacing of no more than 450 feet. Recommended spacing requirements for hydrants are identified in Table 4.9.

Table 4.9 Recommended Spacing Requirements for Hydrants

Fire Flow Requirements (gpm)	Minimum No. of Hydrants	Average Spacing Between Hydrants (feet)	Maximum Distance from Hydrant to Any Point on Street or Road Frontage (feet)
1,000	1	500	250
2,000	2	450	225
3,500	4	350	210

Notes: Source: Anchorage Water & Wastewater Utility, 1994, 60.06.03
gpm = gallons per minute

For locations where hydrants are not needed for the protection of buildings, hydrants should be spaced at least every 1,000 feet to provide for vehicle fires and future growth. Hydrants should be connected to mains that are a minimum of 6 inches in diameter.

Hydrant spacing in Homer is generally sufficient to provide coverage to most areas of the future distribution system, but there are areas where hydrant coverage is lacking. Additional locations where hydrants are recommended for installation are described in Section 7.5 and shown on Sheet 38.

4.8.4 Airport Fire Protection

ADOT&PF provides fire protection equipment for the aircraft and the runway using the crash/rescue station located on Kachemak Drive. The airport terminal building is equipped with a wet and dry sprinkler system.

4.8.5 ISO Rating

The last Insurance Services Office (ISO) rating for the City was conducted in July 1996. An ISO fire protection Class-3 rating is provided in areas served by hydrants, and a protection Class-8 and higher is provided in the surrounding areas. A lower ISO rating generally signifies lower fire insurance rates for the City.

4.9 SCADA SYSTEM

The water system has a radio- and telephone-based Supervisory Control and Data Acquisition (SCADA) telemetry system. The City uses this system to monitor and control operations at the WTP, WWTP, raw water pump station, Bridge Creek dam, Spit booster pump station, A-Frame tank, A-Frame PRV, PR09 (Main and Danview PRV), PR13 (HEA PRV), PR16 (Ocean Drive meter), and PR24 (East End Road meter). The City is planning to convert to a total radio-based system in the future, and to include all PRV stations. The system was manufactured by Seamons, initially installed in 1983 by Stead and Bagley, and has been continuously upgraded.

4.10 ISOLATION VALVES

Approximately 500 main line isolation valves are located throughout the water distribution system. The valves are typically gate valves for smaller diameter mains (12-inch or smaller), and butterfly valves for larger diameter (16-inch or larger) mains. The City reports no known inoperable valves, and has a program to regularly "exercise" valves to check for proper operation.

4.11 SERVICE CONNECTIONS

There are approximately 1,430 service connections on the City water system. A total of 1,031 connections are residential. Existing service connections are copper piping (last 20 years) or galvanized steel (older). The galvanized steel service connections are gradually being replaced. Metering is performed at virtually every service connection. Freeze-ups are generally not a problem at water services between the mains and curb box. In general, there are no known problems with the water services or meters. The City installs service connections from the water main to the property line.

City water services from the dwelling to the curb stop are the responsibility of the property owner. Thaw wires are required for new service connections. If service lines do not have a minimum 7-foot depth of burial, they are installed with board insulation in the trench.

Fees for the various categories of service connections are described in detail in Appendix I. There is an ordinance mandating that structures must be connected to the water system. The City requires that service connections have individual PRVs installed where service pressures exceed 70 psi.

4.12 CROSS CONTAMINATION

Facilities with the potential to contaminate water supplies through backflow are required under 18 Alaska Administrative Code, Part 80.025 to install (reduced-pressure) backflow prevention devices on their water service. Such devices should be evaluated and installed in a manner that would not cause a fire hazard due to a loss of pressure from the backflow device (for example, loss of pressure in sprinkler systems in a building). Backflow prevention devices are installed at all commercial service connections in the Homer water system.

There are no known cross contamination problems for the City water system. The City requires, but does not regularly check, backflow prevention devices at water service connections. Regular (annual) inspections of backflow prevention devices at commercial services may be a future requirement the City may wish to pursue.

4.13 METERS

The City uses monthly meter readings to bill essentially all residential, commercial, and industrial users, as well as public facilities such as schools and the hospital. Meter readings are also taken at: the WTP; PRV stations PR09, PR13, PR16, and PR24; and many other selected points of the distribution system. Readings are recorded on a regular basis. Meters are not calibrated, unless they are suspected of being inaccurate. The City is currently installing remote monitoring on all meters in Homer. As part of this effort, meters 10 years or older are being replaced.

It is recommended that major system meters be checked and calibrated periodically to ensure that they are presenting accurate readings. Such data will be essential for future water expansions and WTP/WST upgrades.

4.14 MAINTENANCE / OPERATIONAL RECORDS

The City has seven licensed water/wastewater operators: three level III operators, three level II operators, and one Operator in Training. The operators are specifically assigned to monitor and maintain the WTP, WSTs, WWTP, and lift stations; as well as the piping systems. The operators are on call 24 hours a day by radio or telephone to respond to emergencies related to the water system. Other City employees assist in the maintenance and repair of water system as needed.

Meter readings for raw water entering and potable water exiting the WTP are taken on a continuous basis, and recorded on the SCADA system. Operational information for water in the system, as well as chlorine concentrations, are recorded on a daily basis. Residual chlorine is measured at the end of the Homer Spit, East End Road, and the town proper. Potability tests (total coliform) are taken on a regular basis and reported to ADEC.

As previously noted, the WSTs were last cleaned in 2005, and are cleaned on an “as-needed” basis. Other regularly scheduled maintenance procedures are performed according to O&M manuals and the City’s annual maintenance program.

Detailed water production records are kept by the HDPW, which are also complied through the Alaska Water Use Data System, managed by the Alaska Hydrologic Survey (ADNR). Monthly data sheets are forwarded to ADNR for inclusion in the statewide data network. Water use data is available dating back to at least 1990.

The City keeps an inventory of critical replacement pumps, valves, and equipment available for the water supply facility and WTP in the event of equipment failure. Replacement equipment is also stocked for all PRVs.

Maintaining updated information will become more crucial as the City expands its sewage system. The City is currently developing a GIS-based inventory of all water lines, hydrants, pumps, and valves, as well as sewer appurtenances such as lift stations and manholes. In time, the database will be used to keep track of installation dates, equipment manufacturers, material of construction, date of last maintenance, and other information. This information will allow the City to perform routine repairs and maintenance more easily, as well as provide engineers with detailed information about the existing water system to accommodate design and construction of system upgrades or expansions. It will also allow the City to better define future needed or required improvements.

4.15 DEACTIVATED SYSTEMS

Before the construction of the current WTP, the only treatment to the Homer water system was chlorine injection at the A-frame building, which was later retrofitted as the “A”-frame PRV station (PR-03).

The City used six wells at the Bridge Creek Drainage for the town water supply, before the construction of the reservoir. These wells were discontinued because they developed operational problems including declining production and casing misalignment (Olympic, 1983).

4.16 PRIVATE WELLS / WATER HAUL

Homes and businesses not on the public water system typically maintain their own wells, or pay to have private contractors haul potable City water to a holding tank. It is estimated that approximately 800 customers, both inside and outside the City limits, have water hauled (customer list provided by bulk water haulers). Many of these customers are believed to be temporary residents that require only sporadic service. Bulk water hauled to holding tanks accounts for approximately 20 MG/year of the water production at the WTP.

Groundwater in Homer is generally unsuitable for residential and commercial water wells. Problems with wells include: shallow groundwater, low yields, the lack of a large well-defined freshwater aquifer, concerns about potential contamination, and the frequency of saltwater in wells near Kachemak Bay. Wells often have poor water quality, typically with high concentrations of minerals and sulfides. Most residents who are not on the City water system choose to have water hauled to their homes.

4.17 OTHER SYSTEMS

There are no other known public or private water systems within the City boundaries. The Homer Salvation Army Center operated a water system (Water System ID AK2245422), which was officially closed in December 2000.

5.0 EXISTING WASTEWATER SYSTEM

5.1 WASTEWATER TREATMENT PLANT (WWTP)

Homer's WWTP is a deep shaft sewage treatment plant with a capacity of 880,000 gpd. The WWTP is located on a 30-acre site near Beluga Slough that also houses the HDPW and repair shops. The WWTP has a National Pollutant Discharge Elimination System (NPDES) permit for the discharge of effluent to Kachemak Bay. The permit is currently being renewed. Refer to Appendix B for a detailed description of the WWTP, including general NPDES permit requirements.

5.1.1 Outfall

The WWTP has a single discharge point. After treatment, effluent is discharged into Kachemak Bay through a welded steel, 5/8-inch thick outfall pipe. The pipe diameter is 12-inch to the beach, and 20-inch from the beach to the diffuser. The discharge line extends approximately 2,500 feet from the shoreline to its discharge point. The line was installed in 1987. There are no known structural problems associated with the outfall line. The line was last inspected in 2005; no problems were identified during the inspection. Inspections of the outfall are difficult due to strong currents and the lack of visibility in Kachemak Bay.

High tidal conditions currently push saltwater back into the final clarifier at the WWTP, possibly due to the absence of sufficient air relief valves along the outfall line. There is currently a single air relief valve on the outfall. It is recommended that the need for additional air relief valves be studied, and that valves be installed along the outfall line if needed.

5.1.2 Biosolids Disposal

Homer continues to use dewatered, dried, and stabilized biosolids in beneficial land use applications. In the past, the City provided the material to local farms, and as fill material at the Homer Landfill. The City does not have a designated disposal area for the biosolids. Approximately 350 cubic yards of dewatered biosolids was disposed of in 2004 (Appendix B).

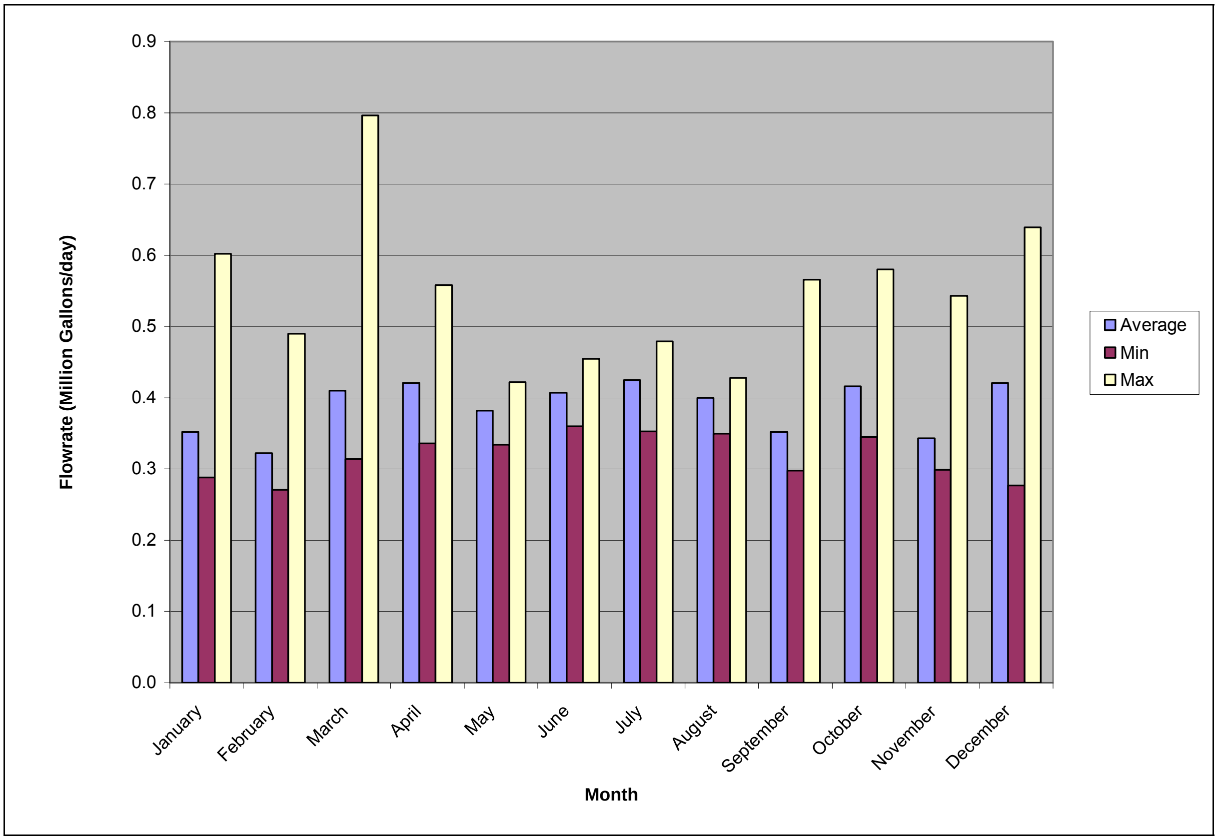
5.2 WASTEWATER FLOWS

5.2.1 Wastewater Production/Use

A total of 1,366 customers are served by the City sewer system. The WWTP currently treats an average daily flow of approximately 390,000 gpd. During intense rain storms, flowrates can increase to as much as 1.5 MG per day – which typically occurs several days each year. The design capacity of the WWTP is 880,000 gpd for a peak flow design. Flowrates into the WWTP can vary significantly, depending on the amount of inflow entering the system.

The 2003 I/I study established a residential wastewater flow of approximately 63.5 gpcd, which does not include I/I (USKH, 2003, page 6). Summer population influxes (primarily due to tourism) and storms can increase flow into the WWTP substantially. Chart 5.1 shows the average, maximum, and minimum WWTP daily influent flowrates for 2005. On October 23, 2002, a total of 1.74 MG of flow through the WWTP was recorded due to a major rainstorm.

Chart 5.1 Wastewater Treatment Plant Daily Flow Rates (2005)



5.2.2 Infiltration and Inflow (I/I)

A significant amount of I/I enters the Homer wastewater collection system on an annual basis, and is the major contributor to the system during severe storms. Inflow is defined as stormwater entering the piped sewer system from: 1) surface conduits (holes in Manhole lids, etc), 2) roof leaders, 3) foundation drains and sumps, and 4) connections to the storm sewer system. Infiltration is defined as groundwater entering the sewer system through cracks, holes, faulty connections, or other openings.

USKH completed a limited I/I study on the wastewater collection system in 2003. The study did not specifically determine the individual components of the I/I. The I/I Study (USKH, 2003) divided up the existing sewer system into four drainage basins (A, B, C, and D) and analyzed the amount of I/I that was occurring through these areas. Refer to Sheet 37 for the basin boundaries. The study was completed during relatively dry precipitation events. The results of the study concluded the following:

- Basin A (generally north of WWTP) – This basin has significant I/I.

- Basin B (all sewer mains east of Heath Street and North of Beluga Lake) – This basin has significant inflow.
- Basin C (approximately west of WWTP) – This basin may have some inflow, and moderate infiltration.
- Basin D (Homer Spit to Lake Street/Homer Bypass Intersection) – This basin may have some I/I.

The estimated base flow, inflow, and infiltration for each of the basins during an assumed typical storm are provided in Table 5.1.

Table 5.1 Estimated Flow Composition (% of Total Basin Flow) – Typical Rainfall

Basin	Base Flow (% of Total Basin Flow)	Inflow (% of Total Basin Inflow)	Infiltration (% of Total Basin Flow)
A	46	24	30
B	40	60	0
C	74	5	21
D	82	13	5
All Basins	**61**	**23**	**16**

Note: % = percent

During heavy rainstorms, inflow accounts for a substantially more significant component of the sewer system flows than infiltration. It should be noted that inflow reduction would be substantially less expensive to reduce than infiltration. The City has recently taken steps to reduce the amount of inflow coming to the WWTP. Section 8.3 discusses general measures to reduce inflow. It is highly recommended that the City enact a citywide inflow reduction program, which would have the following goals:

- Detailed investigations to determine sources of inflow;
- Public education – flyers could be sent with City sewer bills;
- Establishment of permitting, inspection, and regulatory requirements for sewer system connections;
- Mandate on-site inspections for suspected illegal connections and drains;
- Removal of illegal roof and foundation drains;
- Rehabilitation and repair of manholes and service connections;
- Regular smoke testing of collection mains to identify sources of inflow; and
- Create penalties for illegal connections and drains.

Infiltration is believed to account for an average of approximately 100,000 gpd of the flow entering the WWTP. Typical allowable infiltration rates in sanitary sewer systems are 450 to 625 gpd/inch-diameter/mile for an 8-inch sewer main, and 375 to 500 gpd/inch-diameter/mile for a 12-inch sewer main. These are the allowed infiltration rates typically defined by state and local government agencies. The *Standard Handbook for Civil Engineers* (1996, pg. 22-5) lists a typical value of 500 gpd/inch-diameter/mile. Inflitration rates for each basin are provided in Table 5.2. The infiltration rate in Basin A indicates that mains in this area are probably due for replacement. Basin C has reached, or is approaching the maximum infiltration rate that utilities typically allow. It is expected that significant sections of the gravity mains within Basins A and C will require replacement over the next 20 years. It is believed (though not proven) that the original asbestos cement (AC) mains account for a significant component of this infiltration. It is expected that no significant sections of mains within Basins B and D will require replacement over the next 20 years (although some individual pipes may warrant replacement).

Table 5.2 Infiltration Rates

Basin	Estimated Infiltration Rate (gpd/inch-diameter/mile)
A (upper basin)	625
A (lower basin)	1,500
B	0
C	471
D	152

Note: gpd = gallons per day

On November 23, 2002, Homer had a major storm event with a total one-day accumulation of 2.96 inches of water. A total of 1.46 MG of flow entered the WWTP, with over 1 MG of flow contributed by I/I (mostly inflow). The normal dry weather base flow for this month was approximately 0.375 to 0.5 MG per day.

In the future, it is recommended that the City perform smoke or dye testing, TV inspections, and/or additional monitoring on older sewer lines to determine specific areas where gravity mains should be replaced. Based upon the results of the I/I Study (USKH, 2003), the majority of the infiltration appears to be occurring where the older (1970 era) AC pipe is installed. Depending on precipitation conditions, I/I can contribute up to 1-MG per day of flow during extreme storms. Sewer mains in the following areas should be examined in order of priority:

1. The lower part of "Basin A" – Pipelines bounded by Fairview Avenue (to the north), the Sterling Highway (to the south), Heath Street (to the east, and including this street), and Main Street (to the west, and including this street);
2. The upper part of "Basin A" – Pipelines to the north of Fairview Avenue, between and including mains along Bartlett Street (to the west) and Heath Street/Anderson Street (to the east).
3. Basin C.

5.2.3 Leachate (Old Landfill)

The WWTP currently receives leachate from the Old Homer Landfill located on Fairview Avenue and Bartlett Street. The old landfill was capped and reconstructed as a ballfield. Landfill leachate can typically be detrimental to the operation of the WWTP, depending on its composition, and can cause fouling of WWTP components (such as the ultra-violet bulbs), and much higher biochemical oxygen demand (BOD). This leachate is believed to have high levels of BOD, chemical oxygen demand (COD), and iron.

5.3 GRAVITY MAINS

5.3.1 General

Approximately 37 miles of gravity sewer main exist in the current wastewater collection system. Components of the wastewater collection system were originally constructed in 1970. Gravity mains consist of AC, DI, and PVC. There are no active CI mains in the Homer wastewater collection system. The majority of the gravity collection mains within the City sewer system (excluding the Kachemak City VGES) are 8-inch ID mains. There are also some 6-, 10-, 12-, and 24-inch ID gravity mains in the sewer system. Refer to Table 5.3.

Table 5.3 Estimated Linear Feet of Active Gravity Sewer Main

Pipe Material	Linear Feet of Nominal Pipe Diameter				
	6-inch	8-inch	10-inch	12-inch	24-inch
Ductile Iron	700	73,200	7,900	200	-
Polyvinyl Chloride	-	11,000	-	-	2,000
Asbestos Cement	-	61,900	4,900	11,700	-

Gravity main blockage is generally not a problem, except for occasional clogs from the Land's End resort (located at the end of Homer Spit), and some minor clogs near the intersections of East Bunnell Avenue and Beluga Place, and Lake Street and Krueth Way.

In general, the collection system appears to have sufficient capacity to handle existing (year 2006) sanitary sewer flows. The sewer collection system capacity (not to be confused with the WWTP capacity) also appears to be generally sufficient (at its current size) to accommodate a typical 1-year and 10-year maximum storm event, based upon the results of the sewer modeling analysis and the findings of the I/I Study (USKH, 2003). Velocities throughout the system are below the level (10 feet per second) that erosion of sewer mains would typically occur. Very severe storm events could exceed the capacity of system, depending on the duration and magnitude of the inflow contribution. The City has taken steps in recent years to identify leaks in the sewer collection system (see Section 5.2.2). Sewer lines are generally repaired only when major leaks or breakages are discovered.

Based upon available survey data and as-built information, there is a possibility that overflows could occur on sections of main sewer line along Ocean Drive during existing (year

2006) peak flowrates (additional survey data will be required to confirm whether this is a concern).

5.3.2 Asbestos Cement Mains

About half of the Homer wastewater collection system is constructed of AC pipe, which was installed in the early 1970s. ADEC requires inventories of all AC piping in the water and sewer systems in the State of Alaska. AC pipe was not included in the list of banned asbestos-containing materials under the 1989 Toxic Substances Control Act; however, AC pipe can deteriorate in the presence of low pH groundwater, sulfides, sulfates, and chlorides.

The existing AC piping and manholes are believed to have high infiltration rates based upon the results of the I/I Study (USKH, 2003). The City may have to consider the gradual replacement or repair of at least some of the existing AC piping within the next 20 years (at which point the majority of the AC mains will be approximately 50 to 55 years old). This could be an extremely expensive capital improvement cost if the replacement or repair of a large number of AC mains is warranted. It is likely that piping in the lower (southern) half of Basin A will be scheduled for replacement first.

Representative sections of the City's AC piping should be tested to determine if deterioration has occurred. If significant deterioration is observed, the City should provide for a future replacement program. Similarly aged AC pipe is often recommended for replacement by other communities, especially in corrosive soils. The AC pipe that was installed in Homer may have a slightly longer lifespan due to design standards and the silty, clayey material where piping was installed (which minimizes overall infiltration/exfiltration). Over the next 20 years, major breaks in the existing AC sewer mains should be anticipated. It is recommended that the City replace these mains whenever road projects are scheduled above AC pipeline corridors.

The City should record known (past and future) breakages for the sewer lines in the Homer GIS database. Such records will be essential for the development of future pipe replacement programs.

5.4 WASTEWATER PUMP STATIONS

Homer maintains seven sewage pump stations. Two additional lift stations (10 and 11) serve private Septic Tank Effluent Pump (STEP) systems on the Homer Spit. Lift Station 5 is used to grind and dispose of fish carcasses from the processing plant. These lift stations pump sewage from topographical low points to maximize gravity flow in the sewage collection system. All of the lift stations in the Homer area are submersible units, with wet well pump installations. All lift stations in the City are dual-pump (duplex) stations, typically with Flygt brand pumps. Electrical panels for individual lift stations are located adjacent to each lift station. Table 5.4 presents lift station estimated capacities. Appendix K presents a summary of the individual lift stations, including pump models, sizes, and installation dates. Refer to Sheets 16 through 30 for lift station locations and force main layouts.

Table 5.4 Lift Station Summary

Lift Station	Name	Estimated Maximum Pumping Capacity (gpm)	Average Daily Flowrate 2025 (gpm)	Typical Maximum Daily Flowrate 2025 (gpm)
2	Beluga	800	140	390
3	Bay Avenue	380	10	20
5	Spit Outfall	Not calculated	Not calculated	Not calculated
6	Kachemak City	600	40	100
7	Campground	800	30	90
8	Launch Ramp	700	70	170
9	30-Acre	500	10	30

Notes:
gpm = gallons per minute

Mobile, standby generators are available for the lift stations in the event of a power outage. Pig launchers were installed for the Launch Ramp (Lift Station 8) and Kachemak City (Lift Station 6) force mains.

Based upon their age, the pumps in all lift stations but Beluga (Lift Station 2) may be in need of rebuilding or replacement in the next 5 years. The Beluga Lift Station was rehabilitated in October 2000.

The results of the sewer system modeling (refer to Section 8.6) indicate that wet well replacements will probably not be required for these lift stations over the next 20 years, and perhaps for a significant time period thereafter. The wet well for LS8 may end up being undersized for year 2026 – 2035 average demands.

Lift Stations 7, 8, and 9 may require a complete resizing of the systems, or special waste handling systems, if large atypical flows are sent to the system. For instance, cruise ships, coast guard vessels, and other large vessels that provide large contributions to the wastewater system may require large storage vaults at these stations to average out wastewater flow contributions. Wet wells and pump stations could also require upsizing, depending on expected contributions. If a typical cruise ship discharged 250,000 gallons of stored sewage, it would need the maximum pumping capacity of Lift Station 9 for over 8 hours, ignoring pump down time requirements and wet well volumes.

5.5 FORCE MAINS

There are approximately 43,800 LF (8.3 miles) of force main in the Homer sewage collection system, not including sewer outfalls. Force mains in the sewage system consist of DI and HDPE mains. Pumped flows along the Homer Spit from Freight Dock Road to FAA Road are served by dual HDPE force mains (3- and 6-inch ID). The 3-inch main is used during low winter flows, and the 6-inch main is used during high summer flows. Refer to Table 5.5.

Table 5.5 Estimated Linear Feet of Active Sewer Force Main

Lift Station	Description	Pipe Material	Linear Feet of Nominal Pipe Diameter		
			3-inch	4-inch	6-inch
2	Lake Street	Ductile Iron	-	-	1,000
3	Bay Avenue	Ductile Iron	200	-	-
6	Kachemak City	HDPE	-	-	3,900
7	Campground	HDPE	-	2,300	-
8	Launch Ramp	HDPE	18,200	-	18,200
		Total	**18,400**	**2,300**	**23,100**

Note: HDPE = high-density polyethylene

Lift Station 9 (30-Acre) does not have a force main; this lift station discharges into a gravity collection main.

Table 5.6 presents the expected maximum velocities that would be expected from peak daily flows in 2025. Assuming that a major (i.e. 10-year) storm or a significant surcharge (i.e.: cruise ship) does not occur, the force mains are believed to be adequate for typical maximum daily flows. Such atypical conditions could potential exceed the capacity of the force mains and lift stations, depending on the severity or magnitude of the event.

Table 5.6 Force Main Velocities

Lift Station	Description	Maximum Typical Velocity (ft/sec) – Year 2025
2	Lake Street	4
3	Bay Avenue	<1
6	Kachemak City	<1
7	Campground	2
8	Launch Ramp	2

Note: ft/sec = feet per second
< = less than

5.6 MANHOLES

There are approximately 700 manholes in the existing wastewater collection system. Manhole spacing throughout the Homer sewage collection system is generally sufficient for cleaning and flushing the sewage collection lines using existing equipment. The City has labeled all manholes with unique identification numbers, which have been incorporated into the City's GIS database for the sewer collection system.

5.7 SERVICE CONNECTIONS

There are currently approximately 1,366 customers on the City sewer system. A total of 956 customers are residential. Approximately 103 of these sewer service connections are located in the Kachemak City VGES, which is maintained by the City. Sewer service connections typically consist of 4-inch pipe. Significant freezing problems have not been reported at sewer service connections. Structures within 200 feet of a City sewer main fronting the property are required by City Ordinance to be connected. The City provides sewer service to property lines, and the remaining service is the responsibility of the property owner.

5.8 KACHEMAK CITY VGES

Parts of Kachemak City are served by a VGES, which connects to the City of Homer collection system. The VGES is maintained by the City of Homer and has approximately 110 service connections. Approximately 30,000 gpd of effluent from this collection system currently enters the Homer sewer system. The Kachemak City VGES consists of 12,000 feet of 6-inch and 28,000 feet of 4-inch HDPE main.

Kachemak City has an Intergovernmental Agreement with the City of Homer regarding costs for maintaining the VGES. The City of Homer historically disposed of septage from the Kachemak City septic tanks at the WWTP once every 2 years; however, disposal currently occurs every 3 years. Kachemak City is not served by the Homer water system; the majority of these residents have water holding tanks.

Future sewer systems such as a VGES or STEP system are not generally recommended for installation in the Homer area, due to the O&M requirements for these systems.

5.9 GREASE TRAPS

The City requires, by ordinance, that grease traps or similar devices to be installed at "generator" businesses and facilities to prevent downstream grease congealing and clogging of sewer lines. Grease traps are typically required for restaurants, hotels, hospitals, schools, garages, and car washes. The City does not make regular inspections of the traps to ensure they are operating properly, and it is unknown if the grease traps are properly maintained. The traps are ineffective if they are not regularly cleaned. Clogging problems on the Homer Spit could be attributed, in part, to the lack of maintenance for grease traps at restaurants.

5.10 MAINTENANCE / OPERATIONAL RECORDS

Refer to Section 4.14 for general O&M information relating to both the water and sewer utilities. Lift stations are monitored daily by City personnel. Periodic readings at the lift stations relative to pump run time and noted deficiencies are maintained. One quarter of the sewer main system is flushed every year; therefore, sewer mains are flushed once every 4 years. The City keeps certain critical replacement pumps, valves, and equipment available for the WWTP and lift stations in the event of equipment failure. Replacement equipment is available for all lift stations.

5.11 ON-SITE WASTEWATER DISPOSAL

The majority of homeowners and businesses in the study area that are not currently served by the City wastewater maintain individual septic systems. A few residents have holding tanks. Approximately 47 percent of the residents of Homer are believed to be served by on-site wastewater systems. A significant number of on-site systems in Homer function poorly due to the soil conditions in the area. There is no data available on the types of systems installed or the frequency of failures. It is believed that the majority of active wells in town use water from shallow aquifers, or aquifers with poor water quality. As development increases in the City, the potential for groundwater contamination from septic systems and other sources could increase.

The City accepts residential holding tank wastewater (which includes small bed and breakfasts and churches), and sludge from the Kachemak City VGES, but not commercial or other residential septic tank wastewater. There are approximately six wastewater holding tanks in the community. Septic tank haulers reportedly dispose of sludge at permitted septage monofills in the area, although at least one hauler travels as far as the Girdwood WWTP, approximately 184 road miles away (368 miles round trip) to dispose of the waste.

Costs for conventional residential septic systems in the Homer are estimated at between $6,000 and $9,000. If mound systems or aerobic treatment systems are required, costs are estimated at between $15,000 and $30,000. If effluent disinfection is included, the cost for installation typically increases by an estimated $1,000. Engineered systems are required for the majority of the septic systems within the City, due to local soil conditions. System failure rates are unknown.

6.0 FUTURE EXPANSION AREAS AND FLOWS

6.1 LAND USE / DEVELOPMENT

As discussed in Section 3.1, the population of Homer is expected to experience moderate growth over the life of this WSMP (2006 through 2025). The population of the City is estimated to double over the next 20 years, based on an average annual growth rate of 4.5 percent for the next 10 years, then 3.0 percent for the subsequent 10 years. Future growth is expected to increase the burden on the water and sewer system's ability to meet overall user and emergency fire demands, and minimum storage requirements. Future population demands will also increase the demand on the Bridge Creek Reservoir.

Growth is expected to occur throughout the City as the population increases. Recent utility expansions were completed at Hillside Acres and along East Road. Significant development is expected to occur at the following locations within the next 5 years:

- South Slope Road and Vicinity;
- Kachemak Drive;
- West Hill Road; and
- Chistensen Tracts.

There are numerous septic tanks and wastewater holding tanks in the community. Many residents have private wells, or have water delivered to storage tanks at their homes. The majority of future growth will be single and multifamily homes, with the accompanying commercial, light industrial, and public facilities.

6.2 PROJECTED WATER FLOWS

The current (2005) per capita usage for the City (as discussed in Section 5.2) is estimated at roughly 82 gpcd, which does not include leakage, bleeding, hauled water, and transient demands. For comparison, the estimated per capita usage for Anchorage, Alaska, is approximately 73 gpcd (Spano, 2003).

Although the population of the City has increased since the closure of the Icicle Sea Foods Plant in 1998, the total yearly consumption between 1998 and 2003 remained relatively stable. In 2004 and 2005, consumption rose notably over rates between 1990 and 2003.

Current (2006) average winter and summer daily water flows are estimated at 520,000 and 800,000 respectively. An estimate of the total future water demands for the system is presented in Table 6.1. This estimate assumes that 85 percent of the city residents are connected to the distribution system in 2025, with a population of 11,244 residents. This estimate also assumes that bleeding, leakage, and transient demands will increase linearly with population growth.

Table 6.1 Projected Daily Water Demands[1] (2006-2025)

Year	2006	2011	2016	2021	2025
Summer[2] Average Flowrate (gpd)	800,000	1,040,000	1,330,000	1,600,000	1,860,000
Yearly Average Flowrate (gpd)	570,000	790,000	1,060,000	1,330,000	1,590,000
Winter Average Flowrate[3] (gpd)	490,000	590,000	750,000	870,000	980,000
Summer Maximum Daily Flowrate (gpd)	1,280,000	1,660,000	2,130,000	2,570,000	2,980,000

Notes:

[1]These projections assume that future leakage, and "bleeding" are proportionate to existing flowrates.

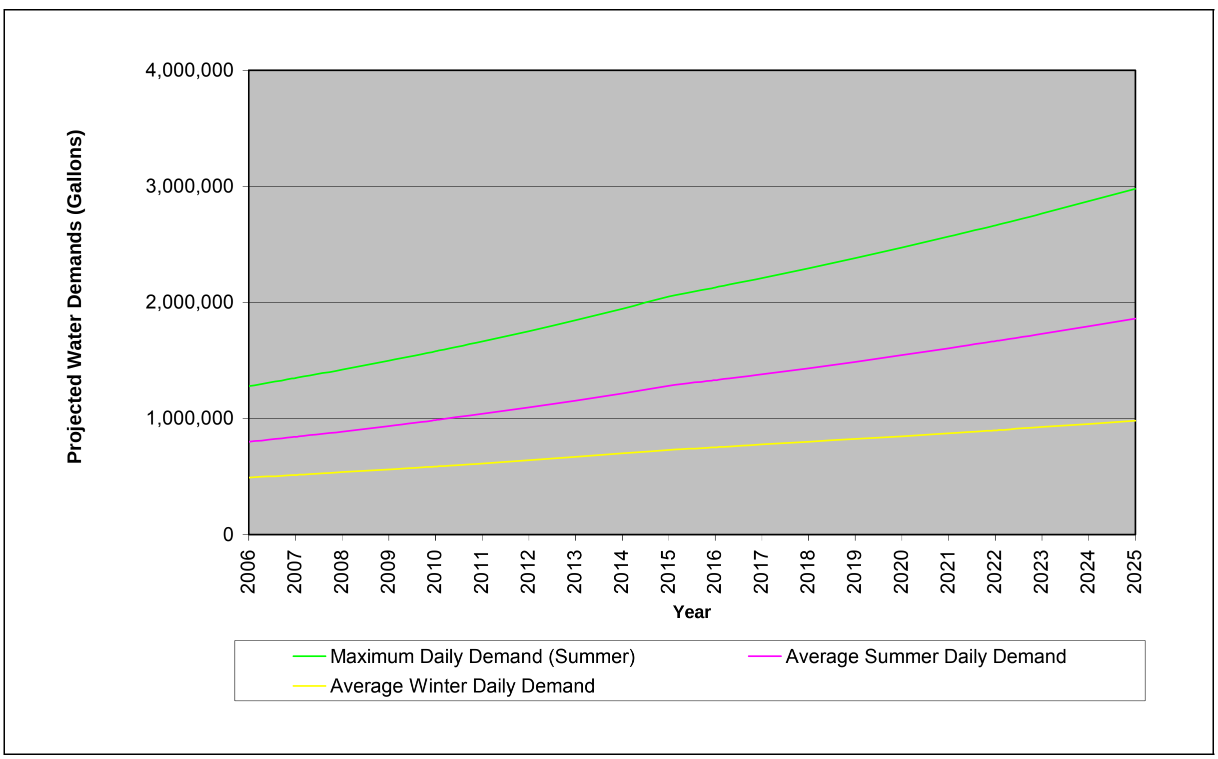
[2]June, July, and August

[3]November, December, January, and February

gpd = gallons per day

Water demand by 2025 is estimated to increase to an average of 1.9 MG per day (summer) and 1.0 MG per day (winter). Summer maximum daily demands for 2025 are estimated at approximately 3.0 MG per day (or 1.6 times the average daily demand). The maximum summer hourly demand for 2025 is estimated at approximately 3,200 gpm (or 2.5 times the average summer day demand). Projected summer and winter flowrates for 2006 through 2025 are presented on Chart 6.1.

Chart 6.1 Projected Water Demands (2006-2025)



The community has sufficient capacity for a worst-case fire demand (3,500 gpm for 3 hours = 630,000 gallons). If the water source were to shut down, the community would have sufficient storage for approximately 3.3 days during the summer, assuming that the WTP has a stable power supply. Fireflow requirements are discussed in Section 4.8.

6.3 PROJECTED WASTEWATER FLOWS

Projected wastewater production rates for 2025 are presented in Table 6.2. Wastewater collection and treatment facilities are sized based upon flow volumes and waste loading. Limiting wastewater loading from I/I production will allow for smaller collection mains, pumps, and process equipment.

Table 6.2 Projected Wastewater Production Rates (2025)

Flow Type	Rate
Minimum flow	1.0 -1.1 MGD (Infiltration & base flows only - no inflow)
Flow during a Typical Storm	1.35 MGD (includes inflow)
Average flow	1.1 to 1.2 MGD
Flow during a severe storm	3.9 MGD
Minimum daily flow (winter)	550,000 - 750,000 gpd (includes infiltration)
Peak flow (morning or evening)	2,000 gpm
Average Flow - typical storm (not during peak hourly flows)	850 gpm

Notes: [1]MGD = million gallons a day
[2]gpm = gallons per minute

The sewer system expansions described in Section 9.3 will place additional loadings on the collection system. Over time, the capacity of certain piping will not be sufficient to provide for peak flowrates without overloading. Overloading must be avoided to prevent pressurized conditions (surcharging) from developing in the mains, and to prevent sewer overflows into service connections and manholes. It is anticipated that four mains will require replacement by 2025, based upon their expected peak flow capacities (Table 6.3). In these instances, the main size will have to be increased to accommodate peak loadings. Three additional mains may approach peak loading capacities by 2025. Replacement of the mains listed in Table 6.3 may or may not be warranted, depending on actual growth.

Table 6.3 Sewer Mains Exceeding or Approaching Capacity in 2025

Main Location	Inlet Manhole	Outlet Manhole	Existing Main Diameter (inches)	Replacement Main Diameter (inches)
Exceeding Peak Loading Capacity				
Beluga Place, Hansen Ave	SSMH201	SSMH398	12	24
South End of WWTP[1]	SSMH398	SSMH580	12	24
Sterling Highway	SSMH210	SSMH198	8	16
Sterling Highway	SSMH190	SSMH189	8	12
Approaching Peak Loading Capacity				
Sewer main east of Lake Street (near Ben Walters Lane)	2-86	SSMH224	12	24
Bunnell Avenue	SSMH204	SSMH201	8	12
Ocean Drive[2]	SSMH28	SSMH272	10	≥ 12

Notes:

1) Survey required to verify existing slopes. The main may only need partial replacement.

2) Survey required to verify existing slopes. Partial sections of this main may or may not need replacement.

7.0 WATER SANITATION FACILITIES ALTERNATIVES

This section provides a discussion of selected alternatives for future water facilities, and is the basis of the CIP recommendations in Section 9. A general overview of the proposed sanitation improvements is provided in the Executive Summary for this WSMP.

7.1 WATER SUPPLY (PHASE 1)

The development of various surface water supply options is described in detail in the Hydrology Report provided in Appendix C. This study is an "empirical," rough order of magnitude analysis, and does not include detailed record flow data from Bridge Creek, Twitter Creek, and other potential sources of surface water. It is believed that the development of a new water supply will not be required between 2006 and 2015. Sometime after 2016, a supplemental reservoir may be required to provide additional water to the system in the event of a low flow event (drought). Refer to Appendix C, which provides the basis of this recommendation. Calculations supporting the study's conclusions are also provided in Appendix C. It is recommended that a detailed hydrology study be started in 2006 to better define the extent of flows, and a potential impoundment.

As the population expands, the City will face increased demands on the current reservoir's ability to provide reliable, low turbidity water to the WTP. As previously discussed, wells in the Homer area are not known to have sustained, high yield, high quality water. Wells with good quality water are believed to be typically shallow, and susceptible to contamination.

The potential new impoundment would be expected to be located on the upper part of Twitter Creek. The specific location of the impoundment would be determined at a later date, assuming it is possible to construct a facility in the basin. A cross-basin pipeline would be built to divert water from the Twitter Creek impoundment in low flow situations, and would either divert water directly to the Bridge Creek Reservoir, or to the Bridge Creek pump station. The choice of where the diversion is located would depend on the location of the impoundment in the Twitter Creek basin, and the treatment requirements for the water.

A separate watershed protection boundary would have to be established for the impacted part of the Twitter Creek basin, similar to the current Bridge Creek Watershed boundary. A pipeline approximately 3 miles in length would be required to transfer water to the Bridge Creek Reservoir; approximately 3.5 miles of pipe would be required to serve the Bridge Creek pump station. A pump station with a pig system would be installed at the Twitter Creek diversion point. Depending on the location of the Twitter Creek Reservoir, the pipeline could have minimal pumping requirements, since the new reservoir could potentially be at or above the current elevation of the Bridge Creek Reservoir. Refer to Table 7.1 for very preliminary costs for the Twitter Creek source development.

Table 7.1 Water Supply

Phase	Recommended Improvement	Design/Construction Costs ($)
1.1	Twitter Creek/Bridge Creek hydrology study	75,000 - 150,000
1.2	Twitter Creek impoundment	1,200,000 - 2,000,000
1.3	Twitter Creek pump station (and power requirements)	500,000 - 750,000
1.4	Twitter Creek water main (16,000 to 18,500 linear feet)	1,600,000

Desalination is a potential option, but is very expensive, and is not currently cost effective. Desalination typically costs approximately $2 to $3 per thousand gallons in the United States to produce (which does not include distribution), and is very “energy intensive.” Significant pumping requirements would be required to serve areas at higher elevations and, depending on valve configurations, backflow up through the existing PRV stations may not be possible. There is also a possibility of potential damage to such a treatment facility located on the Homer shoreline from tsunamis.

7.2 WATER TREATMENT PLANT (PHASE 2)

Recommendations for future improvements to the WTP are discussed in detail in Appendix A.

7.3 SKYLINE DRIVE PRESSURE ZONE (PHASE 5.26)

Recommended Alternative (Option 1)

The most practical alternative to providing Skyline Drive with water is the installation of a pressure pump station at the WTP. This station would provide a nominal pressure equivalent to an elevation head of 1,250 feet above MSL. The new pressure zone could serve areas adjacent to Skyline Drive, such as Glacier View Court, Scenic Place, Horizon Court, Tulin Terrace Boulevard, Crestwood Circle, and Ridgeway Court. The new pressure zone could also serve Paintbrush Street, and replace the (future) booster pump station planned for installation on Paintbrush Street near Fireweed Avenue. Service to Scenic Place and Horizon Court would require a PRV station (48), which would provide a pressure drop of approximately 45 psi.

The cost for a pump station to serve Skyline Drive is estimated at approximately $400,000. It is assumed that land would have already been purchased or made available for the pump station.

Not recommended

Several potential tank sites were evaluated relative to providing Skyline Drive with adequate water service using a conventional tank/standpipe, as alternatives to a pump station at the WTP. The following alternatives were identified:

- Option 2 – An undeveloped hill (elevation = 1,134 feet above MSL) 1,600 feet northwest of the WTP, within the existing City Limits (Parcel ID 17307068).

- Option 3 – An undeveloped hill (elevation = 1,182 feet above MSL) 4,600 feet northeast of the WTP. This site is approximately 450 feet outside of the City limits.
- Option 4 – Crossman Ridge (elevation = 1,326 feet above MSL), approximately 1.5 miles north of the WTP (at a minimum). Approximately 2.0 miles of transmission line (one-way) would be required from the WTP to the WST site.

Table 7.2 provides approximate service pressures for these tank options, based upon various tank heights. In summary – the option of a dedicated storage tank is too expensive.

Table 7.2 Skyline Ridge Water Storage Tank Options

Option	Tank Height (feet)	Skyline Drive Static Pressures (psi) [1, 2]	Crestwood Circle Static Pressure (psi)[1,3]	Ridgeway Court Static Pressure (psi)[1,2]
1	40	1 - 56	62	25
1	80	18 - 74	80	42
2	40	22 - 77	73	46
2	80	39 - 94	101	63
3[3]	20	75 - 131	137	100

Notes:

[1]Assumes PRV stations are not installed to serve this area(s).

[2]Static pressures below 30 psi and above 80 psi are not considered adequate for water service.

[3]For Option 4, a PRV could be installed below the tank, assuming the site is 1,326 feet above MSL. Another option would be to install a 20-foot tall tank at a site 1,240 feet above MSL.

MSL = mean sea level
PRV = pressure reducing valve
psi = pounds per square inch

7.4 South Peninsula Hospital (Phase 5.27)

The South Peninsula Hospital currently has low-pressure problems, especially on the second floor of the hospital. This is due in part to head losses across backflow prevention device(s), and elevation head losses on the second floor. Fire hydrants near the hospital can currently provide a combined fireflow of approximately 2,000 gpm at 20 psi residual. Six options were evaluated that would allow for increased static pressures at the hospital. These options included:

- Option 1 – Providing a 1,500 LF, 12-inch ID transmission line from the Midhill pressure zone, just above the inlet of the A-frame PRV station, to the hospital. This would allow the City to decrease the outlet pressure of the A-frame PRV to 36 psi (currently 49 psi), thereby decreasing the high static pressures in certain areas of the existing A-frame service zone. A smaller PRV station would be installed along this main to decrease the service pressure into the hospital. Two small areas of the "A-frame" zone (Calamari Court and Daybreeze Court; and Fairview Avenue between

Hohe Street and Main Street) would be revalved in order to be connected to the Pressure Zone 9 (Main Street / Kachemak / Hodel).

- Option 2 – Increasing the outlet pressure of the A-frame PRV with some valve (zone) modifications.
- Option 3 – Providing a new PRV station along Hohe Street between Danview and Mainview.
- Option 4 – Reconfiguring zone valving, with a possible extension of the Cityview Avenue main.
- Option 5 – Install a booster station and a small WST at the hospital.
- Option 6 – Connect the A-frame zone to the East Hill transmission line. Option 6 is recommended by this WSMP (Phase 5.9), but was determined to not significantly affect static pressures at the hospital. This improvement is required to provide an approximately 3,400-gpm (20 psi residual) fire flow at the hospital. Fire flows would be required to be taken from a combination of both the new "Midhill zone" transmission main, and the existing A-frame zone mains. The maximum fire flow currently available at the hospital is approximately 2,000 gpm (20 psi residual).

The recommended option for increasing service pressures at the hospital is Option 1, the construction of a 1,500 LF transmission line from the Midhill pressure zone to the hospital. Option 1 will provide generally more desirable service pressures in the A-frame zone, while maintaining minimum fire flow requirements. Static pressures in the A-frame zone are currently 47 to 108 psi. The new static pressures for the expanded A-frame zone would be approximately 30 to 94 psi. The static pressure at the hospital would be 90 psi (prior to entering the backflow prevention device).

Calamari Court and Daybreeze Court would be revalved from Pressure Zone 4 (A-frame outlet) to Pressure Zone 9 (Main Street/Kachemak/Hoedel's). Static pressures in this area would be reduced to the 43 to 49 psi range (currently 100 to 108 psi). The hydrants adjacent to these lots would provide sufficient fire flows, although an additional hydrant may be required on Mullikin Street between Noview Avenue and Soundview Avenue.

Option 1 will not affect planned Water Improvement Districts (WIDs). Total available fire flow at the hospital would increase from approximately 2,000 gpm to 3,400 gpm at 20 psi. It is assumed that fire flows would be available from both the new Midhill pressure zone main to the hospital, and the A-frame pressure zone. It is also assumed that Option 6 (East Hill Road / A-frame zone transmission main) will be installed. The cost of the main extension and new PRV station is provided in Section 9.2.

7.5 FIRE HYDRANTS (PHASE 5.28)

Hydrant spacing is generally sufficient to provide coverage to most areas of the distribution system, but there are areas where hydrant coverage is lacking. Sheet 38 shows those areas where hydrant coverage is sufficient, and where additional hydrants are recommended for installation. A total of 53 additional hydrants are recommended for installation within the current water distribution system. The projected cost is approximately $270,000

($5,100/hydrant x 53 hydrants). Refer to Sheet 38 for the recommended locations. Refer to Section 4.8.3 for additional information.

7.6 FIRE FLOW IMPROVEMENTS (PHASES 5.11 AND 5.29)

7.6.1 Recommended Fire Improvements

Fire flows are generally adequate throughout the majority of the City. Fire flows at the locations listed in Table 7.3 are not sufficient to meet minimum recommended fire flow requirements. Refer to Section 4.8 for additional discussion of fire flow requirements.

Table 7.3 Deficient Fire Flows – Improvements Recommended for Fire Protection

Phase	Location	Available Fire Flow at 20 psi (gpm)	ISO Recommended Fire Flow at 20 psi (gpm)	Existing Main Size (nominal)
5.11	Paintbrush Court	See Note 1	1,000	6
5.29	High School	2,275	3,500	6/8

Note:

1 Static pressures on Paintbush Court are below 20 psi; this area will not meet minimum static pressures and fire flow requirements without a new booster station, or a higher pressure zone (new, higher storage tank) for this area.

gpm = gallons per minute
ISO = Insurance Services Office
psi = pounds per square inch

Paintbrush Court Booster Station (Phase 5.11)

The design for a booster station to provide normal service pressures to Paintbrush Court was completed in 1998, but the station was never constructed. The station would have been equipped with one 1.5 hp and one 7.5 hp booster pumps. The cost to construct this station is estimated at $190,000 (this estimate was $154,517 in 1998). The development of this booster station is currently recommended because of the low (6 to 17 psi) static pressures in this area. The development of a pipeline along Skyline Drive may eventually eliminate the need for this booster station.

High School (Phase 5.29)

There are two viable options to provide adequate fire flow the High School.

- **Recommended:** Extend Heath Street to Rainbow Court. Replace the existing 6-inch main along Rainbow Court with a 16-inch main from Kachemak Way to the High School Loop. Because this improvement is based on fire flow requirements, and not Rehabilitation and Repair (R&R) requirements, it is included under the Phase 5 CIP improvements, and not the Phase 4 R&R improvements.
- **Option:** Extend Heath Street to Rainbow Court. Extend Anderson Street approximately 1,200 LF to Rainbow Court. Install a new PRV station along the new

Anderson Street Main to provide the same pressure drop as the Kachemak City, Main Street, and Hodel PRV stations.

7.6.2 General Discussion

Fire flows at the locations listed in Table 7.4 are not sufficient, but the development of the future West Hill Road water extension and other Phase 5 improvements should allow for adequate fire protection at these areas without the need for additional improvements in the general vicinity of the schools.

Table 7.4 Deficient Fire Flows – Fire Protection Provided by Other Improvements

Location	2006 Fire Flow (gpm)	2025 Expected Fire Flow (gpm)
Middle School	2,500	5,200
West Homer Elementary School	1,600	4,100

Note: gpm = gallons per minute

Fire flows that are deficient in areas of the existing system, but not recommended for upgrades, are presented in Table 7.5. In one case, the modeled flows was just slightly below the minimum recommended requirement for residential and commercial flows (1,000 gpm at 20 psi). It was determined that the main improvements would not be economically beneficial (example: replacing the 4-inch ID main from the spur line off Fire Hydrant 212 on North Larkspur Circle with a 6-inch ID main would provide increased fire protection to one property (530 gpm for 4-inch to 1,150 gpm for 6-inch), for a projected cost of approximately $51,700 ($110/LF x 470 feet).

Table 7.5 Deficient Fire Flows – Improvements Not Recommended

Location	Flow at 20 psi (gpm)	ISO Recommended Fire Flow at 20 psi (gpm)	Existing Nominal Main Size (inches)
FH330 (Sea Breeze)	905	N/A	6
FH222 & FH222A (WTP)	N/A	N/A	6 & 12
North Larkspur Circle	550	N/A	4
Lakeside Court	660	N/A	4
Airport (FAA Road)	1,100	3,500	10
Airport (Kachemak Drive)	2,100	3,500	12

Notes:

FH	=	Fire Hydrant	psi	=	pounds per square inch
gpm	=	gallons per minute	WTP	=	Water Treatment Plant
N/A	=	not available			

7.7 LOW-WATER-USE IMPROVEMENTS

The City could consider requiring the installation of low-flow toilets and shower nozzles for newly constructed homes and businesses served by the Homer water distribution system, including all proposed expansion areas. The intent would be to lower the usage for toilet flushing and showering, and reduce the demand on the City WTP and WWTP. Other potential improvements could also be implemented, such as hot water recirculation pumps and faucet aerators.

7.8 WATER SYSTEM MODELING

The Homer water system model was calibrated with hydrant flow data provided by the City. These tests were conducted in June 2004. The water system modeling platform was H20Map Water GIS 5.0. The model was used to identify static and fire flow pressures throughout existing and planned future expansion areas of the City. The model was also used to determine flow rates during fire flow conditions, and PRV setting modifications. The final calibration of the existing water model is provided in Appendix L.

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8.0 SEWER SANITATION FACILITIES ALTERNATIVES

This section includes a discussion of selected alternatives for future wastewater facilities, and is the basis of the CIP recommendations in Section 9.0.

8.1 WASTEWATER TREATMENT PLANT (PHASE 6)

Recommendations for future improvements to the WWTP are discussed in detail in Appendix B and Sheets 33A and 33B.

8.2 EXISTING SEWAGE LIFT STATIONS (PHASE 7)

There are seven City-operated submersible sewage lift stations in Homer. Refer to Section 5.4 for a detailed description of these stations. It is expected that all the lift stations will require a complete rehabilitation of the pumps, electrical systems, and associated rails at least once over the next 20 years. At a minimum, most pumps will have to be rebuilt or replaced in the next 5 years. Assuming no significant deterioration of the wet wells occurs, wet well capacities should be adequate until 2025. There is a possibility that the wet well for Lift Station 8 (Launch Ramp) may need to be updgraded by 2025 for increased capacity, depending on the ultimate development of the Homer Spit.

8.3 I/I REDUCTION (PHASE 7.1)

It is recommended that the City enact an I/I reduction program. The development of this program is essential because overloaded sanitary sewers can cause basement flooding, and increase the amount of wastewater that has to be treated at the WWTP. In general, a reduction in I/I can result in capital and O&M savings due to reduced treatment and pumping costs, and reduced main sizes.

8.3.1 Inflow

General steps to reduce inflow include:

- Remove foundation and roof drains to sanitary sewer system.
- Smoke testing.
- Identify downspouts, groundwater sump pumps, foundation drains, drains from window wells, drains from outdoor basement stairwells, and drains from driveways.
- Rebuild old and damaged manholes to reduce inflow sources.
- Manholes – Replace manhole covers that have pick and vent holes, with watertight manholes that have gaskets. Install plastic or metal manhole inserts. Raise manholes in known floodplain and wetland areas. Apply low temperature sealants around manhole frames and grade ring exterior joints.

8.3.2 Infiltration

General steps to monitor for and reduce infiltration include:

- Use flow monitoring equipment to identify specific main segments that have significant infiltration;
- Purchase TV inspection equipment to inspect inside sewer lines and identify sources of infiltration;
- Rehabilitate service laterals and connections;
- Slip line sewer mains to repair cracked or broken joints;
- Pipe replacement – this can be extremely expensive, but may be required where severe structural degradation has occurred;
- Replace or repair leaky manholes;
- Pipebursting; and
- Sealing/Resin coating.

As the sewer system continues to age, the City will face a growing problem with increased infiltration from deteriorating mains. The City should consider enacting an annual pipe replacement program that identifies key areas of leakage, and provides an annual budget and schedule for pipe replacements. This will at reduce the potential for large scale infiltration to create a significant burden on the WWTP.

8.4 KACHEMAK DRIVE PHASES I, II, AND III (PHASE 8.12)

A conventional gravity/force main system is recommended to serve Kachemak Drive and Beach Road between Airtaxi Place and East Road. Phase I of this project is currently being designed and constructed. Approximately 16,100 LF of gravity main and 2,150 LF of force main would be installed, and would connect to existing sewer mains on Kachemak Drive and East End Road. A summary of the estimated construction costs is provided in Section 9.4. Refer to Sheets 26 and 27.

8.5 STERLING HIGHWAY DUAL FORCE MAIN (8.34)

Although not required, an additional force main could be installed along the Sterling Highway from Lift Station 2 to Lake Street. The force main would provide redundancy for Homer Spit services, and would be used to accommodate peak flows during high summer demands. Approximately 1,000 LF of force main would be installed, and would parallel the existing 6-inch DI force main. It is not expected that this main would be required until approximately 2021 through 2025. The estimated construction cost is provided in Section 9.4. Refer to Sheet 25.

8.6 WASTEWATER MODELING

The sewer system modeling platform was H20Map Sewer GIS 6.0. The modeling effort identified mains that would develop insufficient capacity within the planning period, during peak flows and typical storm events. The model provided a basis for calculating replacement main sizes, as well as sizing requirements for new mains in the system. The sewer system model was developed based upon known invert elevations, WWTP and lift station flowrates, and I/I data (USKH, 2003). Hydraulic roughness factors were assumed for all piping materials. Normal daily demands were estimated based upon existing population densities, commercial meter records, WWTP meter records, and other data provided by the HDPW. The following assumptions were made for estimating year 2025 flowrates and collection system capacities:

- The hourly peak flowrate for 2006 is assumed to be 3.3 times the average daily flowrate. The hourly peak flowrate for 2025 is estimated to be 2.9 times the average daily flowrates, based upon year 2025 population projections.
- Average daily flowrates will increase linearly with population growth.
- Inflow will remain the same for existing manholes (reductions due to manhole repairs and removal of illegal drains will match any increased inflow for newly installed services and further deterioration).
- It is assumed that infiltration will remain constant for existing piping (reduced infiltration from piping replacements will hopefully keep pace with increased infiltration from deteriorating piping).
- New piping will experience relatively little infiltration over the next 20 years, and was modeled to match Basin D flows (as outlined by the 2003 USKH I/I study).
- Inflow into newly served areas is assumed to mirror existing inflow in the lower part of Basin C (as outlined by the 2003 USKH I/I study).

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9.0 SELECTED IMPROVEMENTS AND PHASED CONSTRUCTION

9.1 WATER AND SEWER PHASING / BASIS OF COST ESTIMATES

The projects identified in this section form the basis for the recommended City CIP list. The CIP list will be used by the City to plan, fund, design, and construct sanitation projects. Cost criteria are described in Section 9.4. The projects have been divided into eight categories:

- Phase 1 (Water Supply);
- Phase 2 (WTP);
- Phase 3 (Water Storage);
- Phase 4 (Water Distribution System R&R);
- Phase 5 (Water Distribution System Expansions);
- Phase 6 (WWTP);
- Phase 7 (Sewer Collection System R&R); and
- Phase 8 (Sewer Collection System Expansions).

The CIP projects for Phases 1, 2, 3, and 6 have been divided into phases, based upon the year of design/construction: 1) 2006 through 2010, 2) 2011 through 2015, and 3) 2016 through 2025. The development of project phasing priorities for Phases 4, 5, 7, and 8 are based upon annual requirements, general subdivision boundaries, and construction practicality. Projects in the 2011 through 2025 timeline address long-term needs. Project priorities depend on many variables that can change over time, and should be periodically re-evaluated by the City. Population growth rates, funding, and other construction projects can influence the ultimate priority sequence. Costs in this document are presented in year 2006 dollars.

9.2 WATER SYSTEM CAPITAL IMPROVEMENT PROGRAM

9.2.1 Water Supply Capital Improvement Schedule (Phase 1)

An estimate of the City's future water demands is provided in Tables 6.1 and 9.1. By 2025, the total demand on the Bridge Creek Reservoir raw water source is estimated to be approximately 644 million gallons per year. This includes bleeding and leakage, as well as transient, residential, commercial, and industrial demands.

As previously discussed in Section 7.1, the proposed Twitter Creek and Bridge Creek improvements have a preliminary estimated cost of approximately $4,000,000 (+/- 560,000).

9.2.2 Water Treatment Plant Capital Improvement Schedule (Phase 2)

In the near future, the community will need to provide for increased water treatment capacity or increased efficiency to provide for existing needs, as well as expanding service. Increases in the treatment efficiency of the WTP, including less wasting of backwash water, are described in Appendix A. Other recommended immediate improvements to the current WTP are also summarized in Appendix A. Sheets 31A, 31B, and 31C show the recommended WTP improvements.

9.2.3 Water Storage Capital Improvement Schedule (Phase 3)

Water demands will mirror population growth. A 4.5 percent population growth rate is assumed for 2006-2015, and a 3 percent annual average growth rate is assumed for 2016-2025, based upon population projections provided by the City. The worst-case fire demand includes an assumed fireflow is a 3 hour duration, 3,500 gpm flow, with a minimum of 3 days average day demand plus two fire demands (AWWU, 2005). With the nominal base storage requirement of 3 days (average summer demand) plus the additional worst-case fire demand, the following is assumed:

- The City *currently* requires approximately 350,000 gallons of additional storage.
- By 2025, it is expected that maximum summer day demands will approach 3.0 MG per day. The current (2006) storage capacity is approximately 2.65 MG.
- By 2025, the current (2006) storage capacity will only be sufficient for approximately 1.4 days of storage for average summer demands, not accounting for fire flows.
- By 2025, the current (2006) storage capacity will only allow for approximately 1.1 days of reserve storage, plus a 3,500 gpm, 3-hour fire demand.

Table 9.1 presents existing and proposed water storage versus water and fire flows during the years that additional tanks are recommended for installation.

Table 9.1 Projected Storage Versus Demand

Year	2006	2007	2010	2016	2022	2025
Average Winter Daily Demand (gpd)	490,000	510,000	590,000	750,000	900,000	980,000
Average Summer Daily Demand (gpd)	800,000	840,000	990,000	1,330,000	1,670,000	1,860,000
3 Days Average Summer Daily Demand + Maximum Fireflow[1] (gallons)	3,030,000	3,160,000	3,590,000	4,620,000	5,630,000	6,220,000
Maximum Summer Daily Demand (gpd)	1,280,000	1,350,000	1,580,000	2,130,000	2,660,000	2,980,000
Maximum Summer Daily Demand + Maximum Fireflow[1] (gallons)	1,910,000	1,980,000	2,210,000	2,760,000	3,290,000	3,610,000
Proposed Water Storage (gallons)	2,650,000	3,650,000	4,650,000	5,650,000	6,650,000	6,650,000
Number of days existing storage (Average Summer Demand + Fireflow)	2.5	2.4	2.0	1.5	1.2	1.1
Number of days proposed storage (Average Summer Demand + Fireflow)	2.5	3.6	4.1	3.8	3.6	3.2

Notes:
[1]Assumed fireflow is 3 hour duration, 3,500 gallons per minute flow = 630,000 gallons
gpd = gallons per day

A summary of proposed WST installations is as follows:

- It is recommended that the City construct an additional 1 MG storage tank in 2007-2008. The first WST is recommended for installation between Kachemak Way and East Hill Road near the Crandall Addition subdivision. A proposed tank location is shown on Sheet 6.
- The additional 1 MG of storage should be sufficient until approximately 2010, when another 1 MG of storage would be required. The second tank would likely be installed near Linstrang Way, or around the Foothills Subdivision, and could be developed prior to the development of the West Hill Road main, if needed. A proposed tank location is shown on Sheet 9.
- A third 1-MG storage tank is recommended for installation around 2016. This tank is recommended for installation on or near Mission Avenue near the proposed Thompson Drive Main. A proposed tank location is shown on Sheet 7.

- A fourth tank may be required by 2022, depending on population growth. A recommended tank location is on a hill (essentially the highest topographic point in the area) between Garden Park Drive and Highland Road. See Sheet 5.

Costs for each 1 MG storage tank are estimated at $1.6 million each, not including additional transmission main and access road costs. It is recommended that the City pursue the acquisition of property and easements for these tanks in the next several years, so that the land will be available in the future for these expansions. Table 9.2 provides a summary of the proposed tanks. Chart 9.1 shows existing and proposed water storage versus water and fire flows for 2006 through 2025, and is the basis for the recommended WSTs in Table 9.2.

Table 9.2 Proposed Water Storage Tanks

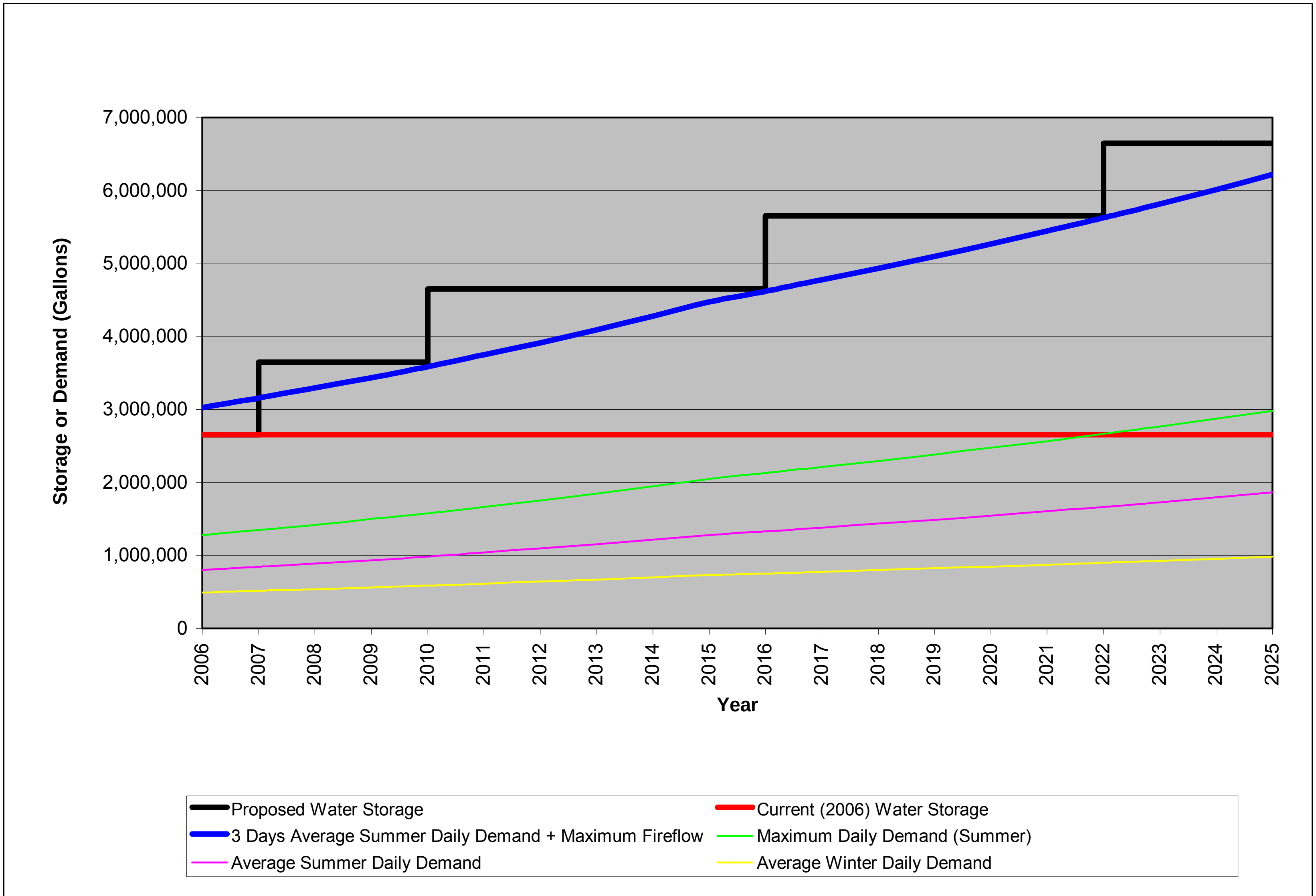
Phase	Proposed Location	Size (gallons)	Projected Year of Construction	Cost ($)
3.1	Crandall Addition Subdivision (1,500 LF transmission main required)	1,000,000	2008	1,800,000
3.2	Linstrang Way / Foothills Subdivision	1,000,000	2010	1,700,000
3.3	Mission Avenue	1,000,000	2016	1,700,000
3.4	Sprucewood Drive / Bay Ice Road	1,000,000	2022	1,700,000
			Total	**6,900,000**

The construction of additional WSTs could exacerbate the high disinfection byproducts (DPBs) and total trihalomethanes (TTHM) levels the City is experiencing, since additional contact time would be made available for the formation of these compounds. Therefore, the construction of any future WSTs would likely be completed during or subsequent to the development of treatment processes to reduce DPB/TTHM production. The WTP modifications recommended in Appendix A should reduce the organic loading to the system, therefore reducing the potential for DBP/TTHM formation.

9.2.4 Water Distribution System R&R Capital Improvement Schedule (Phase 4)

There are no sections of piping in the water system that are identified as needing immediate R&R. Recommendations for some water main replacements based on increased nominal domestic and fire flow demands are listed under the Phase 5 water improvements (Section 9.3). The 8-inch CI “west trunk” main is the only main where future deterioration of the pipeline could be a concern, primarily due to erosion and lateral movement of the bluff (and therefore the pipeline) between the Hilltop and A-frame PRV stations. The replacement of the West Trunk Main is also recommended to reduce the potential for air pockets to develop in this main during high demand conditions, and associated water hammer concerns. The increased main size will allow for higher flowrates into the distribution system. The construction of the new West Hill Road transmission main will reduce the dependence on the existing West Trunk Main.

Chart 9.1 Storage Versus Demand[1]



[1]Assumed maximum fireflow is 3 hour duration, 3,500 gpm flow (630,000 gallons total)

An estimate total of $50,000 has been included to cover biannual leak testing, to identify any water mains in need of repair or replacement. A total of $140,000 has been included to account for future main repairs and replacements.

It is expected that the majority of the existing PRV stations will require at least one rehabilitation over the next 10 to 20 years. A total of $1,500,000 is included to account for station rehabilitations and valve replacements for 13 of the existing 16 City PRV stations. Stations PR01, PR02, and PR11 are not anticipated to require major repairs or rehabilitations over the next 20 years.

Table 9.3 presents a list of 4-inch and 6-inch mains that are recommended for replacement mains during any road rehabilitation work that occurs within the City. These improvements are listed as Phase 4.5. The cost estimate in Table 9.4 assumes that replacements will consist of 8-inch mains. Refer to Table 9.4 for a listing of the water distribution system improvements discussed in this section.

Table 9.3 Existing 4-inch and 6-inch main replacements (Phase 4.5)

Street	Pipe Length (LF)
Elderberry Court	550
Bayview Court	240
Calhoun Court	260
Barnett Place	540
Latham Lane	570
Paintbrush Street	1,590
Raspberry Court	600
N. Larkspur Circle	470
Tamara Street	880
Shannon Lane	550
Triton Court	280
Clover Lane	1,350
Clover Place	260
Hillview Place	190
Sitka Rose Circle	380
Weber Subdivision	2,500
Rangeview	500
Fairview Avenue	2,000
Wright Street	330
Beluga Court	180
Iris Court	250
Aurora Court	160

Street	Pipe Length (LF)
Cityview	580
Gavin Court	270
Danview Avenue	1,900
Main off of Fairview	550
Lee Drive	1,300
Svedlund Street	730
Svedlund Circle	180
Klondike Avenue	1,300
Bonanza Avenue	1,300
Grubstake Avenue	1,300
Lucky Shot Street	820
Lakeside Court	180
Ben Walters Court	140
Pennock Street	1,060
Beluga Circle	460
Douglas Place	360
E Street	370
B Street	350
Main N of Shirlene Circle	350
Krueth Way/Seabreeze Ct.	930
Lake Street	820
Bay Avenue	1,790

Notes:
Above mains replaced with 8' diameter main
LF = Linear Feet

Table 9.4 Water Distribution System Replacement and Repairs

Phase	Improvement	Project Cost ($)	Construction Year /Timeline
4.1	Leak testing	50,000	Biannual
4.2	Water main repairs	140,000	Annual
4.3	PRV station rehabilitations	1,500,000	Varies
4.4	West Trunk replacement. Replace 8'' CI main from WTP to Meadows Avenue (2,140 LF) w/ new 16'' ID main. Install new PRV station [PR36] on West Terrace Blvd. Install 700 LF 16'' ID main on north West Terrace Blvd. Abandon existing west trunk main from Meadows Avenue to West Terrace Blvd.	611,000	2020-2025
4.5	4-inch and 6-inch main replacements (31,700 LF)	1,584,000	Varies
	Total	**3,885,000**	

Notes:

"	=	inches	LF	=	linear feet
ID	=	inner diameter	PRV	=	pressure reducing valve

9.2.5 Water Distribution System Expansion Schedule (Phase 5)

Proposed phasing for future WIDs are provided in this section. Water modeling information derived from H2Omap Water GIS (version 5.0) was used as the basis for determining main sizes and PRV locations.

Up to 24 new PRV stations would be installed in the future at the following locations:

- PR27 – Mariner Drive;
- PR28 – Sterling Highway / Bay Vista Place intersection;
- PR29 – Pineview Road / Sprucewood Drive intersection;
- PR30 – Tundra Rose Road;
- PR31 – West Hill Road (near Dewberry Lane);
- PR32 – West Hill Road (between Alpine Way and Wythe Way);
- PR33 –South Slope Road;
- PR34 – East Hill Road / Fireweed Avenue;
- PR35 – Mission Avenue (east of Willow Drive);
- PR36 – West Terrace Boulevard;
- PR37 – North end of Thompson Drive;
- PR38 – Thompson Drive (near East Road);

- PR39 – Sterling Highway / Mount Augustine Drive;
- PR40 – Mount Augustine Drive (between Judy Rebecca Court and Tanja Court);
- PR41 – Sterling Highway (between Watson Place and Mount Augustine Drive);
- PR42 – West Hill Road (between Claudia Street and Seascape Drive);
- PR43 – West Hill Road (above Linstrang Way);
- PR44 – Highland Drive (Bidarka Heights);
- PR45 – Crittenden Drive (near Sterling Highway intersection);
- PR46 – Meadow Drive;
- PR47 – East Road (directly east of East Road / Birch Lane intersection);
- PR48 – Scenic Place;
- PR49 – Foothills Subdivision; and
- PR50 – North end of Kallman Road.

Existing PRV outlet settings would be changed to the settings shown in Table 9.5. The outlet of PR10 (Bus Garage), PR11 (Jeep Sales), PR12 (Lucky Shot), PR13 (HEA), and PR14 (Lakeside), and the proposed new PRV at East Road (directly east of the East Road / Birch Lane intersection, connecting to Kachemak Drive) would be combined into one zone.

Table 9.5 New PRV Settings

PRV Name	PRV Location	Existing Inlet Setting (psi)	Existing Outlet Settings (psi)[2]	New Inlet Setting (psi)	New Outlet Setting (psi)[2]
PR01-Hilltop [old]	Hilltop [old]	75	25	Station replaced	
PR02-Midhill [old]	Midhill [old]	112	21	Station replaced	
PR01-Hilltop [new]	Hilltop [new]	New station		76	20
PR02-Midhill [new]	Midhill [new]	New station		117	21
PR03-A-frame	Dehel Street	112	49	106	36
PR04-Efflers	Diamond Willow Circle	86	25	89	25
PR05-Switchback	East Hill Road	96	26	109	26
PR06-Barnett	East Hill Road	145	45	150	45
PR07-Hoedel’s	East Hill Road	101	21	101	21
PR08-Kachemak	Kachemak Way	97	36	82	36
PR09-Main/Danview	Main And Danview	60	34	56	35
PR10-Bus Garage	Ohlson Lane	115	54	110	51
PR11-Jeep Sales	Main Street	94	33	TBD	TBD
PR12-Lucky Shot	Lucky Shot Street	104	44	101	42
PR13-HEA	Lake Street	97	36	96	36
PR14-Lakeside	Ben Walters Lane	95	41	97	37
PR15-Bear Creek	Early Spring Street	110	50	106	50
PR18-Bartlett	Bartlett Street	110	55	101	54
PR21-Ridgeline	Ridgeline Off Fireweed	113	50	113	50
PR25- Barcus	Fairview Avenue / West Hill Road intersection	87	30	79	32
PR26- Sterling Highway	Sterling Highway, between Watson Place and Saltwater Drive	84	26	75	28

Notes:
[1]Elevation of inlet pipe to PRV station, based upon survey information provided by the City of Homer Department of Public Works.
[2]PRV outlet pressures are flow dependent.
PRV = pressure reducing valve

There are 29 proposed phases for future WIDs as presented in Table 9.6.

Table 9.6 Water System Expansions

Phase	Water Improvement	Pipe Length (LF)	Main Size (Inches ID)	Project Cost ($)
5.1	**West Hill Road / Jeffery Avenue Extension**			
	Whispering Meadows Avenue	3,780	12	491,400
	Tundra Rose Road (Whispering Meadows Avenue to Jeffery Avenue)	750	12	97,500
	Jeffery Avenue (Tundra Rose Road to West Hill Road)	1,750	12	227,500
	West Hill Road (Jeffery Avenue to new PRV above Lindstrang Way)	7,170	12	932,100
	PRV [30] (Tundra Rose Road and Jeffery Avenue)	-	-	110,000
	PRV [42] (between Claudia Street & Seascape Drive on West Hill Road)	-	-	130,000
	PRV [31] (on West End Road, between Dewberry Lane and Highland Drive)	-	-	130,000
	PRV [32] (on West End Road near Alpine Way)	-	-	130,000
	PRV [43] (on West End Road, near Linstrang Way)	-	-	130,000
			Subtotal	2,379,000

Phase	Water Improvement	Pipe Length (LF)	Main Size (Inches ID)	Project Cost ($)
5.2	**West Hill Road Extensions**			
	Reber Road	900	8	103,500
	Claudia Street	535	8	61,500
	Wythe Way	760	8	87,400
	Bell Avenue	1,100	8	126,500
	Dewberry Lane	966	8	111,100
	Miller Lane	1,215	8	139,700
	De Graffenried Court	300	8	34,500
	Alpine Way	1,160	8	133,400
	Rosewood Court	370	8	42,600
	Linstrang Way	1,300	8	149,500
	Cheryl Lane	1,027	8	118,100
	Westwood Avenue (Tundra Rose to West Hill)	1,340	8	154,100
	Tundra Rose Road (Whispering Meadows Avenue - Westwood Avenue)	650	8	74,800
	Ero Court	400	8	46,000
	Pressure Sustaining Valve at Westwood Avenue / Westhill Road intersection	-	-	130,000
			Subtotal	1,513,000

Phase	Water Improvement	Pipe Length (LF)	Main Size (Inches ID)	Project Cost ($)
5.3	**Eagle View Drive and Surrounding**			
	Eagle View Drive (West Hill Road to Garden Park Drive)	2,320	12	301,600
	Eagle View Drive (east of Garden Park Drive)	1,250	8	143,800
	Diamond Creek Place	1,080	8	124,200
	Emerald Road (Natalie Circle To Jeffery Avenue)	1,270	10	158,800
	Emerald Road (south of Jeffery Avenue to Highland) – valve closed ~200 feet north of Highland Drive	740	8	85,100
	Jeffery Avenue (West Hill Road to Emerald Road)	1,740	10	217,500
	West Hill Road (Jeffery Avenue to Eagle View Drive)	850	8	97,800
	Goldberry Court	500	8	57,500
	West Hill Road (north of Eagle View Drive)	690	12	89,700
	Garden Park Drive (North Park Lane to Lincoln Circle)	1630	12	211,900
	Natalie Circle (Golden Park Drive to Highland Drive)	270 / 1,460	8/10	213,600
	Lincoln Drive	1,510	8	173,700
	Lincoln Circle	450	8	51,800
			Subtotal	1,927,000
5.4	**East Highland Drive**			
	Highland Drive (Jade Drive to West Hill Road)	2,450	10	306,300
	Emerald Court	310	8	35,700
	Emerald Road (partial – starting north of Highland Drive)	200	8	23,000
	Upland Court	426	8	49,000
	Jade Drive	329 / 742	10/8	130,600
			Subtotal	544,000
5.5	**Garden Park Drive & Highland Drive (Partial)**			
	Garden Park Drive (North Park Lane north to end of road)	676	8	77,700
	North Park Lane (east of Garden Park Drive)	503	8	57,800
	North Park Lane (Garden Park Drive to Sprucewood)	808	12	105,000
	Garden Park Drive (Lincoln Circle to Eagle View Drive)	1590	12	206,700
	Highland Drive (Jade Drive to Mountain Park Street)	3,237	10	404,600
	Mountain Park Street	563	10	70,400
	Bayridge Circle	500	10	62,500

Phase	Water Improvement	Pipe Length (LF)	Main Size (Inches ID)	Project Cost ($)
5.5 (cont.)	Sprucewood Drive (North Park Lane to Bayridge Circle)	200	10	25,000
	Bayridge Road (Sprucewood Drive to Mountain Park Street)	370	10	46,300
	South Park Circle	260	8	29,900
	Kelley Court	400	8	46,000
			Subtotal	1,132,000
5.6	**Foothills Subdivision, Lillian Walli Estate, & Saltwater Drive**			
	Robert Avenue	816	8	93,800
	Shelly Avenue	1,371	8	157,700
	Lillian Drive	364	8	41,900
	Soundview Avenue (north, from Shelly Ave intersection)	450	8	51,800
	Soundview Avenue (south, from Hydrant 110B)	430	8	49,500
	Unnamed Street – from Fairview Avenue to New Soundview Main	590	8	67,900
	Foothills PRV [49]	-	-	110,000
	Saltwater Drive	1,183	8	136,000
			Subtotal	708,000
5.7	**W.R. Benson's Subdivision**			
	East Bunnell Avenue	520	8	59,800
	Main Street (Bunnell Ave to Charles Way)	352	8	40,500
	Beluga Place	372	8	42,800
	Charles Way	1,018	8	117,100
			Subtotal	260,000
5.8	**Virginia Lyn Subdivision**			
	Dahl Way	1,030	8	118,500
	Mattox Road (Dahl Way to Virginia Lyn Way)	257	8	29,600
	Virginia Lyn Way	1,140	8	131,100
	Aurora Court to Pennock Street	586	8	67,400
	Pennock Street (existing main south to Ben Walters Lane)	760	8	87,400
			Subtotal	434,000

Phase	Water Improvement	Pipe Length (LF)	Main Size (Inches ID)	Project Cost ($)
5.9	**East Hill Rd to Mountain View Drive Extension**			
	South Slope Road (East Hill Road, south)	1940	10	242,500
	South Slope Road (to south end)	610	8	70,150
	Unnamed Street (from Anderson Street to South Slope Road)	1300	10	162,500
	Unnamed Street (east of Kallman Road)	430	8	49,800
	Unnamed Court	290	8	33,350
	Tasmania Court	680	8	78,200
	South Slope Road PRV [33]	-	-	110,000
	Anderson Street (Mountain View Drive to Elderberry Drive)	380	10	47,500
		1940	Subtotal	794,000
5.10	**Shellfish Avenue / Kallman Road**			
	Shellfish Avenue (Anderson Street to South Slope Road)	1660	8	190,900
	Anderson Street (south of Shellfish)	320	10	39,750
	Unnamed Street (from South Slope Road to Kallman Road)	800	10	100,000
	Kallman Road (north of Kramer Lane intersection)	630	10	78,750
	Anderson Street (Shellfish Drive to Mountain View Drive)	300	8	34,500
	Kallman Road PRV PRV [50]	-	-	110,000
			Subtotal	555,000
5.11	**Forget Me Not Lane / Paintbrush Court**			
	Forget Me Not Lane	1,300	10	162,500
	Paintbrush Court Booster Station	-	-	190,000
			Subtotal	340,000
5.12	**N. East Hill Road / Cottonwood Drive**			
	East Hill Road (from Fireweed Avenue)	580 / 776	12/8	205,300
	Race Road (East Hill Road to Janeview Drive) / (east of Janeview Drive)	2,167 / 426	12 / 8	330,700
	Janeview Drive	690	12	89,700
	Cottonwood Lane (east of Janeview Drive)	1,600 / 800	12/10	308,000
	Fireweed Avenue PRV [34]	-	-	110,000
			Subtotal	1,044,000

Phase	Water Improvement	Pipe Length (LF)	Main Size (Inches ID)	Project Cost ($)
5.13	**Mission Avenue**			
	Spruce Circle	236	8	27,100
	Willow Drive	916	8	105,300
	Rosebud Court (Willow Drive to Mission Avenue)	470	8	54,100
	Mission Avenue (Rosebud Court to Willow Drive)	285	8	32,800
	Mission Avenue (Willow Drive to eastern end)	5,190	10	648,800
	Hillslope Court	450	8	51,800
	PRV [35] (Mission Avenue)	-	-	110,000
	Mission Avenue to East End Road (Thompson Drive)	2,900	10	362,500
	PRV [37] (north Thompson Drive)	-	-	110,000
	PRV [38] (lower Thompson Drive)	-	-	110,000
	East End Road (Rosebud Court to Mission Road)	473	8	78,000
			Subtotal	1,690,000

Phase	Water Improvement	Pipe Length (LF)	Main Size (Inches ID)	Project Cost ($)
5.14	**Scenic View Drive & Other**			
	Scenic View Drive (East Road to Spruce Lane)	747	8	85,900
	Adams Drive (East Road to Spruce Lane)	863	8	99,300
	Adams Drive (south of Spruce Lane)	587	8	67,500
	Spruce Lane (Scenic View Drive to Adams Drive)	556	8	64,000
	Tilly Court	241	8	27,700
	Starboard Way (West)	306	8	35,200
	Williams Place	560	8	64,400
	PRV [27] Mariner Drive	-	-	110,000
			Subtotal	537,000

Phase	Water Improvement	Pipe Length (LF)	Main Size (Inches ID)	Project Cost ($)
5.15	**Sterling Hwy (Lake Street to Greatland Street) & Misc.**			
	Sterling Highway (Greatland Street to Poopdeck Street)	1,870	12	336,600
	Sterling Highway (Heath Street to Lake Street)	1,200	12	216,000
	Poopdeck Street (Bonanza Avenue to Hazel Avenue)	581	8	66,800
	Woodside Avenue	534	8	77,400

Phase	Water Improvement	Pipe Length (LF)	Main Size (Inches ID)	Project Cost ($)
5.15 (cont.)	Lee Drive (Kachemak Way to Heath Street)	661	8	76,000
	Crittenden Drive PRV [45]	-	-	130,000
			Subtotal	903,000
5.16	**Kachemak Drive**			
	Kachemak Drive Phases I-III (includes Beach Road)	14,700	12	2,646,000
	PRV Installation (East End Road, west of Birch Lane) [47]	-	-	140,000
			Subtotal	2,786,000
5.17	**Parson / Cape Douglas**			
	Parson Lane	770	8	88,600
	Cape Douglas Way	553	8	63,600
			Subtotal	152,000
5.18	**Tietjen Subdivision**			
	Meadow Drive	1,300	12	169,000
	Meadow Drive PRV [46]	-	-	110,000
	Little Fireweed Lane (East End Road to Eagle Place)	547	8	62,900
	Little Fireweed Lane (Eagle Place to Ternview Place)	637	8	73,300
	Eagle Court	362	8	41,600
	Eagle Place (Little Fireweed Lane to Spruce Lane)	1,300	8	149,500
	Spruce Lane (Alder Lane to Meadow Drive)	985	10	123,100
	Spruce Lane (Meadow Drive to Eagle Place)	990	8	113,900
	Spruce Lane (Eagle Place to Ternview Place)	640	8	73,600
	Spruce Lane (west of Alder Lane)	770	8	88,600
	Clover Lane (connection to Spruce Lane)	170	8	19,600
	Helen Circle	500	8	57,500
	Alder Lane (Spruce Lane north towards East End Road)	694 / 314	10/8	122,900
	Alder Lane (Spruce Lane to Bottom Lane)	1,300	8	149,500
	Moon Lane	1,300	8	149,500
	Star Lane	1,300	8	149,500
	Eagle Place (Spruce Lane to Bottom Lane)	1,300	8	149,500

Phase	Water Improvement	Pipe Length (LF)	Main Size (Inches ID)	Project Cost ($)
5.18 (cont.)	Ternview Place (Spruce Lane to Bottom Lane)	1,300	8	149,500
	Bottom Lane (Alder Lane to Ternview Place)	2,620	8	301,300
	Ternview Place (Spruce Lane to Little Fireweed Lane)	1,300	8	149,500
			Subtotal	2,404,000
5.19	**Spencer Drive / Larry Lane**			
	Spencer Drive	1,141	8	131,200
	Larry Lane	1,050	8	120,800
			Subtotal	252,000
5.20	**Tulin Terrace Subdivision**			
	West Terrace Boulevard (southern end)	1,019	16	163,000
	Meadow Court	600	8	87,000
	Whisper Court	374	8	54,200
	Tulin Bluff	480/480	8/12	146,400
			Subtotal	481,000
5.21	**W. Highland Drive / Sprucewood Drive**			
	Highland Drive (Mountain Park Street to Rogers Loop)	4,150	8	477,300
	Sunnyside Drive	1035	8	119,000
	Sprucewood Drive (North Park Lane to Rogers Loop)	4700	10	587,500
	Sprucewood Drive PRV [29]	-	-	110,000
	Bayridge Road	2,987	8	343,500
	Rogers Loop (Sprucewood Drive to Highland Drive)	427	10	53,400
	Highland Road PRV [44]	-	-	110,000
			Subtotal	1,801,000
5.22	**Sterling Highway**			
	Sterling Highway (Mount Augustine Drive to Hillside Place Extension)	3,470	12	624,600
	Sterling Highway (Mount Augustine Drive to Rogers Loop)	3,240	12	583,200
	Rogers Loop [Sterling Highway (east) to Highland Drive]	573	10	71,600
	Sterling Highway PRV (near Bay Vista Place) [28]	-	-	140,000

Phase	Water Improvement	Pipe Length (LF)	Main Size (Inches ID)	Project Cost ($)
5.22 (cont.)	Sterling Highway PRV (near Mt Augustine Drive) [39]	-	-	140,000
	Sterling Highway PRV (near Hunter Street) [41]	-	-	140,000
			Subtotal	1,699,000
5.23	**W. Sterling Highway / W. Rogers Loop**			
	Rogers Loop [Sterling Highway (west) to Spucewood Drive]	1,000	10	125,000
	Sterling Highway (western City Boundaries to east Rogers Loop)	7,190	12	1,294,200
			Subtotal	1,419,000
5.24	**Mount Augustine Subdivision**			
	Mount Augustine Subdivision Mains (Mount Augustine Dr. to Saltwater Dr.)	3,021	8	347,400
	Mount Augustine Subdivision PRV [40]	-	-	110,000
			Subtotal	457,000
5.25	**Kachemak City**			
	Bear Creek Drive (northeast Bench Circle to Birch Lane)	772	12	100,400
	Nordby Avenue	1,540	8	177,100
	Birch Lane (Nordby Avenue to Bear Creek Drive)	987	12	128,300
	Birch Lane (East End Road to Nordby Avenue)	752	10	94,000
	Bear Creek Drive (Bear Creek Court to southwest Bench Circle)	771	12	100,200
	Bear Creek Drive (East End Road to Bear Creek Court)	1,240	8	142,600
	Bryant Court	420	8	48,300
			Subtotal	791,000
5.26	**Skyline Drive**			
	Pumping Facility / Pressure Tanks	-	-	400,000
	Skyline Drive	10,251	12	1,640,200
	Ridgeway Court	907	8	99,800
	Tulin Terrace Boulevard	2,050	8	297,300
	Crestwood Circle	1,600	8	176,000
	Spruce Terrace Circle	350	8	40,300
	Glacier View Court	1088	8	125,100

Phase	Water Improvement	Pipe Length (LF)	Main Size (Inches ID)	Project Cost ($)
5.26 (cont.)	Scenic Place	1,580 / 821	10/8	291,900
	Horizon Court	1,400	8	161,000
	Scenic Place PRV [48]	-	-	110,000
	Hemlock Street	200	8	23,000
			Subtotal	3,364,000
5.27	**South Peninsula Hospital Main**	1,500	12	195,000
5.28	**Hydrant Installations** (53 total)	-	-	270,000
5.29	**High School Fire Flow Improvements**	1,000	16	140,000
			Total	**31,411,000**

Notes:

ID = Inner Diameter

LF = Linear Feet

PRV = pressure reducing valve

9.3 SEWER SYSTEM CAPITAL IMPROVEMENT PROGRAM

9.3.1 Wastewater Treatment Plant Capital Improvement Schedule (Phase 6)

Details of the recommended WWTP improvements are provided in Appendix B.

It is recommended that the City pursue acquisition of the property adjacent to the WWTP for future plant expansions (see Sheets 33A and 33B). This acquisition would be made to ensure adequate land is available for future WWTP expansions.

9.3.2 Sewer Collection System R&R Capital Improvement Schedule (Phase 7)

There are no sections of sewer main that are identified as needing R&R at this time. Specific sections of the collection system will probably require replacement once further monitoring and testing have been conducted to identify the problem areas for I/I. Table 9.7 provides a very preliminary listing of potential actions the City can undertake to better identify I/I problems and ultimately prioritize corrective work.

Table 9.7 Corrective Actions – System I/I (Phase 7.1)

Action	O&M Cost ($)
TV lines	40,000
Flow Monitoring (for infiltration)[1]	50,000
Smoke testing	25,000
Manhole repairs	20,000 – 50,000
Foundation/roof drain removal	15,000 – 80,000
Total	150,000 – 255,000

Notes:
[1]May involve mandated property owner corrective work.
O&M = operation and maintenance

Sewer main R&Rs are presented in Table 9.8 for Phases 7.2 through 7.8. As discussed in Section 8.6, resizing requirements for mains was determined by a computer model of the wastewater collection system. Resizing of single pipes was chosen over installation of second parallel mains due to cost considerations. It was assumed that this option would reduce the potential for infiltration, replace potentially deteriorating mains, remove protection requirements for existing excavated pipes, and reduce costs through a reduction in trench excavation requirements.

Table 9.8 Sewer Main Replacement and Repairs (Phases 7.2 – 7.8)

Phase	Improvement	Pipe Length (LF)	Existing Main Size (inches)	Future Main Size (inches)	Project Cost ($)	Replacement Required by 2025?
7.2	Sterling Hwy (Thomas Street to Waddell Loop)	300	8	12	58,500	Yes
7.3	East line entering WWTP	2,000	12	24	420,000	Yes
7.4	Bunnell Avenue (Greatland to Beluga)	1,150	8	12	224,250	Possible
7.5	Sterling Hwy (Soundview – Thomas)	1,300	10	16	273,000	Possible
7.6	Mattox Road to Lake Street	3,700	12	24	666,000	Possible
7.7	East trunk to WWTP[1]	400 (est.)	24	> 24	72,000	Possible
7.8	Ocean Drive (survey required)[1]	400 (est.)	10	≥ 12	78,000	Possible
				Total	1,792,000	

Notes: [1]Survey information required to verify slope, and replacement requirements.
Est. = estimated LF = linear feet
Hwy = highway WWTP = wastewater treatment plant

9.3.3 Sewer Collection System Expansion Schedule (Phase 8)

Proposed sewer line expansions will generally follow the layouts shown on Sheets 16 to 30. Sewer lines will be constructed of HDPE SDR 11 pipe, and will be 8-inch ID (nominal), unless otherwise specified. Sewer layouts were designed assuming a minimum slope of 0.50 vertical feet per 100 horizontal feet, and a minimal burial depth of 8 feet below ground surface. In some instances, sewer mains will need to be buried at depths less than 8 feet (but not less than 4.5 feet) below grade. In such cases, insulation will be required. Collection lines will be connected to standard 4-foot concrete manholes. A list of the proposed system expansions is provided in Table 9.9. Sheets 33A and 33B provides a schematic of the recommended expansions to the WWTP.

Sewer layouts were designed to avoid crossing private property, where feasible. In some instances, the requirement to avoid deep excavations required the crossing of private property boundaries. In these instances, easements will be required during the design phase. Some lots along the gravity collection main may require small wastewater pump stations at individual services because of the steep topography of the lot.

Future O&M costs for the sewer distribution system are expected to be moderately higher than existing O&M costs due to aging mains. It is expected that an increased emphasis will be placed on the disconnection and monitoring for illegal service connections.

Table 9.9 Sewer System Expansions

Phase	Sewer Improvement	Pipe Length (LF)	Main > 15-foot burial depth (LF)	Insulation (LF)	Total Cost ($)
8.1	**West Hill Road (Middle)**				
	De Graffenried Court	248	-	-	32,240
	Seascape Drive, Wolf Way, lower Crestline Street, and Linstrang Way	4,400	-	-	572,000
	West Hill Road (Linstrang Way to Hillside Place)	310	-	-	49,600
	West Hill Road (Highland Drive to Seascape Drive)	3,400	-	-	442,000
	Claudia Street	600	-	-	78,000
				Subtotal	1,174,000
8.2	**Tundra Rose Road to West Hill Road / Highland Drive**				
	West Hill Road (Bell Avenue to Highland Drive)	700	-	-	112,000
	Tundra Rose Road (Westwood Ave to Rosewood Court main)	1,200	100	50	160,000
	Rosewood Court to Tundra Rose Road	1,400	-	-	182,000
	Tundra Rose Road to Bell Avenue	1,600	-	-	208,000
	Bell Avenue	1,100	500	-	158,000
	Westwood Estates ROW to Tundra Rose Road	600	-	-	78,000
	ROW between Jeffery Avenue and Bell Avenue, connecting to Tundra Rose Road/Bell Avenue Main	600	-	-	78,000
	Dewberry Lane	600	-	-	78,000
				Subtotal	1,054,000
8.3	**West Hill Road (Upper)**				
	West Hill Road (Westwood Avenue to Bell Avenue)	2,700	-	-	432,000
	Eagleview Drive, Jeffery Avenue	2,000	-	-	260,000
	Westwood Avenue (west)	1,100	-	-	143,000
	Goldberry Court	500	-	-	65,000
				Subtotal	900,000

Phase	Sewer Improvement	Pipe Length (LF)	Main > 15-foot burial depth (LF)	Insulation (LF)	Total Cost ($)
8.4	**West Hill Road Extensions**				
	Upland Court / Wythe Way (Highland Drive to West Hill Road)	2,200	50	-	287,500
	Reber Road, upper Crestline Street	1,200	-	-	156,000
	West Hill Road (Switchback to Linstrang Way)	1,900	-	-	304,000
	Fairview Avenue(partial) from West Hill Road	250	-	-	32,500
	Cheryl Lane	850	-	-	110,500
				Subtotal	891,000
8.5	**Bell Subdivision**				
	Bell Subdivision to Fairview Avenue (Miller Lane, Alpine Lane, unnamed ROW)	5,000	-	-	650,000
	Fairview Avenue (from Bell Subdivision to Foothills Subdivision)	820	-	-	106,600
	Foothills Subdivision (Fairview Avenue to Soundview Avenue)	1,600	-	-	208,000
				Subtotal	965,000
8.6	**East Road / Christiansen Tracts**				
	East Road (from Christiansen Tracts to Mariner Drive)	1,400	-	-	252,000
	Christiansen Tracts (unnamed street)	1,050	-	-	136,500
	Christensen Tracts (future unplatted streets)	2,500			325,000
				Subtotal	714,000
8.7	**Mount Augustine Drive**				
	Mount Augustine Subdivision (Mount Augustine Drive, Judy Court, Ursula Avenue, Tanja Court, Baycrest Ave, Saltwater Drive) to Sterling Highway	9,900	400	2,150	1,342,000
8.8	**Sterling Highway (Phase I)**				
	Sterling Highway (Mount Augustine Drive to Watson Drive)	2,200	-	-	396,000
	Coyote Way, Hunter Street	1,400	-	-	182,000
	Watson Drive	500	-	-	65,000
				Subtotal	643,000

Phase	Sewer Improvement	Pipe Length (LF)	Main > 15-foot burial depth (LF)	Insulation (LF)	Total Cost ($)
8.9	**Lillian Walli Estate Subdivision**				
	Fairview Avenue (partial), Ero Court, Shelly Avenue, ROW to Soundview Avenue	3,800	-	-	494,000
	Robert Avenue, Lillian Drive (to Sitka Rose Circle)	1,900	100	-	250,000
				Subtotal	744,000
8.10	**Mattox Subdivision**				
	Cook Way	750	-	-	97,500
	Dahl Way	900	-	-	117,000
	Lynn Way	650	-	-	84,500
	Mallard Way	650	-	-	84,500
	ROW north of Lynn Way and south of Mallard Way	650	-	-	84,500
				Subtotal	468,000
8.11	**Benson Subdivision** (Main Street, Bunnell Avenue (east), Beluga Place, and Charles Way)				
	Gravity Mains	2,350	150	100	382,500
	Force Main	360	-	-	36,000
	Lift Station	-	-	-	175,000
				Subtotal	594,000
8.12	**Kachemak Drive (Phases I-III)**				
	Kachemak Drive Phase I – Gravity	4,500	-	600	822,000
	Kachemak Drive Phase II – Gravity	2,800	-	800	520,000
	Kachemak Drive Phase II – Lift Station	-	-	-	175,000
	Kachemak Drive Phase II – Force Main (4-inch)	50	-	-	7,500
	Kachemak Drive Phase III – Gravity (includes Beach Road)	9,800	-	100	1,766,000

Phase	Sewer Improvement	Pipe Length (LF)	Main > 15-foot burial depth (LF)	Insulation (LF)	Total Cost ($)
8.12 (cont.)	Kachemak Drive Phase III – Lift Station	-	-	-	175,000
	Kachemak Drive Phase III – Force Main (through Davis Street)	2,100	-	-	336,000
				Subtotal	3,802,000
8.13	**Paradise South Subdivision/Scenic View Subdivision** (Paradise Place, Upland Place, Carson Place, Spencer Drive)	4,000	-	-	520,000
8.14	**Mission Road / Eker Estates Subdivision**				
	Cottonwood Lane, Janeview Drive, Race Road to Mission Road	4,100	200	-	539,000
	Mission Avenue	5,300	-	-	689,000
	Thompson Drive (Mission Avenue to East Road)	2,800	-	-	364,000
				Subtotal	1,592,000
8.15	**Highland Drive**				
	Highland Drive (~700 feet NE of Kelley Court to new lift station West Hill Road) + 400 LF of main east of new Lift Station	3,300	-	-	429,000
	Highland Drive Force Main to West Hill Road (4-inch)	1,200	-	-	120,000
	Lift Station	-	-	-	175,000
				Subtotal	724,000
8.16	**Natalie Woods / Emerald Highland Estates**		-	-	
	Garden Park Drive (partial) and Lincoln Drive to Natalie Circle	2,200	-	-	286,000
	ROW between Garden Park Drive and Lincoln Drive connecting to Natalie Circle	615	-	-	79,950
	Natalie Circle to Highland Drive	1,500	-	-	195,000
	Emerald Road (to Highland Drive)	1,400	-	-	182,000
				Subtotal	743,000
8.17	**Eagleview Subdivision**				
	Force Main to Emerald Road / Natalie Circle Intersection	1,620	-	-	162,000

Phase	Sewer Improvement	Pipe Length (LF)	Main > 15-foot burial depth (LF)	Insulation (LF)	Total Cost ($)
8.17 (cont.)	Lift Station	-	-	-	175,000
	Diamond Creek Place	2,700	-	-	351,000
	Eagle View Drive to new Eagle View Drive Lift Station	2,100	100	100	278,000
	North Park Lane, Garden Park Drive (west), Lincoln Circle to new main from Eagle View Drive	3,400	-	-	442,000
				Subtotal	1,408,000
8.18	**Mountain Park Subdivision** [Highland Drive (west), Sunny Side Drive, South Park Circle, Bayridge Circle, Mountain Park Street] to Baycrest Subdivision	7,200	-	-	936,000
8.19	**Willow Drive/Spruce Circle to Larkspur Court**	1,650	-	-	215,000
8.20	**Bayview Gardens Subdivision**				
	Mountain Ash Street, Hemlock Street, and Skyline Drive (partial) to Paintbrush Court / Paintbrush Street	2,200	-	-	286,000
	Paintbrush Court / Painbrush Street to Fireweed Avenue	3,050	-	-	397,000
	Fireweed Avenue (west) to East Hill Road [Fireweed Avenue (west), Willow Circle, Rosebud Court]	3,900	100	-	627,000
	ROW connecting to Diamond Willow Circle	400	-	-	52,000
	Ridgeway Court, Forget-Me-Not Lane, Glacier View Court (west)	3,500	100	50	459,000
	East Hill Road (Rosebud Court to existing main south of Rosebud Court)	200	-	-	36,000
				Subtotal	1,857,000
8.21	**East Hill Road**				
	East Fireweed Avenue to East Hill Road	1,350	-	-	175,000
	East Hill Road (~ 800 feet southwest of Skyline Drive to ~300 feet northeast of Fireweed Avenue)	2,200	-	-	396,000

Phase	Sewer Improvement	Pipe Length (LF)	Main > 15-foot burial depth (LF)	Insulation (LF)	Total Cost ($)
8.21 (cont.)	East Hill Road (~300 feet northeast of Fireweed Avenue to Rosebud Court)	1,300	-	-	234,000
	East Glacier View Court to East Hill Road + ROW	1,900	-	-	247,000
				Subtotal	1,053,000
8.22	**Tulin Terrace**				
	Tulin Terrace Blvd, Spruce Terrace Circle, & Pine Terrace Circle to Whispering Meadows Avenue	4,900	-	-	637,000
	Whispering Meadows Avenue	3,000	-	-	390,000
	6" Force Main (Whispering Meadows to Fireweed)	2,400	-	-	264,000
	Whispering Meadows Avenue Lift Station	-	-	-	175,000
	Crestwood Circle (to Whispering Meadows Avenue)	1,400	-	-	182,000
	Westwood Estates ROWs (to Whispering Meadows Avenue)	900	-	-	117,000
				Subtotal	1,765,000
8.23	**West Terrace Boulevard**				
	Gravity Mains (Whisper Court, West Terrace Boulevard, Tulin Bluff Court, Meadow Court)	4,800	300	-	633,000
	Force Main	2100	-	-	273,000
	Lift Station	-	-	-	175,000
				Subtotal	1,081,000
8.24	**Vineyard Estates**				
	Latham Lane (to Alm Lane)	530	-	-	68,900
	ROW south of Latham Lane (to Alm Lane)	510	-	-	66,300
				Subtotal	135,200
8.25	**South Slope Road** (to Heidi Court) + Tasmania Court, and two ROWs	3,700			481,000

Phase	Sewer Improvement	Pipe Length (LF)	Main > 15-foot burial depth (LF)	Insulation (LF)	Total Cost ($)
8.26	**East End Road Extensions**				
	Hopkins/Williams Place to East Road	2,200	100	-	289,000
	Scenic View Drive, Tilly Court, Spruce Lane (west)	1,200	50	-	157,500
	Shannon Lane	550	-	-	71,500
	Starboard Way (west)	470	-	-	61,100
	Mary Allen/Slavin Drive	1,650	-	-	214,500
				Subtotal	794,000

Phase	Sewer Improvement	Pipe Length (LF)	Main > 15-foot burial depth (LF)	Insulation (LF)	Total Cost ($)
8.27	**Tietjen Subdivision**				
	Gravity Mains [Ternview Place, Bottom Lane, Eagle Place (south), Star Lane, Moon Lane, Alder Lane (south), Spruce Lane (east)]	9,900	-	-	1,287,000
	4" Force Main (Bottom Lane and Alder Lane)	3,600	-	-	360,000
	Lift Station (Bottom Lane/Ternview Place)	-	-	-	175,000
	Gravity Mains [Meadow Drive, Spruce Lane (west), Eagle Place (north), Eagle Court, Helen Circle]	4,500	500	-	600,000
				Subtotal	2,422,000

Phase	Sewer Improvement	Pipe Length (LF)	Main > 15-foot burial depth (LF)	Insulation (LF)	Total Cost ($)
8.28	**West Highland Drive**				
	Highland Drive (from Rogers Loop to new Lift Station)	1,600	300	-	217,000
	Highland Drive Lift Station	-	-	-	175,000
	Highland Drive Force Main	1,300	-	-	221,000
	Highland Drive (Sunny Side Drive to Lift Station)	1,100	-	-	143,000
				Subtotal	756,000

Phase	Sewer Improvement	Pipe Length (LF)	Main > 15-foot burial depth (LF)	Insulation (LF)	Total Cost ($)
8.29	**Sterling Highway (Phase II)**				
	Sterling Highway (Rogers Loop east to Mount Augustine Drive	3,500	-	-	630,000
	Rogers Loop (east, from Lift Station to Sterling Highway)	1050	-	-	136,500
	Sterling Highway (from ~ 1,200 feet east of west Rogers Loop to east Rogers Loop)	1,500	-	-	270,000
				Subtotal	1,037,000

Phase	Sewer Improvement	Pipe Length (LF)	Main > 15-foot burial depth (LF)	Insulation (LF)	Total Cost ($)
8.30	**East Rogers Loop**				
	Rogers Loop (east, to Lift Station near Sprucewood)	950	-	-	123,500
	Rogers Loop Force Main	300	-	-	39,000
				Subtotal	163,000

Phase	Sewer Improvement	Pipe Length (LF)	Main > 15-foot burial depth (LF)	Insulation (LF)	Total Cost ($)
8.31	**Sprucewood Drive**				
	Sprucewood Drive (east, to Sprucewood Drive Lift Station)	500	-	-	65,000
	Sprucewood Drive (west, to Sprucewood Drive Lift Station)	3,700	450	100	496,500
	Sprucewood Drive Force Main	600	-	-	102,000
	Sprucewood Drive Lift Station	-	-	-	175,000
				Subtotal	839,000

Phase	Sewer Improvement	Pipe Length (LF)	Main > 15-foot burial depth (LF)	Insulation (LF)	Total Cost ($)
8.32	**Bayridge Road**				
	Bayridge Road (to Sprucewood Drive)	900	-	-	117,000
	Bayridge Road (to Bayridge Road Lift Station)	1,500	-	-	195,000
	Bayridge Road force main	500	-	-	50,000
	Bayridge Road lift station	-	-	-	175,000
				Subtotal	537,000

Phase	Sewer Improvement	Pipe Length (LF)	Main > 15-foot burial depth (LF)	Insulation (LF)	Total Cost ($)
8.33	**Sterling Highway (Phase III)**				
	Rogers Loop (west, to Sterling Highway)	900	-	-	117,000
	Sterling Highway (west, traveling east to new Lift Station ~ 2,000 feet west of Rogers Loop)	2,600	-	-	468,000
	Lift Station (~ 2,000 feet west of Rogers Loop)	-	-	-	175,000
	6" Force Main (from ~ 2,000 feet west of west Rogers Loop to ~ 1,200 feet east of west Rogers Loop	3,200	-	-	512,000
	Sterling Highway (from ~ 1,100 feet east of west Rogers Loop to ~ 2,000 feet west of west Rogers Loop)	3,100	-	-	558,000
				Subtotal	1,830,000

Phase	Sewer Improvement	Pipe Length (LF)	Main > 15-foot burial depth (LF)	Insulation (LF)	Total Cost ($)
8.34	**Lake Street/Sterling Highway Force Main**				
	Sterling Highway – redundant force main (ending at Lake Street) – 4-inch ID assumed	900	-	-	135,000
8.35	**Skyline Drive (eastern end)**				
	Skyline Drive (eastern end)	3,000	-	-	540,000
	Lift Station	-	-	-	190,000
	4" Force Main	500	-	-	75,000
				Subtotal	805,000
8.36	**Scenic Place/Horizon Court**				
	Scenic Place & Horizon Court	2,913	-	-	466,080
	Skyline Drive (partial)	513	-	-	92,340
	Horizon Court to Carson Place	1,936	-	-	484,000
				Subtotal	1,042,000
8.37	**Garden Park Drive (North end)**				
	Gravity Mains	470	-	-	75,200
	Lift Station	-	-	-	150,000
	4" Force Main	540	-	-	81,000
				Subtotal	306,000
				Total	36,467,000

Notes:

~	=	approximately	LF	=	linear feet
>	=	greater than	NE	=	northeast
ID	=	inner diameter	ROW	=	right-of-way

9.4 COST ESTIMATE CRITERIA

Cost estimates presented in this WSMP are approximate estimates made without detailed engineering information. Order-of-magnitude estimates were incorporated into these estimates, as detailed soils information, existing utilities, seasonal construction timeframe, and other specific engineering information was not available. All construction and O&M costs are on a current-worth (2006) basis. A 20-year planning period was used for analyzing project alternatives and costs. Replacement of pumps and treatment facility equipment are the major maintenance costs expected over the 20-year planning period. The following service life is assigned to equipment and piping:

- Piping and Appurtenances – 25 years minimum
- Structures and Tanks – 30 years
- Pumps – 10 years
- Treatment Process Equipment – 10 years

Table 9.10 presents cost estimates for standard water and sewer main equipment installations in Homer. Construction cost estimates for pipelines are based on a unit cost per linear foot of pipe installed. The costs were adjusted based on the type of road material the piping would be installed under or along, constructed and burial depth. Cost estimates did not incorporate detailed information for right-of-way acquisitions.

The basis of these prices include a recent water and sewer project completed by the HDPW, as well as current project costs in Anchorage. Smaller projects will typically have higher installation costs per LF (and vice versa). Water and sewer main LF costs include costs for shutoff valves, manholes, and cleanouts.

9.5 FUTURE ENVIRONMENT WITH AND WITHOUT IMPROVEMENTS

Numerous benefits would result from the proposed expansions to the City's water distribution and wastewater collection systems. The following is a list of the more immediate benefits that would result from the improvements recommended by this WSMP:

- Community health conditions will improve.
- Problems with contaminated wells, low-yield wells, and potential waterborne diseases will be eliminated.
- The need for residents to haul and store water will be reduced. The potential for loss of disinfection residual, or the development of DBPs in private storage tanks will be reduced.
- Malfunctioning septic tanks could be abandoned.
- Fire protection will improve.
- Service pressures will increase.
- Water quality will improve.

Table 9.10 Estimated Water and Sewer Main Installation Costs (2006)

Installation	Unit Cost	Gravel Roads	Paved Roads	Paved “Major” Road
Water Main				
6-inch HDPE Water Main	$/LF	110	140	160
8-inch HDPE Water Main	$/LF	115	145	165
10-inch HDPE Water Main	$/LF	125	155	175
12-inch HDPE Water Main	$/LF	130	160	180
PRV Station - Standard	$/each	110,000	130,000	140,000
Sewer Main				
8-inch DIP Gravity Sewer Main	$/LF	130	160	180
12-inch DIP Gravity Sewer Main	$/LF	145	175	195
16-inch DIP Gravity Sewer Main	$/LF	160	190	210
24-inch DIP Gravity Sewer Main	$/LF	180	210	230
Sewer Lift Station (large)	$/each	250,000	300,000	350,000
Sewer Lift Station (small)	$/each	150,000	170,000	190,000
4-inch HDPE Sewer Force Main	$/LF	100	130	150
6-inch HDPE Sewer Force Main	$/LF	110	140	160
8-inch HDPE Sewer Force Main	$/LF	115	145	165
Insulation	$/LF	20		
Burial Depth > 15 feet below surface (additional cost)	$/LF	30		
Outfall Air Relief Valve Station – Standard	$/each	60,000		

Notes:

$/LF	=	dollars per linear foot	HDPE	=	high-density polyethylene
>	=	greater than	PRV	=	pressure reduction valve
DIP	=	ductile iron pipe			

- The reliability of water quality and supply for areas of town that are currently serviced will increase.
- Fire insurance rates for homeowners will be lower. The ISO ratings in Homer for properties within 1,000 feet of a hydrant are currently "3" for properties served by hydrants and "8" for properties in surrounding areas. Those areas not on the water system would be upgraded to lower ISO ratings, if water service upgrades were to occur.
- Areas of expansion that are not currently served would meet home lender's requirements, allowing the resale and development of residential and commercial properties in the study area. Homeowners with malfunctioning septic tanks or wells could have difficulties if they wished to sell their homes or businesses. Most lending agencies will not approve home loans without an engineered and approved septic system, and sometimes a new well. Costs for approved septic system and well installations can often be expensive. Compounding this problem is the fact that many areas of town where septic systems are currently installed often have poorly functioning or failing systems.

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10.0 IMPLEMENTATION AND FUNDING

10.1 IMPLEMENTATION

The City and the community maintain responsibility for implementing the recommended improvements. ADOT&PF and the City retain authority over expansions within the boundaries of the airport.

10.2 SUMMARY OF PROJECT PRIORITIES

It is important that the proposed water and wastewater improvements be given a high priority by the City and its residents if these projects are to be eligible for state and federal grants and funding. As a practical matter, it would be exceedingly difficult to undertake the proposed improvements all at once. A project priority listing should be prepared annually by the City. This resolution will be submitted, along with the funding grant request, to appropriate state and federal agencies for consideration in prioritizing funding for each fiscal year.

Appendix D provides a recommended conceptual schedule for future water and wastewater projects. This schedule is not meant to serve as a compulsory timeline for construction projects, but rather is meant to serve as a guideline for future planning efforts.

10.3 FUNDING SOURCES

Currently, homeowners contribute 50 percent of the capital costs for expansion of the water and sewer systems, with the remaining 50 percent paid for by the Homer Accelerated Water and Sewer Program. This program does not apply to subdivisions formed after June 28, 1999. Future capital costs will likely be funded primarily (or completely) by homeowners in their respective Local Improvement District.

Potential funding for future public water and sewer projects in the State of Alaska include, but may not be limited to:

- ADEC grants and/or revolving loan funds. The interest rate for revolving loan funds is currently set at an annual rate of 2.5 percent for water, and 1.5 percent for wastewater.
- ADOT&PF.
- Alaska Legislative Direct Appropriation Grant(s).
- Municipal Bond Issue (includes the Alaska Municipal Bond Bank and Private Sector Bonding).
- Rural Utilities Service (RUS) Water and Environmental Programs (administrated by the U.S. Department of Agriculture). These include federal grant/loan programs, including Farmer's Home Administration programs available to communities with fewer than 10,000 residents. Low interest loans are made available for water and waste disposal systems. Public bodies and not-for-profit corporations are eligible. Funds may also be used for solid waste disposal and storm drainage systems, as well as training. Some applicants may qualify for grant funds to supplement a loan. U.S.

Department of Agriculture Rural Development staff in Alaska can be contacted at (907) 761-7705.

- The U.S. Environmental Protection Agency and RUS provide funding to organizations that provide training and technical assistance to small water systems. These organizations include the National Rural Water Association, the Rural Community Assistance Program, and the National Drinking Water Clearinghouse.

10.4 Permits and Approvals

There are several permits and approvals that would be required prior to the construction of the recommended facilities. These permits and approvals include, but may not be limited to:

- ADEC plan review, approval to construct and operate water and sewer improvements;
- ADNR approval for construction activities that cross streams and other water bodies;
- ADOT&PF approval for work in state rights-of-way (ROWs) and road crossings;
- Federal Aviation Administration (FAA) approval for work in the boundaries of the airport and its vicinity;
- USACE permits for work in shorelines or wetlands;
- Section 401 Water Quality Certification;
- Easements from individual property owners and public agencies;
- Archeological clearance from SHPO;
- Coastal Zone Consistency Determination; and
- Water rights from ADNR.

If there were work on the outfall at a location in tidal and submerged lands in Kachemak Bay, an Alaska Department of Fish and Game (ADF&G) Special Area Use Permit may be required, in addition to authorization by the USACE. Work would need to be determined consistent with the Alaska Coastal Management Program.

If impoundments are established in either Twitter Creek or Fritz Creek, the impoundments will be subject to authorizations granted by the ADNR Dam Safety Program to construct and operate the facilities. Establishment of a reservoir would also require acquisition of water rights and Fish Habitat Protection permits from ADNR.

A pipeline from a new reservoir to the existing WTP could require:

- ADEC plan review, and approval to construct and operate water and sewer improvements;
- ADNR approval for construction activities that cross streams and other water bodies;
- ADOT&PF approval for work in state ROWs and road crossings;
- FAA approval for work in the boundaries of the airport and its vicinity;

- USACE permits for work in shorelines or wetlands;
- Section 401 Water Quality Certification;
- Easements from individual property owners and public agencies; and
- Archeological clearance from SHPO.

Depending on the permitting approach adopted for the project, several activities could be covered by a single permit or consistency review.

Any work performed under a grant/loan program would require approval from the agency funding the program. The City should ensure that no subsurface or ROW conflicts exist prior to construction. All water and sewer installations will require utility locates before the final design and construction of the proposed improvements. The City does not require building permits for the construction of public utilities. The City does not have a zoning program, although it is in the process of developing one. The design of future facilities should take into account any future regulations, or changes to general permits.

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11.0 REFERENCES

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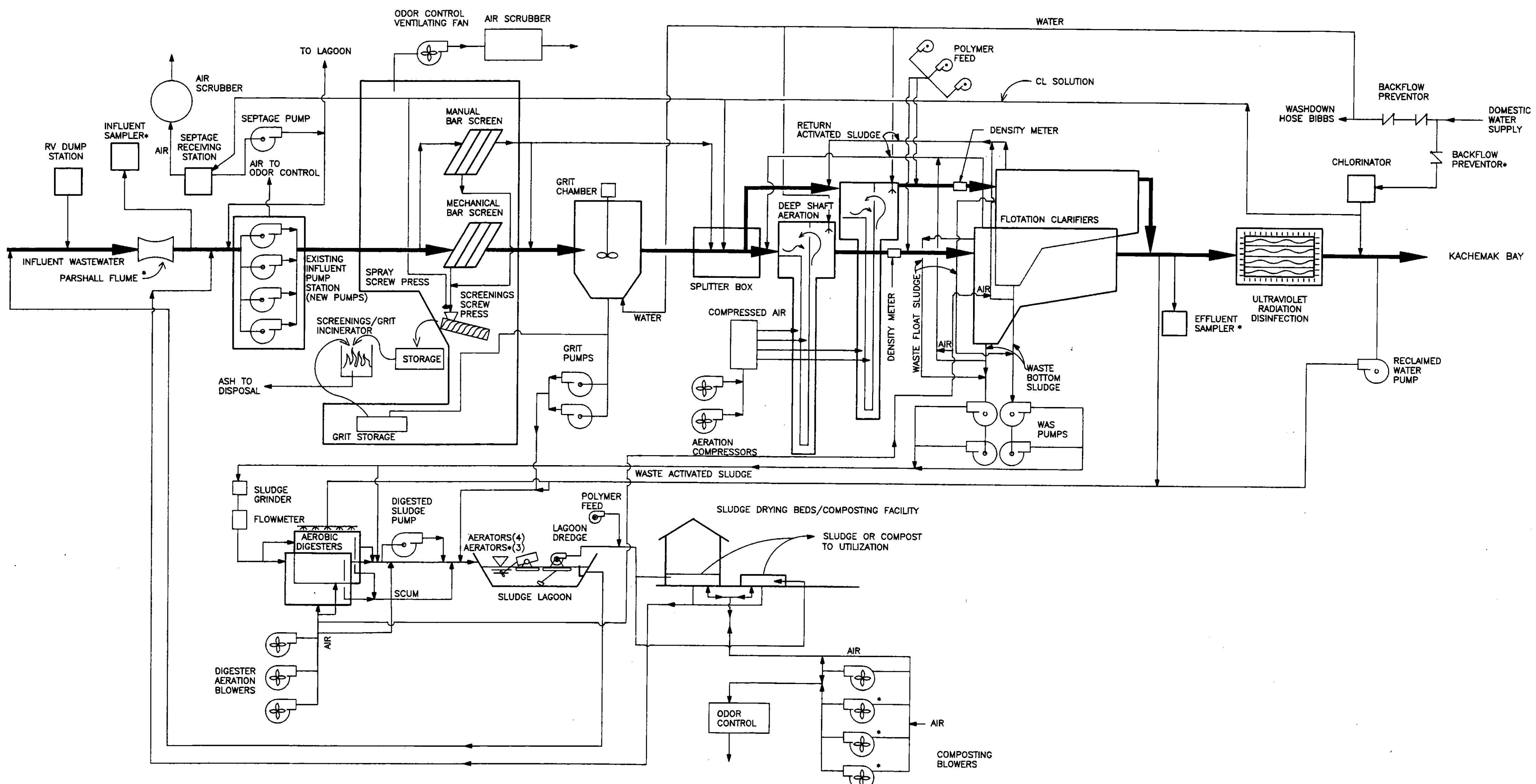
DRAWINGS

REFERENCE DRAWING

Wastewater Treatment Plant Process Flow Diagram



• EXISTING EQUIPMENT

NOTE: DRAINAGE AND NON-PROCESS RELATED WATER SYSTEMS ARE NOT SHOWN.

PROCESS FLOW DIAGRAM

DRAWING CONFORMED TO CONSTRUCTION RECORDS
Date 5/13/92 By [signature]
R. W. Beck and Associates

STATE OF ALASKA
49TH
LOWELL W. WARREN
No. CE 4807
REGISTERED PROFESSIONAL ENGINEER

REV.	DATE	BY	CK'D	APPROVALS	DESCRIPTION
2	5-13-92				DRAWING CONFORMED TO CONSTRUCTION RECORDS
1	6-15-89				ISSUED FOR CONSTRUCTION
0	3-2-89				ISSUED FOR BIDS

DRAWING REVISION RECORD

DESIGNED LWS	CIVIL REVIEW DJC	MECHANICAL REVIEW LWS
DRAWN SCS	STRUCTURAL REVIEW ETH	ELECTRICAL REVIEW GTG
CHECKED LWS	DATE 3-2-89.	APPROVED LWL

R. W. BECK AND ASSOCIATES, INC
ENGINEERS AND CONSULTANTS

CITY OF HOMER, ALASKA – WASTEWATER TREATMENT FACILITIES
MAIN TREATMENT FACILITY AND OUTFALL
PROCESS FLOW DIAGRAM 01127

WORK ORDER NUMBER UU-1957-ST2-AY
DRAWING NUMBER 824-G-3.2

APPENDIX A

Homer Water Treatment Plant Review

HOMER
WATER TREATMENT PLANT
REVIEW

Prepared for
Bristol Environmental and Engineering Services, Corporation

for the
City of Homer

July 2006

GV Jones & Associates, Inc.
WATER AND WASTEWATER PROCESS ENGINEERS
1200 E. 76th Ave., Unit 1207
Anchorage, AK 99518

Final

Table of Contents

Figures

Tables

Attachments

ACRONYMS AND ABBREVIATIONS

%	percent
µg/L	micrograms per liter
AAC	Alaska Administrative Code
ADEC	Alaska Department of Environmental Conservation
CCR	Consumer Confidence Report
CFR	Code of Federal Regulations
City	City of Homer
CT	chlorine contact
D/DBPR	Disinfectants and Disinfection By-Product Rule
DAF	Dissolved Air Floatation
EPA	U.S. Environmental Protection Agency
EPS	extracellular polymeric substances
ft^2	square feet
GAC	granular activated carbon
gal	gallons
gpd	gallons per day
gpm	gallons per minute
GWR	ground water rule
HAA5	LCR
Hp	horsepower
ICR	Information Collection Rule
LT1 ESWTR	Long-Term Stage 1 Enhanced Surface Water Treatment Rule
LT2 ESWTR	Long-Term 2 Enhanced Surface Water Treatment Rule
MCL	maximum contaminant limits
MEF	Microsand-enhanced flocculation
MF	microfiltration
mg/L	milligrams per liter
NF	nanofiltration
NPDES	National Pollutant Discharge Emergency System

ACRONYMS AND ABBREVIATIONS (continued)

PAC	powdered activated carbon
pCi/L	picoCuries per liter
psi	pounds per square inch
PWS	public water system
RO	reverse osmosis
SWTR	Surface Water Treatment Rule
TCR	Total Coliform Rule
TOC	total organic carbon
TTHM	Total Trihalomethanes
UFRV	unit filter run volume
WTP	water treatment plant

Water Treatment Plant Review

Section 1: Introduction

The City of Homer (City) operates a direct filtration plant treating surface water from Bridge Creek Reservoir. A pump station at the reservoir draws through a screened intake and delivers raw water to the water treatment plant (WTP). At the WTP, raw water is metered, pre-chlorinated, dosed with alum coagulant, and receives static mixing. Coagulated water feeds eight pressure filtration vessels. Filtrate is pH adjusted, dosed with corrosion inhibitor, post-chlorinated, provided with contact time, stored, and distributed.

Filter run times reportedly deteriorate during the summer and appear to correspond to changes in raw water quality as a result of algal and turbidity events. Net filtrate production during these events decreases markedly due to increased backwash frequency. With recent drought conditions, and corresponding decreases in Bridge Creek Reservoir levels, frequent filter backwashing requirements impose higher water demands on the limited water in the reservoir.

Though constructed to produce a design flow rate of 1,400 gallons per minute (gpm), the WTP is now normally operated at approximately 800 gpm and when necessary up to 1,000 gpm.

The objective of this report is to review options for upgrading or replacing the existing WTP to meet current water demands, as well as provide the City with water treatment capacity to meet future demands and current and future regulatory requirements.

This review of the WTP begins with an initial descriptive review of the existing system (Section 2) followed by a discussion of the operational problems noted by the City (Section 3). Based on data collected during a visit to the WTP and subsequent data provided by the operations staff, an evaluation of individual unit processes used in the treatment process is presented (Section 4). Section 5 describes the regulatory outlook for the facility, addressing both microbial and disinfection byproduct regulations that will affect the WTP. Section 6 addresses alternative upgrade options to meet future demands, and current and future regulatory requirements. Recommendations for upgrades are addressed in Section 7.

Section 2: Description and Operation of the Existing Water Treatment Plant

The City operates a water treatment system identified by the Alaska Department of Environmental Conservation (ADEC) as a Class A community system, public water system number 240456 (PWS ID# 240456).

The following paragraphs describe water demands, water quality, and the configuration and operation of the existing WTP.

Summary of Current and Future Water Demand

Current and future water demands, which are discussed in detail in other sections of the Water and Sewer Master Plan, are summarized below for purposes of this WTP review.

Year 2006

Average day demand:	570,000 gallons per day (gpd)	
Average day demand (summer):	800,000 gpd	(~560 gpm)
Maximum day demand (summer):	1,280,000 gpd	(~890 gpm)
Peak hour demand (summer):	2,000,000 gpd	(~1,390 gpm)
Year 2025		
Average day demand:	1,540,000 gpd	
Average day demand (summer):	1,860,000 gpd	(~1,290 gpm)
Maximum day demand (summer):	2,980,000 gpd	(~2,070 gpm)
Peak hour demand (summer):	4,660,000 gpd	(~3,230 gpm)

Source Water Quality

Homer's surface water source can be described as cold, soft, and with low to moderate alkalinity. Water quality varies considerably throughout the season from summer to winter, with turbidity spikes related to wet weather events and runoff, as well as algal events related to warmer water and extended daylight hours. **Table 1** summarizes available water quality data for the source reservoir.

Table 1: Raw Water Quality Data

Parameter	Summer (June – September)	Winter (October – May)	Season Unspecific
Conductivity, µmohs			50 – 60*
Color, APHA units*			40 - 90*
Hardness, mg/L as $CaCO_3$			20 - 25*
Calcium, mg/L			4 – 8*
Magnesium, mg/L			2.44*
Iron, mg/L			0.011*
Alkalinity, mg/L as $CaCO_3$			13 - 25.2
pH	6.4 – 7.9	5.8 – 9.0	
Temperature, ^{o}F	61	37	38-42*
Turbidity, NTU	0.1 – 9.8	1.1 - 22.5	
Total Organic Carbon			1.6 – 22.5**

Notes:

*Table 3-1 Corrosion Control Strategy Report, HDR Engineering, Inc., March 1995

**Alaska Department of Environmental Conservation database.

µmohs	=	micromohs	mg/L	=	milligrams per liter
°F	=	degrees Fahrenheit	NTU	=	Nephelometric Turbidity Unit
$CaCO_3$	=	calcium carbonate			

Compatibility of Direct Filtration with Raw Water Quality

The U.S. Environmental Protection Agency (EPA) provides recommendations regarding the general treatment capability of filtration systems based on raw surface water quality. A summary of published criteria is provided in **Table 2**. According to Table 2, Homer's direct filtration plant would be categorized as "in-line filtration".

Table 2: Filtration System Compatibility with Raw Water Quality Conditions*

Treatment	Total Coliforms (#/100 ml)[1]	Turbidity[1] NTU	Color[1] unit	TOC mg/L	Algae ASU/ml
Conventional with Pre-disinfection	<20,000	No Restrictions	<75		
Conventional without Pre-disinfection	<5,000	No Restrictions	<75		
Direct Filtration with Flocculation	<500	<7 – 14	<40	<2.5	$<10^3$
In-line Filtration	<500	<7 – 14	<10		
Slow Sand Filtration	<800	<10	<5		
Diatomaceous Earth Filtration	<50	<5	<5		

*Alaska Drinking Water Procedures Manual, Table 3; the Alaska Water Treatment Guidance Manual Table 4-4; and the EPA Surface Water Treatment Rule Guidance Manual, Table 4-2.

#/100ml	=	number of colonies per 100 milliliters	mg/L	=	milligrams per liter
<	=	less than	ml	=	milliliters
ASU	=	Areal Standard Units	TOC	=	Total Organic Carbon

Based on the raw water quality summarized in **Table 1,** the published recommendations of the EPA, and experience with direct filtration plants operating around the State, Homer's direct filtration plant may not perform well during turbidity spikes (ranging from 1.2 to 22.5 Nephelometric Turbidity Units [NTU]), periods of high color (40 to 90 APHA units) and total organic carbon (TOC) (ranging from 2.9 to 11 milligrams per liter [mg/L]), and algal events. Thus, potentially compromising filter capability of meeting the minimum *Giardia lamblia* and virus removal requirements as listed in Section 4, and *Cryptosporidium* (turbidity) removal requirements, in addition to impacts of poor filter performance.

Original design criteria for the direct filters were based on a filtration flow rate of 1,400 gpm and associated parameters with the average and range of values summarized in **Table 3**.

Table 3: Original Design Criteria for Homer's Direct Filter Plant

Parameter	Average Value(Range)	Units
Pre-Chlorination	2 (0.3 - 3.0)	mg/L
Pretreatment pH Adjustment – Soda Ash	11 (1.5 – 15)	mg/L
Alum Coagulation	20 (3 – 30)	mg/L
Filter Aid (Cationic) Polymer	3 (0.5 – 5)	mg/L
Nonionic Polymer	0.5 (0.1 – 7)	mg/L
Filtration Rate	3.48	gpm/ft^2
Backwash Rate	18	gpm/ft^2
Post-Chlorination	2 (0.5 - 4.0)	mg/L

Notes: ft^2 = square foot gpm = gallons per minute mg/L = milligrams per liter

The current configuration of the water system operates at 800 gpm and, as required, up to 1,000 gpm. Plant performance parameters under these operating conditions are summarized in **Table 4**.

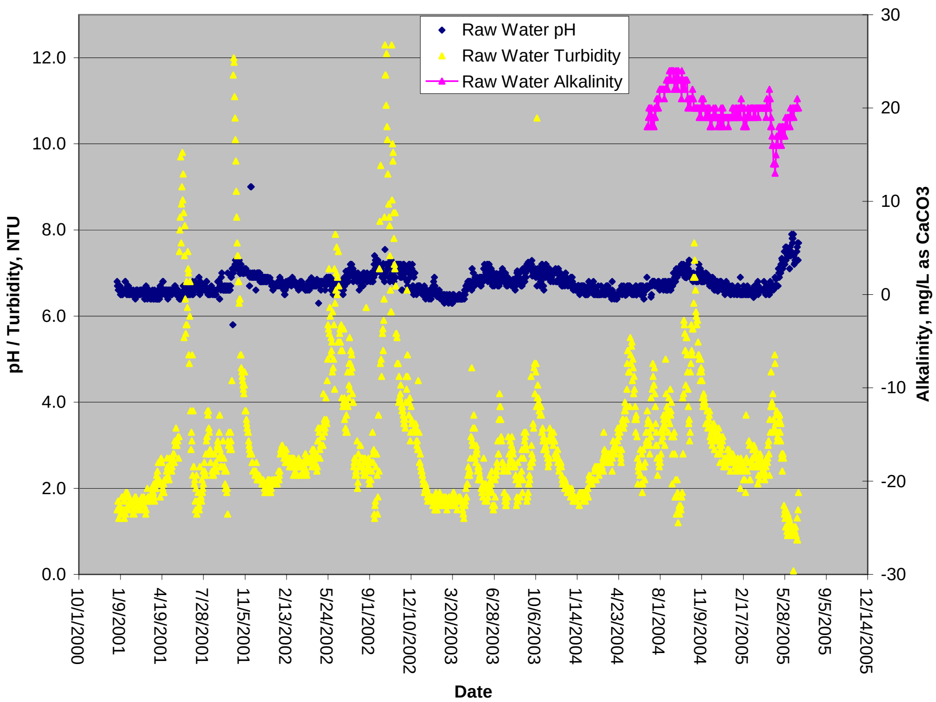
Table 4: Current Operation Parameters for Homer's Direct Filter Plant

Parameter	Value/Range	Units
Pre-Chlorination	1.6	mg/L
Pretreatment pH Adjustment – Soda Ash	(Not Currently Used)	
Alum Coagulation	17 - 22	mg/L
Filter Aid Polymer	(Not Currently Used)	
Filtration Rate at 800 gpm	1.99	gpm/ft2
Filtration Rate at 1,000 gpm	2.49	gpm/ft2
Backwash Rate	18	gpm/ft2
pH Adjustment – Caustic Soda	7 - 10	mg/L
Corrosion Inhibitor – Blended Polyphosphate	0.5	mg/L
Post-Chlorination	2.2	mg/L
Chlorine Residual for CT	0.8 - 1.0	mg/L

Notes: ft^{2} = square foot gpm = gallons per minute mg/L = milligrams per liter

Representative raw water quality data are provided in **Figure 1.**

Figure 1: Raw Water Quality Data



Treated Water Quality

Table 5 presents treated water quality parameters. TOC and alkalinity samples were collected at the discharge of the water treatment plant. Total trihalomethanes (TTHMs) and five haloacetic acids (HAA5s) were collected at the Spit pumping station.

Table 5: Treated Water Quality Data

Parameter	Range of Results	Running Annual Average	Maximum Contaminant Limit	Trigger Requiring Disinfection Profiling
Total Organic Carbon, mg/L	1.6 - 2.8			
Alkalinity, mg/L	19.8 - 34.7			
TTHM, µg/L	22 – 60.1	41	80 *	64 **
HAA5, µg/L	34.4 – 72.0	58.8	60 *	48 **

Notes:

*Requirements of Stage 1 D/DBPR (See Section *Regulatory Outlook)*

**Requirements of LT1 ESWTR (See Section *Regulatory Outlook)*

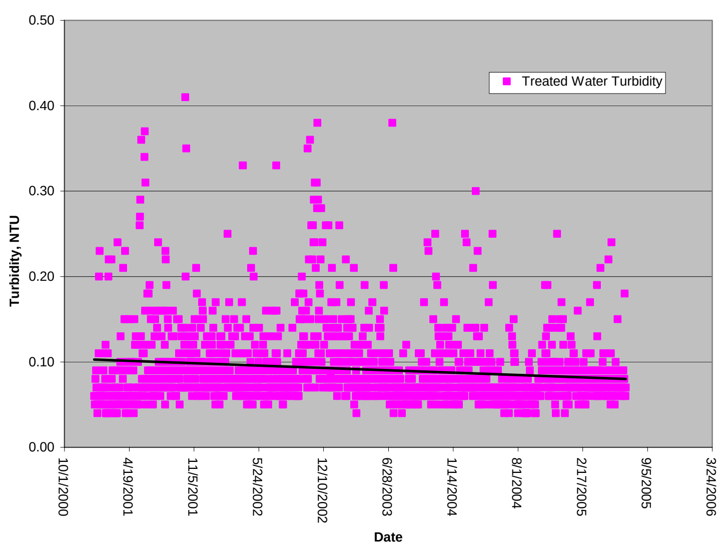
µg/L = micrograms per liter

HAA5 = Haloacetic Acid

mg/L = milligrams per liter

Additional treated water quality data for 2001 through November 2005 are provided in **Figure 2**. Reported treated water turbidity during this period did not exceed 0.4 NTU. The 95 percentile of reported treated water turbidity was 0.16 NTU during summer months, 0.2 NTU during winter months, and overall 0.18 NTU.

Figure 2: Treated Water Quality Data



Existing Plant Configuration and Operation

Source Water Supply and Control

Water from the Bridge Creek Reservoir is collected in an adjustable depth intake structure, 4- by 4- by 2-foot high, with sides constructed of ½-inch stainless steel bars at 1-1/2-inch spacing. Water is pumped from the collection structure by three vertical turbine pumps, each with a rated capacity of 700 gpm discharging through two transmission pipelines (8- and 10-inch) approximately ¾-miles to the WTP. The pumps are Byron Jackson, line shaft turbines, 75 horsepower (Hp), Model 806W1380.

Pre-Chlorination

An on-site generator provides hypochlorite for both pre- and post-chlorination. Raw water is injected with hypochlorite upstream of an in-line static mixer. Average solution strength is 0.33 percent free available chlorine; currently dosed at 1.6 mg/L. Operators report pre-chlorination is used to prevent algal growth in the pressure filters. Past studies suggest pre-chlorination enhances oxidation of excessive iron and manganese, which is subsequently removed by filtration.

Coagulation

The primary coagulant used in the treatment process is aluminum sulfate (alum). Alum is injected downstream of pre-chlorination and upstream of the inline static mixer. Operators report current solution strength at 1.25 pounds alum per gallon of water (15%), dosed at a range of 15 to 20 mg/L.

Static Mix

A 10-inch diameter in-line static mixer is provided primarily to provide flash mixing for the coagulation process. The unit is an FRP Kenics Mixer.

Filtration

Coagulated water is fed to two trains of four pressure vessels each. Each pressure vessel is 8 feet in diameter, has a 5-foot vertical shell height, and an approximately 10-foot overall height. Filtration area provided per vessel is 50.3 square feet, with a total of 402.1 square feet available during normal filtration and 351.9 square feet available during backwash operations. Surface wash capability was designed for 33.5 gpm at 60 pounds per square inch (psi) provided by two Aurora 99-7369, Type 341A 9F, 1.25x1, 5x7, 7.5 Hp pumps.

Though the WTP was designed at a filtration rate of 1,400 gpm and per regulations at that time, the WTP now normally operates at 800 gpm. Operators report the WTP's filtration rate may be increased to about 1,000 gpm, but this rate has not been exceeded with recent source water quality. The original design backwash flow rate was 905 gpm, with 12 minutes of backwash time. Currently, backwash is operated at 780 gpm for 12 minutes. During the winter, backwash duration is set at 8 minutes. Operators indicate they vary backwash rates periodically to account for seasonal water temperature variations. **Table 6** provides a summary of filtration and backwash rates at the various flows noted and during backwash of a single filter.

Table 6: Summary of Filtration and Backwash Rates

Flow Rate, gpm	Normal Filtration Rate, gpm/ft^2	Filtration Rate During Single Filter Backwash, gpm/ft^2	Backwash Rate, gpm/ft^2
700	1.74	1.99	
800	1.99	2.27	
1,000	2.49	2.84	
1,400	3.48	3.98	
780			16
905			18

Notes: ft^2 = square foot gpm = gallons per minute

The filtration rates shown for the original design flow rate of 1,400 gpm are in excess of published rapid sand filtration rates (2 to 3 gpm/ft^2), but less than high-rate filtration (5-10 gpm/ft^2)[(8)]. The WTP – operated at a flow of 700 gpm and up to 1,000 gpm – corresponds to filtration rates within rapid sand filtration rate category.

Operators report the vessels are currently loaded with sand and anthracite over gravel. The filter media in use are summarized in **Table 7**.

Table 7: Filter Media

Layer	Depth	Media	Grade	Description
1	3-inch	#1 Gravel	1½" – ¾"	Support Bed
2	3-inch	Gravel	¾" – 3/8"	Support Bed
3	2½-inch	Gravel	1/4" – 1/8"	
4	2½-inch	# 12 Gravel		
5	18-inch	Silica Sand	0.45 - 0.55 mm	Filtration Media
6	12-inch	Anthracite	0.6 mm – 0.8 mm	Filtration Media
TOTAL	**41-inch**			

Notes: " = inch mm = millimeter

The under drain system consists of a 6-inch diameter header, 1-1/2-inch diameter laterals and nozzles.

No filter-to-waste piping and valving is installed.

pH Adjustment

Post treatment pH adjustment is achieved with sodium hydroxide (caustic soda). Average solution strength is 50%, dosed at a range of 7 to 10 mg/L. Target pH is 7.5.

Post-Chlorination

The on-site hypochlorite generator also provides sodium hypochlorite for post-chlorination of filtrate. Sodium hypochlorite is injected upstream of an inline static mixer. Average solution strength is 0.33% free available chlorine. This solution is currently dosed at 2.2 mg/L.

Corrosion Control

Treated water is dosed with 0.5 mg/L of a blended polyphosphate intended to act as a corrosion inhibitor. A proprietary product called Aqua Mag by Carus Corporation is currently used.

Chlorine Contact

Disinfection is achieved by allowing chlorinated water to reside in chlorine contact (CT) tank(s) before distribution. The City's existing reservoirs used for chlorine contact were sized based on the design parameters summarized in **Table 8**.

Table 8: Existing Chlorine Contact Design Summary

Tank	Volume for CT, gallons	Chlorine Residual, mg/L	Temp., °C	pH	Baffle Factor	Log Inactivation, *Giardia Lamblia*
"CT Tank"	150,000	1.0	0.5	7.5	0.5	0.5
0.5 MG Reservoir	325,000	0.8	0.5	7.5	0.08	0.5

Notes: °C = degrees Celsius mg/L = milligrams per liter

In early 2004, the City installed a new 1.0 million gallon (MG) water storage tank off Skyline Drive just south of the WTP and the existing 0.5 MG reservoir. The 0.5 MG tank and the new 1.0 MG tank are linked hydraulically so they operate at the same level. Per the original design of the 0.5 MG reservoir, the minimum water level for CT is 12 feet. Operation of the hydraulically-linked tanks targets the maintenance of water level at 14 feet and greater to enhance pressure delivery at local high elevation services.

Although the City does not currently include the 1.0 MG tank in its CT calculation spread sheets, the tank does provide CT in the same manner as the 0.5 MG tank. Based on the minimum water level for CT maintained in the 0.5 MG tank, the minimum volume for CT contributed by the 1.0 MG tank is approximately 677,000 gallons.

Process Wastes

Process waste water produced at the Homer WTP consists of filter backwash water. Operators report that, during normal operations, backwash water consumes 5% to 7% of water produced. Based on daily demands for 2005 and these percentages, backwash quantities may range from 15,500 gpd to 106,400 gpd. Backwash may be in excess of these figures when short filter run times occur.

Backwash water discharges to the northern most (Pond 1) of two lagoons for clarification by gravity separation of water treatment sludge. The City recently installed a new pipe between Ponds 1 and 2 for the equalizing of lagoon water levels. Clarified water from Pond 2 is returned to the head of the WTP with two decant pumps (Pacific Pump, 2-Hp). The City replaced one of these pumps with the most recent version (Sulzer Cat. No. 45-20700-38060-2562) of the same model with the largest impeller available. The system is now able to achieve a maximum 40 gpm return rate when operating that pump. The other original pump is capable of 20 gpm return rate. Flow rates are modulated with valve throttling. Sludge is removed from the lagoons periodically and disposed of, although disposal has not occurred within the past 10 years.

Section 3: Operational Problems

The performance problems identified through discussions with Operations staff and the City are summarized in this section of the report. This review is intended to support the development of performance goals and criteria for upgrading or replacing the existing WTP to meet current and future water demands. The results of the WTP capacity analyses are addressed in a subsequent section of this report.

Raw Water Quality – Algal Events, Turbidity, and Color

Perhaps the most significant issue affecting the WTP is the variable raw water quality parameters that can exceed the general treatment capability of Homer's direct filtration plant. The most noted operational impact is short filter run times and excessive water use for backwash. The water quality parameters suspected of contributing are algae, turbidity and color. Though very little data exist, excessive iron and manganese have been noted in past reports for Bridge Creek and reservoir.

Turbidity events tend to coincide with wet weather events. Very little color and TOC data exist and no correlation with filter performance or treated water quality is yet known.

Operators report that when algal events occur in warm and sunny months, they are intermittent, vary in intensity, and reoccurrence coincides with pH variations. Algae appear to be more concentrated at the lower levels of the reservoir.

The City has conducted an in-house review of algal events and has twice been able to identify the presence of the diatoms (Bacillariophyceae) class of algae, specifically, *Asterionella*. The presence of diatoms in source water is associated commonly with filter clogging, as well as taste and odor problems. Coagulant studies have shown at equal metal ion concentrations, polyferric sulfate achieved higher algae removals followed by alum, with ferric sulfate yielding the lowest percentage removals of *Asterionella*(1). Algae and algal metabolites often impact water treatment by: clogging reservoir screens; increasing coagulant demand; shortening filter run times due to clogging or premature filtrate turbidity breakthrough; increasing backwash water requirements; increasing chlorine demand; producing tastes, odors, and toxins; and increasing microbial regrowth in the distribution system.

The City notes drops in alkalinity decrease the effectiveness of the coagulation process and causes increased alum consumption.

Short Filter Run Time

Operators report filter run times of up to 3 hours under worst case conditions, and up to 12 hours under the best raw water quality conditions.

Filter to Waste

Currently, piping and valves that enable the WTP to incorporate a filter to waste sequence as part of the filter backwashing cycle are not in place. After a filter is backwashed, the filtrate produced has a higher concentration of colloidal particles than during the filter ripening process. This elevated concentration of particles in the filtrate is often reflected in higher turbidity levels for a period of time for that filter. Using a filter to waste sequence before placing a washed filter back into service prevents poor quality filtrate from entering the supply of treated water, and may better enable the system to comply with current and future filtrate turbidity requirements.

Process Wastes

Operators report several concerns with the backwash lagoons. The northern-most lagoon is filled with sludge due for removal. Even with the recent installation of a new decant pump, the return rates may be less than desired. If further upgrades are deemed necessary, operators suggest variable frequency drives for the pump motor drives. Depending on future upgrades, a National Pollution Discharge Elimination System (NPDES) discharge permit may be required for lagoon discharges.

The method and location of disposal of water treatment sludge may be an issue for Homer to review and plan for because the service available from the Kenai Peninsula Borough varies.

Filtrate and Treated Water Quality

Though reported combined effluent filtrate turbidity is very low, other constituents of the filtrate appear to impart a substantial chlorine demand throughout the year. As a result, the free chlorine residual maintained is relatively high to compensate for extended residence time in the distribution system. HAA5 concentrations have exceeded the trigger (Section 141.531) recently, thus requiring disinfection profiling. Though the current running annual average is less than the maximum contaminant level (MCL) for both HAA5s and TTHMs, HAA5s are very near exceeding the MCL.

Section 4: Unit Process Evaluation

An evaluation was performed on each of the unit processes that comprise the existing WTP. In this section, the basis for analysis of each system element (coagulation, mixing, filtration, and disinfection) is presented, along with a summary of the results. Where appropriate, alternative methods of capacity analysis were performed and compared. Any assumptions required to estimate unit process capacity were based on manufacturers' information, manuals of practice, peer-reviewed journal articles, and experience.

The purpose of this evaluation is to determine performance limiting unit processes for identifying upgrade and/or modification options.

Coagulation

Aluminum sulfate (alum) is a coagulant used commonly and has proven successful with organic/color and algae-laden source waters like Homer's. Coagulant doses may require optimization with the use of cationic polymers to function as coagulant aids (1).

Mixing

The mixing provided by the static mixer should provide a retention time of 1 to 3 seconds (8). At the current filtration rate of 800 gpm, the static mixer would have to be 3.2 to 9.8 feet long. Though the exact length of the mixer is not verified, common static mixer design would result in a mixer of about 42 inches, meeting the minimum recommended retention time. An alternative would be the use of mechanical induction-style mixers.

If mixing were not sufficient, the efficiency of coagulation may be impacted, leading potentially to coagulant over dosing.

Filtration

Hydraulic Loading Rate

The hydraulic loading rate is the flow applied to the filters per square foot of filter media surface area. Loading rates depend upon the type of pretreatment applied to the water upstream of the filters, the type of filter media used in the filters, and the quality of the source water. Homer's dual-media rapid rate pressure filters are intended to allow for storage of solids and are often referred to as depth filters.

Hydraulic loading rates for these filters are summarized in **Table 6**. Traditional rates of filtration were 2.0 gpm/ft^2, until 4.0 gpm/ft^2 became more accepted because of studies performed in the mid 1900s (7), with even higher rates based on suitability of pretreatment. The Alaska Department of Environmental Conservation (ADEC) (15) provides a guideline of 3.0 to 6.0 gpm/ft^2 for high rate filters, with an opportunity to use higher rates if pilot testing is used to verify performance.

Backwash Water to Filtered Water Ratio

One common filter performance evaluation criterion is the ratio of water used for filter backwashing relative to the filtered water produced during a single filter run. Normally, backwash water is between 2% and 3% of the filtered water flow. Backwash flows in excess of 5% are indicative of a problem with the treatment system (8).

For the entire month July 2004, while the WTP experienced short filter runs, Operations staff collected backwash and filtered water production data – yielding ratios ranging between 7% and 44% and averaging 23%. These values, well in excess of 5% confirmed inadequate filter performance.

Unit Filter Run Volume

The unit filter run volume (UFRV) is another indicator of filtration process performance. It is computed as the product of the filter loading rate in gpm/ft^2 times the length of the filter run in minutes. Values of UFRV over 10,000 gallons (gal)/ft^2 are considered normal. Values under 5,000 gal/ft^2 are often considered unacceptable [7,8].

From the data presented in **Table 9**, it appears that while the existing filter loading rate is well within the range considered acceptable elsewhere, the actual performance of the filters is not. The backwash water to filtered water ratios are in excess of recommended criteria is some cases, and the unit filter run volumes are well outside of target range for this parameter. It is possible that either the filter media are graded incorrectly and/or the solids loading to the filters are too high resulting in early breakthrough or filter plugging.

Table 9: Performance Criteria for Rapid Rate Filtration

Performance Criteria	Existing Filters	Criteria Used Elsewhere
Filter Loading Rate at 800 gpm process flow, gpm/ft^2	1.99	2[7,8] 4[7,11] 6[8] 3 to 6[15]
Backwash Water to Filtered Water Ratio, %	1.6 to 6.5*	<5[8]
Unit Filter Run Volume, gal/ft^2	358 to 1433*	>5,000 to 10,000[7,8]

Notes: *Assumes 3 to 12 hour filter runs at 800 gpm with backwash volumes of 9,360 gallons (12 minutes at 780 gpm).

%	=	percent	gal	=	gallons
<	=	less than	gpm	=	gallons per minute
ft^2	=	square foot		=	

Since filter media selection and gradation provided in **Table 7** are reasonably consistent with recommendations found in the literature for this type of filtration, overloading may be a more likely cause of poor filter performance. Filter media may also require replacement or replenishing depending on age, operational impact, and loss over time. In addition, use of coagulant and filter aid may increase efficiency of the filtration process.

Disinfection

The addition of chlorine to filtered water is intended to establish a chlorine residual that will inactivate pathogenic organisms. The effectiveness of inactivation is a function of contact time, free chlorine concentration, water pH, and temperature. The City was targeting a free chlorine residual delivery to the distribution system of approximately 0.8 mg/L, but is now targeting 0.4 mg/L.

State and Federal Drinking Water Regulations require at least 99.9 percent (3-log) removal, inactivation, or a combination of removal and inactivation, of *Giardia lamblia*, and at least 99.99 percent (4-log) removal, inactivation, or a combination of removal and inactivation, of viruses before distribution of the drinking water. *Cryptosporidium* inactivation/removal for surface waters is addressed by a minimum treatment performance criterion that requires maintaining filtered water turbidity at 0.3 NTU or less, for 95% of the time the filters are producing filtrate. The EPA suggests meeting the turbidity standard can achieve a minimum of 2-log *Cryptosporidium* removal.

As published in Table 1 of the Alaska Drinking Water Procedures Manual and Table 2 of the State of Alaska Water Treatment Guidance Manual, minimum log removal credits for *Giardia lamblia* and viruses for systems using direct filtration technologies are between 1-log and 2-log, respectively. These data, along with the required log inactivation for chlorine disinfection, are summarized in **Table 10**. In contrast to the State of Alaska publications, Table 4-1 in the EPA Surface Water Treatment Rule Guidance Manual indicates direct filter minimum log removal credits for *Giardia lamblia* and viruses ranging between 2 to 3, and 1 to 2, respectively.

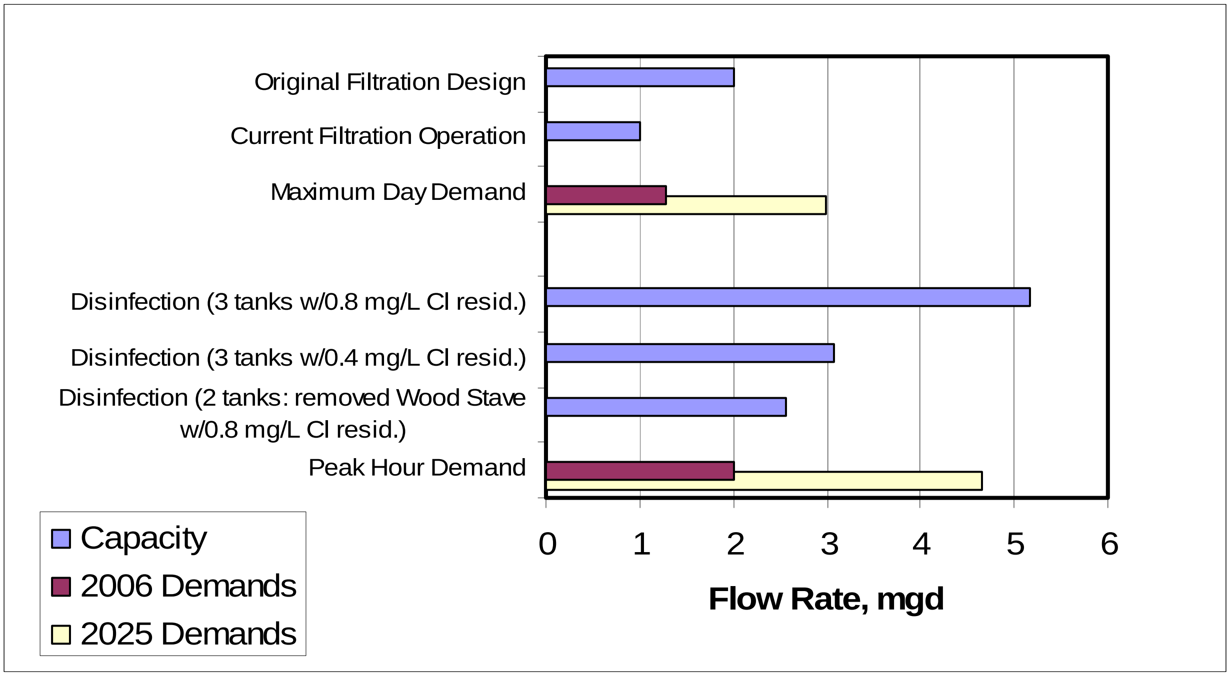
Table 10: Log Inactivation Required per State of Alaska

Regulated Pathogen	Log Removal Credit (Direct Filtration)	Log Inactivation Required (Chlorine Disinfection)
Giardia lamblia Cysts	2	1
Viruses	1	3
Total Coliform	1 to 3	
Cryptosporidium	*See discussion above*	

Summary of Individual Major Unit Process Capacities

Figure 3 summarizes the capacity of the major unit processes at the WTP and compares them to estimated 2006 and 2025 demands. The WTP, as currently configured and operated, provides less than the estimated maximum day demand for 2005 and does not meet the original design projections. Upgrades to the existing treatment process to recover the original design capacity of the filtration system would allow the system to meet current demand and regulatory requirements. However, additional new treatment facilities will be required to meet projected 2025 water demands.

Figure 3: Water Treatment Plant Capacity



Notes: Cl = chlorine mgd = million gallons per day mg/L = milligrams per liter

The graph also evaluates the CT capacity that can be provided by the system, with a variety of different tank and chlorine residual combinations. Using the log inactivation requirements provided in **Table 10**, the system can provide enough CT to treat just over 5 million gallons per day (MGD) if all three existing tanks (the 0.15 MG "CT tank", 0.5 MG Tank and 1 MG Tank) are on line and the chlorine residual is maintained at 0.8 mg/L. If the target chlorine residual is reduced from 0.8 mg/L to 0.4 mg/L, and all three existing tanks remain on line, the system can provide enough CT for approximately 3 MGD. If the

0.5 MG (wood stave) tank is removed to provide room for WTP expansion, enough CT could be provided with the remaining two tanks to treat 2.5 MGD. (The City currently bases CT credit on only two tanks – the 0.15 MG “CT tank” and 0.5 MG wood stave tank – and the parameters listed in **Table 8**).

Upgrades the existing WTP would act to increase CT capacity of the existing tanks. If the existing direct filtration system is upgraded to conventional or membrane filtration, additional removal credits would be granted, resulting in a reduction in the log inactivation requirements – effectively increasing the CT capacity of the existing tanks. In addition, an upgraded treatment system would also improve treated water quality that would allow for the maintenance of higher chlorine residual, which would increase effective CT capacity.

Section 5: Regulatory Requirements for Public Water Systems

The design, operation, and routine monitoring of public drinking water systems is regulated by federal and state legislation. The EPA has granted the State of Alaska the authority to administer federal drinking water regulations. The State has adopted the federal requirements into the Alaska Administrative Code (AAC) and designated ADEC as the state agency responsible for enforcement and compliance. Details on monitoring and reporting, and operation requirements are contained in the State of Alaska Drinking Water Regulations (18 AAC 80) and Water and Wastewater Operator Certification and Training (18 AAC 74) regulations. The following paragraphs summarize the major current and future regulatory requirements facing Homer. More details may be found in 40 Code of Federal Regulations, Part 141 (40 CFR 141).

Phase I/II/IIB/V Rules

These rules, promulgated from 1987 to 1992, establish MCLs and monitoring requirements for chemical contaminants such as: IOC, VOCs, and SOC's. Requirements vary from system-to-system, with nitrate and nitrite typically applicable to all public water systems (PWS's).

Based on data reviewed, Homer complies with this rule.

Total Coliform Rule

The Total Coliform Rule (TCR) promulgated June 29, 1989, sets MCLs and monitoring requirements for coliforms in drinking water. It requires the periodic collection and analysis of a number of samples, depending on system size.

In 2004, the EPA indicated plans to consider future revisions (or Distribution System Rule) to the TCR. The EPA has currently planned revisions or new rule for 2007 with possible delay.

Based on data reviewed for this study, Homer complies with the TCR.

Consumer Confidence Report

The EPA's Consumer Confidence Report (CCR) rule, 40 CFR Part 141, Subpart O, became effective as a federal law on September 18, 1998. This rule requires that all Class A PWS's that serve twenty-five (25) or more residents or 15 service connections year-round deliver their first CCR covering water quality data and violations for the calendar year 1998 to their consumers by October 19, 1999. CCRs are due each year and cover the previous calendar year's water quality data and violations.

The City has provided CCRs as required for this rule.

Rules Applying to Surface Water Treatment

Surface Water Treatment Rule

The Surface Water Treatment Rule (SWTR) was published in the Federal Register June 29, 1989. The SWTR sets maximum contaminant level goals of zero for specific, disease-causing, microbial contaminants. Requirements include using filtration and disinfection processes that will result in a prescribed level of pathogen removal or inactivation, and attainment of finished water turbidity standards. The current requirements of the SWTR are that the filtration system used, with disinfection, can demonstrate a 3-log (99.9%) removal or inactivation of *Giardia lamblia*, and a 4-log (99.99%) removal or inactivation of viruses.

Long-Term 1 Enhanced Surface Water Treatment Rule

The Long-Term Stage 1 Enhanced Surface Water Treatment Rule (LT1ESWTR) was published in the Federal Register on January 14, 2002. The rule requires all public water systems, including small systems like Homer's, meet the same microbial-control requirements larger systems had under the Interim Enhanced Surface Water Treatment Rule (IESWTR), promulgated December 16, 1998.

The IESWTR added further requirements to the SWTR for control of microbial pathogens, including *Cryptosporidium*. The maximum contaminant level goal for *Cryptosporidium* is set at zero. The rule requires that public water systems employing direct or conventional filtration and using surface water or ground water under the influence of surface water, achieve filtrate turbidities of 0.3 NTU, or better, 95% of the time in the combined filter effluent with no reading greater than 1 NTU. In addition, continuous (at least every 15 minutes) turbidity monitoring of individual filters is required and, should turbidity exceed 1 NTU in two consecutive readings, follow-up actions are required. Compliance with this performance criterion is intended to provide an average of 2-log removal (99%) of *Cryptosporidium*, although a direct correlation between turbidity and the occurrence of viable *Cryptosporidium* cysts does not exist.

Homer currently reports to ADEC combined filtrate turbidity measurements every 4 hours. In 2004, 95% of the reported turbidity data were 0.17 NTU or less.

Furthermore, if the concentration of filtered water disinfection byproducts as measured by TTHMs or HAA5s during the month of warmest water temperature, and at the point of maximum distribution residence time, exceed 80% of the Stage 1 DBPR MCLs, (64 and 48 micrograms per liter [µg/L] respectively), the system must conduct disinfection profiling. Disinfection profiling requires monitoring and reporting of the factors that influence the effectiveness of the disinfection process.

Results for HAA5s have exceeded the 48 µg/L level, triggering disinfection profiling. As such, Homer is required to commence disinfection profiling.

Finally, all filtered or unfiltered systems using surface water sources, or ground water under the influence of surface water, must conduct periodic sanitary surveys. Systems like Homer's are required to complete sanitary surveys every 3 years.

Filter Backwash Recycling Rule

In conjunction with the SWTRs, the Filter Backwash Recycling Rule (FBRR) promulgated June 8, 2001, requires PWSs operating direct and conventional filtration plants to review their backwash water recycling practices, and make approved changes as necessary, to ensure they do not compromise pathogenic microbial control, particularly by passing *Cryptosporidium* oocysts through the filter. Generally, the FBRR requires that impacted systems: introduce waters to be recycled to the head of the WTP and treated through all the existing unit processes, report to the state on the configuration and operation of the system, and maintain records of recycle operations. The City has notified ADEC of its backwash recycle practices and maintains on-site records of recycle quantities.

Stage 1 Disinfectants and Disinfection By-Products Rule

Health effects research has demonstrated disinfection by-products (DBPs), including trihalomethanes, haloacetic acids, chlorite, bromate, and other compounds, are formed when naturally-occurring dissolved organic material is exposed to chlorine. This research indicated ingestion of DBPs can result in cancer or other illness. Because of this information, the EPA has promulgated the Stage 1 Disinfectants and Disinfection By-Products Rule (D/DBPR). The rule establishes an MCL for TTHMs and HAA5s of 0.080 mg/L and 0.060 mg/L, respectively. Compliance with the MCL is based on a running average of samples taken quarterly from the distribution system.

The running average for HAA5s reported in **Table 5,** including data from 2004 and 2005, indicate HAA5s near the 0.060 mg/L MCL. An increase in the next quarter of testing may cause Homer to face a Stage 1 D/DBPR compliance issue in the near future.

The Stage 1 D/DBPR requires water systems to remove specified percentages of organic material measured as TOC, or to meet one of several alternative compliance criteria. The requirement applies to systems that use surface water or ground water under the influence of surface water and that use conventional filtration treatment. If this removal is not possible, the system is allowed to achieve the best percent removal of TOC it can demonstrate using enhanced coagulation. The basis of the prescriptive requirement for treatment is that, for source water with a high TOC, only a percentage of the TOC contributes to DBPs. Presumably, reducing the amount of TOC will reduce DBPs to below the MCL, or at least to the best practical level. Routine sampling for DBPs is required quarterly.

The TOC removal requirements of the Stage 1 D/DBPR are not applicable to direct filtration systems, presumably because direct filtration systems are not considered the best available treatment technology to deploy for source waters with significant concentrations of natural organic contaminants. However, achieving the rule's TOC removal levels would enhance the City's treated water quality and contribute to compliance with TTHM and HAA5 MCLs no matter the treatment system in use.

Information Collection Rule

The Information Collection Rule (ICR) is a monitoring and data-reporting rule promulgated by the EPA on May 14, 1996. It required that larger water utilities serving 100,000 people or more collect water quality data on their source water and treated water. These data have been used by the EPA to develop drinking water regulations mandated by the 1986 amendments to the Safe Drinking Water Act related to control of microbial contaminants and DBPs. The ICR also collectes engineering data on how these larger utilities control such contaminants.

The ICR does not affect Homer directly. However, it has generated data on WTP performance that will be used by the EPA to promulgate additional regulations that may apply to Homer in the future.

Lead and Copper Rule

The Lead and Copper Rule (LCR) was promulgated June 7, 1991, to limit the levels of lead and copper at consumers' taps. For systems that exceed the action levels for lead (0.015 mg/L) and copper (1.3 mg/L), a three-pronged mitigation approach is required. The initial step for public water systems not in compliance with the LCR is to complete a desktop study. The goal of the desktop study is to identify a corrective action program that will eliminate the lead and copper from the source water, or, if the metals are coming from corroding pipe materials, to control the aggressive nature of the water. The recommendations of the desktop study are submitted to the state for review and approval before implementation. Once the corrective action program is installed, the state requires additional testing to verify that the upgrade will bring the system into regulatory compliance. In some instances, follow-up testing may still result in non-compliance. If this is the case, the state is obligated to work with a PWS to optimize the corrosion control program it approved for use, thereby achieving the best possible water quality. The LCR does allow states to approve installed upgrades that have been optimized, but that do not completely achieve the targeted action levels.

Homer currently maintains a corrosion control program consisting of pH adjustment and corrosion inhibitor addition, and remains in compliance with the LCR.

Arsenic Rule

The 1996 amendments to the Safe Drinking Water Act required the EPA to propose an arsenic regulation that effectively reduced the MCL from 50 µg/L to 10 µg/L, and established a monitoring framework for routine sampling consistent with some of the other monitoring requirements. The rule was promulgated January 22, 2001, and the new arsenic MCL of 10 µg/L becomes effective January 23, 2006.

Per the ADEC database, Homer complies with the Arsenic Rule.

Radionuclides Rule

The Radionuclides Rule, promulgated December 7, 2000, applies to all PWS. The rule imposes MCLs for radioactive contaminants including: combined radium-226 and radium 228 at 5 picoCuries per liter (pCi/L), gross alpha particles at 15 pCi/L, beta/photon particles at 4 millirems per year, and uranium at 30 µg/L. Initial monitoring is to be completed by December 31, 2007.

Analyses for these new MCLs have yet to be completed for Homer. However, it is unlikely there will be a compliance issue, because elevated concentrations of radioactive contaminants are unusual for systems using surface water sources.

A rule for radon proposes an MCL of 300 pCi/L and an alternate MCL of 4,000 pCi/L. EPA expects to publish a final Radon Rule in 2007 or 2008.

Future Regulatory Requirements for Microbials and DBPs

In addition to the existing surface water related regulations already mentioned, the EPA under the 1996 reauthorization of the 1986 Safe Drinking Water Act, is developing a set of interrelated regulations to strengthen control of microbial and DBP contaminants in public drinking water supplies. These standards are referred to collectively as the Microbial/Disinfection By-Products rules.

The next round of anticipated rules will consist of the Long-Term 2 Enhanced Surface Water Treatment Rule (LT2ESWTR) and the Stage 2 D/DBPR. These rules require additional monitoring of source water, improved treatment for microbial inactivation, and possible lowering of MCLs for DBPs.

The LT2ESWTR is expected to address further means of controlling the occurrence of *Cryptosporidium* in drinking water supplies. Depending upon concentrations of *Cryptosporidium* found in a system’s source water, the rule is expected to require systems to implement one of several alternative treatment technologies and management strategies, termed collectively as the “microbial toolbox.”

One portion of the proposed LT2ESWTR addresses the performance and operation of membrane filter systems operating on surface water sources. Specifically, the proposed rule requires membrane filter equipment to undergo daily membrane integrity verification as a condition of continued use and operation. Membrane integrity verification is, therefore, now a key feature in the design of membrane filter systems proposed for use in potable water production.

The Stage 2 DBPR is expected to either reduce the allowable concentrations of disinfection byproducts in finished water, or maintain the current MCLs for DBPs, but require these limits be met at locations within a system’s distribution where DBP concentrations are expected to be the highest.

The Stage 2 D/DBPR and LT2ESWTR were promulgated on January 4th and 5th, 2006, respectively. Both rules became effective on March 6, 2006.

The **Ground Water Rule (GWR)** was proposed May 10, 2000, and the final GWR is now expected in late 2006. The GWR establishes multiple barriers to protect against bacteria and viruses in drinking water from ground water sources, and will establish a targeted strategy to identify ground water systems at high risk.

Certified Operators

A Class A water system such as Homer's must be supervised actively, as described in 18 AAC 74.010 and 18 AAC 74.410, by operators who are certified in accordance with Alaska Statute (AS) 46.30 and 18 AAC 74. ADEC classified Homer's water treatment system as a Level 3, based on a system score of "56".

The current operators and their certification level are listed in **Table 11**.

Table 11: Certified Operators

Operator Name	Certification/Number	Certification Expiration
Paul Barcus	Water Distribution 2 / 2171	12/31/06
David Bolt	Water Treatment 3 / 912	12/31/07
Kenneth Frazier	Water Distribution 2 / 10221	12/31/06
Jim Hobbs	Water Treatment 3 / 555	12/31/07
Gerald Lawver	Water Treatment 2 / 2392	12/31/06
Steven Martin	Water Treatment 3 / 7566 Water Distribution 1 / 8252	12/31/06

Water Treatment Sludge Disposal Regulations

CFR 503.6(i) and 18 AAC 60.500(c)(5)(F) indicate use or disposal of sludge generated from drinking water treatment processes is not regulated under these respective regulations.

Recommended Responses to Regulatory Requirements

Though ADEC has yet to adopt all of the federally promulgated rules, we recommend the City comply with applicable federal rules. As such, the rules that most affect Homer's water treatment system are: the LT1ESWTR, Stage 1 D/DBPR, FBRR, and the anticipated future regulations LT2ESWTR and Stage 2 D/DBPR.

Homer should first focus on compliance with the LT1ESWTR, Stage 1 D/DBPR, and FBRR with the existing treatment system, in anticipation of future WTP upgrades as follows:

Complete LT1ESWTR prescribed disinfection profiling and determine the disinfection benchmark in anticipation of future possible modifications to disinfection practices (including CT tank, 0.5-MG reservoir, and new 1.0 MG-reservoir).

Review data logs of individual filtrate and combined filtrate turbidity. Evaluate need for filter-to-waste piping and valving to control filtrate turbidity, as required to meet individual (less than 2 consecutive greater than 1 NTU) and combined (less than or equal to 0.3 NTU 95%-ile) filtrate turbidity requirements. Depending on evaluation results, design, construct, and implement filter-to-waste piping and valving.

Plan to design, construct, and implement lagoon sludge management and adequate capacity, variable speed pumping of lagoon supernatant recycle.

Collect TOC data on the source waters and filtered water to determine the amount of TOC removal achieved by the treatment process. Other source water testing should include:

Algae observation (identification) and quantification (Areal Standard Units per milliliter [ASU/ml])

Total Organic Carbon

Alkalinity

UV254

Dissolved Organic Carbon

Molecular Size Distribution Analyses

Silica

Iron

Manganese

Bromide

Conduct bench scale jar testing to identify a primary coagulant and analyze filtrate produced from the settled water for TOC removal, and chlorinated filtrate produced from the settled water for DBPs. The goal of the testing is to determine a primary coagulant for direct filtration that until water treatment upgrades are implemented, may increase the efficiency of the WTP with the intent of maximizing TOC removal and minimizing DBP formation while still achieving turbidity removal.

Evaluate alternative disinfection options for the case that direct filtration optimization does not decrease DBP formation.

Proceed with planning of WTP upgrades, not only to meet production and performance goals based on raw water quality, but also to control DBP formation and meet current and anticipated future regulatory requirements. Perform pilot testing. Planning efforts should include the acquisition of properties for location of treatment facilities and for potential water treatment sludge land application.

Section 6: Plant Upgrade Options

This section provides two sets of options, the first focusing on short-term options to be applied over the next 5-year period, and the second on WTP upgrade options intended for a 20-year planning horizon.

Short-term Options for Next 5-years

The following paragraphs present options for enhancing the performance of the existing WTP, as may be suitable as short-term options for implementation within the next 5 years, although some options may be suitable for continued use with future upgrades.

For most of these options, on-site assessments, bench scale testing, and/or pilot testing could be used to further assess the practicality, benefit, and cost of implementation.

Options for Control of Algae Blooms at Source

Several methods exist to control source water algal blooms. In the watershed, it is important to limit nutrient entry from sources such as runoff and septic systems.

At the source water reservoir, algae control options consist of:

Chemical precipitation of phosphorus in the source water using metal coagulants (aluminum or ferric based) or lime. The objective is to reduce the availability of soluble phosphorous as it is a key nutrient needed for algae growth.

Dredging to remove organic enriched sediment that provides both a carbon and nutrient source for algae growth.

Reservoir circulation to promote elimination of water column stratification, thereby minimizing the duration of photoexposure algae need for their metabolic processes.

Application of sunlight screening dyes to the reservoir. These dyes prevent light of specific wavelengths from reaching the water column and, thereby, inhibit growth of algae.

Biomanipulation of the source water, which can include introduction of aquatic species that feed on algae, thereby limiting their population in the source water.

Modification of the source water intake structure to move the depth of the intake screen in the water column mechanically to locations of lower algae concentration (modification of the intake was made in 2004 to achieve some adjustability).

Application of herbicides to the source water to prevent the growth of aquatic weeds that can harbor and support the growth of algae.

Application of algicides to the source water to inactivate algal organisms. Commonly used products include copper sulfate, copper ethanolamine, copper citrate, potassium permanganate, and chlorine.

Options for Treatment of Algae-Laden Water with Existing Direct Filtration Process

Common treatment techniques include:

Application of powdered activated carbon (PAC) to filter feed water.

Application of oxidants to filter feed water (oxidative pretreatment).

Application of algicides to filter feed water.

Alteration and/or optimization of the filtration media, and a coagulation program during algae events.

Using longer, more frequent, and/or enhanced filter backwashing during algal events.

Deployment of a granular activated carbon (GAC) adsorption process downstream of the filters.

These options are discussed further below.

PAC

As its name implies, PAC is a pulverized form of activated carbon. Common base materials are lignite, coke, coconuts, walnuts, or commercial wood timber products. Activation of these base products involves subjecting them to elevated temperatures and steam in the absence of oxygen, which removes most non-carbon materials and transforms the surface of the base material into a porous media. PAC has an average diameter of 0.04 millimeters. As with other activated carbon materials, PAC has a large affinity for absorption of organic materials. In application to algae-laden feed waters, the objective is for the algae to become attached to the PAC and, thereby, form larger colloidal particles that are more readily removed in the downstream coagulation and filtration processes.

Advantages

An advantage of PAC addition is the ability to absorb some algal cells and facilitate the coagulation process. In addition, it has the ability to absorb some of the intercellular organic material often present in source waters subject to algae blooms.

Disadvantages

The disadvantages of PAC addition are that PAC is highly flammable and poses serious health problems for the respiratory system when inhaled. PAC dosing systems require care in handling and application to avoid ignition or explosion in the work area, and inhalation by operations personnel. In addition, if the objectives are to control taste, odors, and remove the intercellular materials that algae release when stressed, PAC is only partially effective, and may take hours of contact time for the adsorptive process.

Oxidative Pretreatment / Alternative Disinfectants

Oxidative pretreatment is the practice of dosing the filter feed water with an oxidant. The application of hypochlorite to the feed water practiced by Homer is an example of this. Common oxidants used include chlorine, chlorine dioxide, potassium permanganate, and ozone. The objective of oxidant application is to impair the ability of the algal organisms to excrete extracellular polymeric substances (EPS) that (1) enable algal cells to attach to wetted surfaces and subsequently colonize and grow additional biomass, and (2) interfere with charge neutralization in the downstream coagulation process.

Alternative disinfectants (other than hypochlorite) may be used to reduce DBPs in the finished water. Options may include ultraviolet light (UV) chloramination and ozonation, which typically produce lower concentrations of DBPs than chlorination. However, the literature includes reports of case studies where some species of DBPs have increased because of deploying alternative disinfection processes. Therefore, it would be prudent to verify the performance of a given alternative disinfection process before its full-scale implementation. Depending on the alternative disinfection process deployed, chlorine may still be required to maintain entry to and distribution system residual requirements (minimum 0.2 mg/L and trace, respectively). Remote chlorination points are also an option to maintain distribution system residual.

Advantages

The advantage of pre-oxidation is that filter clogging by algal biomass may be reduced and coagulation effectiveness may be increased. Use of certain oxidative pretreatments and disinfectants may decrease DBP formation.

Disadvantages

Disadvantages of oxidation are that it can cause cell stress or damage leading to the release of intercellular material. This intercellular material can, in turn, cause taste and odors in finished water, and/or include substances that are toxic to humans [(1)]. To minimize this, several approaches have been tried, including limiting the concentration of oxidant applied and using downstream processes to remove any intracellular organic material released by the algae.

Where chlorine is used as the oxidant, prechlorination of algae-laden waters can increase halogenated DBP formation.

Some alternative disinfectants have been shown in certain cases to increase formation of DBPs, thus requiring verification of performance before implementation.

Application of Algicides

As with oxidative pretreatment, the application of algicides to filter feed water is also intended to disrupt the algal cell's metabolic processes, including its ability to excrete EPS and interfere with the coagulation process. Typical concentrations of copper sulfate deployed for this purpose are 0.5 mg/L.

Advantages

Most algicides are very efficient in inactivating algae. Newer algicide products are NSF approved for use in drinking water – provided the concentrations used are below the maximum listed values published by the manufacturer.

Facilities with raw water basins, or other such infrastructure, have reported success dosing the raw water with copper sulfate and, to a lesser degree, chlorine or potassium permanganate, to provide solids removal in a presedimetation process[(1)].

Disadvantages

Copper-based algicides must be used carefully in systems where distribution system copper concentrations are near the action level of 1.3 mg/L set by the State and Federal LCR. In addition, requirements of 18 AAC 70, Water Quality Standards, are applicable.

Copper-based algicides can result in WTP residual sludge that has copper concentrations higher than allowed for land disposal practices under existing sludge disposal regulations. To manage this may require frequent solids removal from the waste backwash water lagoons.

Depending on water quality, copper-based algicides may exhibit toxicity to non-target freshwater organisms. Higher concentrations of algicides may also cause cell stress or damage, leading to the release of intercellular material with implications as discussed previously.

Alteration/Optimization of Filtration Media and Coagulation Process

Coagulation and mixing processes do not typically affect the integrity of algae cells. As explained by Amirtharajah [(2)], Edwards[(,3)], Dennett[(4)], American Water Works Association[(5)] (AWWA), and Johnson, et al.[(6)], two mechanisms of coagulation occur when using aluminum or iron salts as the primary coagulant. One is charge neutralization and the other is sweep coagulation. Colloidal material and color-forming organic molecules both have net negative surface charges in solution. These surface charges cause these materials to repel one another upon collision in solution.

In charge neutralization, the positively-charged aluminum (or iron) hydrolysis species absorb as surface complexes onto the negatively-charged dissolved organic and/or colloidal particles in the water to neutralize the negative surface charges, thereby allowing them to collide, attach, and become physically larger, more dense particles that can be removed by gravity sedimentation. In sweep coagulation, higher dosages of aluminum (or iron) result in the formation of metal hydroxide precipitates that physically enmesh suspended material, thereby increasing the density of the composite floc to a point where it will settle out of solution. Both modes of coagulation occur to some degree; however, one or the other is dominant – depending upon the aluminum (or iron) concentration and the pH of the coagulated water. Where coagulation occurs via charge neutralization, the rapid mixing equipment must be configured to impart a high intensity turbulence for short durations of less than one second [5]. However, in sweep coagulation, the formation of colloidal floc is insensitive to the level of turbulence, and depends on detention time in the rapid mix process [2,5].

Studies have shown that coagulation of algae by adsorption and charge neutralization is similar to that for colloidal coagulation [1]. Studies have also found that some algae may be removed affectively with lower coagulant dose and sweep coagulation [1]. Generally speaking, concentrations of the coagulant required increase for a given system when seasonal algae occur in the WTP's source water.

Effective coagulation of algae may be accomplished with: trivalent metal salts (alum, ferric chloride, and ferric sulfate); metal salt coagulants preceded by dosing with lime; inorganic polymers (polyaluminim chloride, and polyferric sulfate); or cationic organic polymers. The effectiveness of these coagulants often depends on other water quality characteristics and the speciation of algae in the source water. No single coagulant stands out as more superior than another for algae removal.

An option for increasing filter performance and reducing DBP concentrations in the finished water may be in the deployment of a coagulant other than the alum currently used at the Homer WTP, and consideration of the use of a polymer filtration aid. Development of new coagulant and polymer products by chemical suppliers is ongoing, and many products appear well suited to use on cold, low turbidity waters with elevated concentrations of dissolved organics. The coagulants are referred to by different names, depending upon the manufacturer, but generally are polyaluminum hydroxychlorosulfates with differing levels of hydroxide basicity. Polymers are often cationic, but may also be nonionic or anionic depending on treatment objectives.

In addition, evaluation of media design may be warranted in order to minimize filter clogging. Use of coarse, deep bed, reverse-graded, or sand beds have been found beneficial.

To verify the potential benefit of these alternatives, bench and pilot studies may be conducted. One approach would be to conduct bench scale jar tests and analyze TOC removal efficiency and/or chlorinated filtrate produced from the settled water for DBPs. Another approach would be to conduct column studies on various media alternatives.

Advantages

Identifying an alternate or enhanced coagulation program may enable a better near-term performance to be attained from the existing filtration process.

Disadvantages

It is unlikely that larger, long-term future demands for treated water will be met with the existing direct filtration system operating on either the existing, an alternate, or an enhanced coagulation process.

Extended/Enhanced Filter Backwashing

Some systems that operate on surface water sources with seasonal algae problems report using extended periods of filter backwashing and/or backwashing their filters with chlorinated water to remove the accumulation of algal biomass from the filter media. Homer has reported it uses more frequent backwashing to maintain filtrate turbidity levels within regulated limits.

Advantages

Where water resources are available, and filter washing turbulence is sufficient, additional washing may limit the amount of biomass accumulation in the filter media.

Disadvantages

For Homer and other systems where source water resources are limited, additional filter washing uses more water and will require the eventual development of supplemental water sources.

GAC

As with PAC, GAC is also an activated carbon absorption process. In GAC, the media are granular as opposed to powdered, with average grain sizes of 1 to 2 millimeters, although there are products available in the range of 0.5 to 5.0 millimeters. Base materials and activation processes are similar for both PAC and GAC.

For Homer, GAC would be used as a downstream process to remove organic material that passes the direct filtration process and otherwise would enter the treated water supply.

Typical use of GAC involves a separate pressure vessel in which the GAC is loaded. Feed water is applied to the filter in a manner similar to the City’s existing direct filters. Where the GAC is used exclusively for absorption, the contactors are not backwashed. In applications for reduction of dissolved organic materials related to the presence of algae, the filters are often provided with both a backwash feature and periodic media chlorination.

Advantages

The advantages of GAC are its operational simplicity and compact physical footprint.

Disadvantages

As with PAC, the disadvantages of GAC are that it is not consistently effective in adsorbing the target organic contaminants contributed by algae in the source water. GAC has been found to be only partially effective in removing taste and odors, algal toxins and, where used, is often provided with contact times of several hours. This would require very large contactor vessels for the flows that Homer is planning to process for the future.

Options for Year 2025

Options more suitable for long-term upgrades that may be implemented to meet estimated requirements for the year 2025 are provided below. Depending on the needs of the City, these options may also be suitable for short-term upgrade consideration.

Upgrade to Conventional Filtration (Option 1)

Upgrading Homer's existing, direct filtration plant to a conventional WTP would require the addition of rapid mix, flocculation, and sedimentation unit processes – as well as pumping systems. These unit processes are described below in detail, and include two variations for the flocculation and sedimentation unit processes.

Rapid Mix

Jar testing would be required to determine the predominant mode of coagulation under conditions of both algal events and normal operation, with seasonal variations to select the most suitable rapid mixing device. Typical performance criteria for an in-line mixer are compared to a back-mix style reactor in **Table 12**.

Table 12: Rapid Mixing Performance Criteria

Performance Criteria	Type of Mixer	
	In-Line Blender	Hydraulic Mixing Reactor
Velocity Gradient (G), sec^{-1}	3,000 to 4,000 (2) 3,000 to 5,000 (7)	300 to 1,000 (2) 100 to 400 (5) 700 to 1,000 (7) ; 300 (8)
Detention Time (t), sec	0.1 to 1.0 (5)	1 to 7 (5) : 60 to 120 (7) ; 10 to 30 (7,8)
Input Power or Headloss Required, Hp/mgd	0.5 (7)	0.9 to 1.2 (7) 0.25 to 1.0 (8)

Notes: Hp = horsepower mgd = million gallons per day sec = second

Flocculation

The objective of the flocculation unit process is to produce particles that will have a mass and resulting settling velocity large enough to be removed by gravity. Mixing induced in the flocculation basins promotes collision of flocculant particles that have been stabilized by coagulant addition. These collisions are to allow individual particles to adhere to one another, thereby increasing the mass and settling velocity of the merged floc particles. If the turbulence created by the mixing is too high, floc particles that have become attached to one another will shear, disintegrating into smaller particles with inherently slower settling velocities. If the settling velocities of the floc are too slow, the particles will not be captured in the sedimentation basin, and the solids loading to the filters will be high and result in very short filter runs between backwashing. Conversely, if the turbulence created by mixing in the basin is too low, flocs will settle to the bottom of the flocculation basin.

The structural integrity of flocculant particles is inherently lower for cold, colored, relatively low turbidity waters than for higher turbidity waters with suspensions of colloidal silts or clays. Polymeric flocculant aids added upstream of the flocculation process improve the size and settleabilty of the flocs, but the final particles remain very light, fragile, and are readily sheared. In addition, high molecular weight flocculant aid has been shown to improve algae removal prior to filtration [(1)].

Vertical shaft mechanical mixers used for flocculation have the advantages of being readily accessible, relatively low maintenance units that impose low head loss to the process flow. However, depending upon the design of the impeller, they can create relatively high shearing velocities at the impeller tips. An alternative mechanical mixing configuration is horizontal immersed paddle wheel type stirrers with lower rotating speeds, larger numbers of blades, larger rotating perimeters, and slower peripheral tip speeds, and more localized flow eddies. The disadvantage of this type of mechanical mixer is the maintenance associated with bearings below the water line.

Key performance criteria for the flocculation process are most often the velocity gradient, G, and the detention time, t. These and other common factors are described below.

Velocity Gradient, G

As explained for the rapid mixing process, the velocity gradient is a measure of the degree of turbulence imparted by the mixer to the process flow. It is the square root of the ratio of the power input to the process fluid being mixed, divided by the product of fluid viscosity and the basin volume.

Detention Time, t

The detention time for the process is the theoretical time for the water to pass through the basin under ideal plug flow conditions.

Number of Flocculation Basins or Stages, n

This is the number of mixing shafts, or individual baffled basins, that can be mixed at speeds independent from one another.

Maximum Mixer Impeller Tip Speed

This is the linear velocity of the outermost tip of the mixer impeller in the basins. It is often the highest velocity in the basin and, therefore, can influence the amount of floc shear occurring in the basin. However, the geometry of the mixer impeller determines the amount of hydraulic shear created by the rotating impeller. A radial impeller similar to what is provided in a common jar testing mixer produces very high hydraulic shear at the tip of the impeller. By contrast, an axial flow impeller designed to pump the fluid axially in a direction parallel to the vertical drive shaft can be constructed with vertical blade tip vortex control tabs (similar to those now used on jet airplane wings) that significantly reduce blade tip vortices and the hydraulic shear induced at the tip of the rotating impeller. The impeller blades in the WTP are intended to minimize blade tip turbulence. If they achieve this, then impeller tip speed is not as critical for these units. Reporting the work of others, Sanks [(10)] indicates there is no limiting tip speed for axial flow impellers designed properly.

Baffle Wall Orifice Velocity

Most flocculator basins are separated by partition or baffle walls constructed with orifices or passageways that allow water to pass between the basins. The size of these baffle wall orifices controls the velocity of flow directly, and can contribute to floc shearing.

D/T Ratio

The D/T ratio is comprised of the diameter of the mixer impeller, D, to the basin diameter, T. In the case of a rectangular tank, the tank diameter, T, is the diameter of a circle with area an equivalent to the area of the rectangular tank.

Impeller Pumped Flow Velocity

Each mixer imparts a pumped flow velocity to the water. Depending upon the geometry of the mixer, flow can be radial, axial, or a combination of both. Maximum flow velocities are often stipulated to minimize floc shearing.

Common flocculation performance criteria are listed in **Table 13**.

Table 13: Flocculation Performance Criteria

Performance Criteria	Criteria Used Elsewhere	Unit
Velocity Gradient, G in initial flocculation basin	40 to 60[7] 70[8]	sec^{-1}
Velocity Gradient, G in last flocculation basin	15 to 25[7]; 10[8]	sec^{-1}
Velocity Gradient to Prevent Floc Settling	10 to 15[7]	sec^{-1}
Theoretical Plug Flow Detention Time*	20 to 40[8]; >30[9,11]	min
Number of Flocculation Basins	2 to 4[8]	n
Maximum Impeller Tip Speed	6 to 9[8]; 0.5 to 3[9]	fps
Baffle Wall Orifice Velocity	1.8, initial basins and 1.2 final basins [8] 0.5 to 1.5 final basins [9]	fps
D/T Ratio	0.2 to 0.4[8]	-
Impeller Pumped Flow Velocity	8 initial basins and 2 final basins [8]	fps

It is common practice to provide larger basins with longer detention times for colder, relatively low turbidity source waters. The EPA [11] recommends using at least 30 minutes for locations where source water temperatures are lower than 0.5 degrees Celsius (^{o}C) and turbidities are less than 5 NTU.

Sedimentation

The objective of the sedimentation basin is to allow flocculant particles formed in upstream coagulation and flocculation processes to be removed by gravity settling, so that relatively solids-free water is passed on to the downstream filtration process.

There are several types of sedimentation basins used in water filtration plants, including: conventional downflow gravity sedimentation basins, upflow radial flow basins, reactor clarifiers, sludge blanket clarifiers, and high rate sedimentation using lamella plates or bundles of immersed tubes. The objective of the high rate alternatives is to reduce the vertical settling distance a particle has to travel before contacting a submerged surface, and subsequently being removed from the process flow, thereby reducing size of the sedimentation process significantly.

Regardless of the type of basin used, if sedimentation basins for conventional water filtration plants are operating on cold, colored surface waters, they must be configured to operate at lower surface loading rates than those operating on source waters with colloidal silts or clays, or softening plants that generate lime sludge [8,10]. This is because colored floc created with alum or ferric coagulants inherently have slower settling velocities.

Turbulence in the basin can deteriorate the quality of the basin's settled water quality. Turbulence can result from: changes in process water temperature, wind (if applicable), poorly configured basin inlet or outlet structures, or operation of sludge removal mechanisms [12]. Another factor that can contribute to turbid settled water quality is incomplete removal of sludge accumulated at the bottom of the basin. Settled sludge is discharged to storage facilities at a frequency dictated by treatment objectives. Key performance criteria for the sedimentation process are described below.

Surface Overflow Rate, SOR

The SOR is computed as the hydraulic flow rate applied to the basin, divided by the surface area of the basin. This factor is also theoretically equivalent to the settling velocity of the slowest particle that will be removed by gravity in the basin under ideal, quiescent settling conditions.

Turbidity of Settled Water

The quality of the settled water exiting the settling basin is an operating control parameter used commonly to verify optimal performance of the sedimentation process. The parameter most often used for operational control is turbidity, as it will reflect carryover of suspended flocculant solids from the sedimentation basin to the filters.

Common sedimentation basin performance criteria are listed in **Table 14.**

Table 14: Sedimentation Performance Criteria

Performance Criteria	Value[8]	Unit
Surface Overflow Rate based rectangular basin horizontal flow	0.34 to 1	gpm/ft^2
Surface Overflow Rate based on radial-upflow type	0.5-0.75	gpm/ft^2
Surface Overflow Rate based on reactor clarifier	0.8-1.2	gpm/ft^2
Surface Overflow Rate based on sludge blanket clarifier	0.8-1.2	gpm/ft^2
Surface Overflow Rate based high rate sedimentation (horizontal projection area created at the basin water surface by the plate/tube assemblies)	2-3.5	gpm/ft^2
Turbidity of Settled Water	less than 2	NTU

Notes:
ft2 = square foot
gpm = gallons per minute
NTU = Nephelometric Turbidity Unit

Variations of Flocculation and Sedimentation Unit Processes

Dissolved Air Floatation (DAF) and Microsand-enhanced flocculation (MEF) are variations that may be incorporated into the unit processes of a conventional WTP replacing flocculation and gravity sedimentation (clarification). These processes are presented here due to their decreased footprint requirements for WTP upgrades, and effectiveness at other sites in algae removal. Studies have shown that DAF and MEF may remove algae equally or more efficiently than with conventional gravity sedimentation [1] using alum or ferric sulfate coagulants and cationic polymer if required for charge neutralization [1].

Dissolved Air Floatation - (Option 2)

The low density of algae/floc (1.1 grams per cubic centimeter) [1] makes DAF a practical alternative for solid-liquid separation. Typically, coagulated water enters the flocculation chamber of the DAF unit for a minimum of 5 to 8 minutes of flocculation, before entering the aeration chamber. Flocculation aids are not normally used, because the objective is to float the floc as opposed to making them large enough to settle. The DAF reactor basin is segregated into three sections. The initial section is a contact zone where the flocculated water is mixed with fine bubbles under pressure. This process is intended to promote attachment of bubbles to the floc particles. The second section is the float basin where floc, buoyed by the attachment of fine bubbles, rises to the surface. The third section is the float beach where traveling mechanical scrapers on the water surface move the waste float to the beach for segregation from the basin's clarified underflow, termed subnatant.

DAF typically is best suited to raw water quality less than 50 NTU (up to 100 NTU for short durations), and maximum TOC values of 8 to 10 mg/L. DAF has also been shown to achieve 2 to 4 log removal of *Cryptosporidium* oocysts and *Giardia lambilia* cysts [8]. Advantages of the process are flocculation times are shorter resulting in a smaller footprint, and DAF is considered especially suitable to treatment of nutrient-rich reservoir waters prone to algal events, relatively low turbidity, low alkalinity, and high color. Disadvantages include potential gas stripping prior to filtration or arrangement of the filtrate piping to maintain positive pressure within the filter media, higher energy cost due to subnatant recycle flow (in the range of 8% of WTP flow) for introduction of entrained bubbles into the contact zone, and costs for developing a supply of compressed air.

A comparison of typical design parameters of the gravity sedimentation and the two variations on flocculation and sedimentation processes are provided in **Table 15**, as reproduced from Table 6.1 of *AWWARF Algae Detection and Removal Strategies for Drinking Water Treatment Plants* [1].

Table 15: Typical Design Parameters for Solids Removal

Parameter	Conventional – Gravity Sedimentation	MEF	DAF	Unit
Flocculation Time	20-45	4-7	5-20	min
Flocculation Intensity	10-70	700	30-70	s^{-1}
Clarifier Loading Rate	0.3 – 1.2	16-30	3-16	gpm/ft^2
Clarifier Detention Time	120-240	3-8	5-15	min
Clarifier Depth	8-16	6.6	5	ft

Pilot testing is recommended for both of these options to determine treatment efficiency as well as providing data from which to predict chemical usage for operation cost determination.

Microsand-Enhanced Flocculation (Ballasted Flocculation) - (Option 3)

The MEF process adds a ballasting agent to increase floc density, resulting in higher settling velocities allowing for greater clarifier overflow rates and substantially reduced footprint. Advantages of the process are that less land is required for expansion and clarified water quality is greater than that achieved by conventional gravity sedimentation. Disadvantages include high operation and maintenance cost associated with energy for suspension and recycle of ballast, as well as microsand and polymer use.

Membrane Treatment Processes

Existing Direct Filtration Process Followed by Nanofiltration – (Option 4)

The WTP may be upgraded by following the direct filtration process with a nanofiltration (NF) system. Advantages of NF following a direct filter, is that the filtrate may be treated to remove contaminants of concern such as organics.

Filtration systems followed by nanofilters exist at two different North Slope water sources, each of which operate on cold, colored, low turbidity, low alkalinity surface waters to produce final treated water with a quality that exceeds current and proposed future drinking water regulatory requirements.

NF concentrate is generated continuously as the system operates. NF concentrate flow streams can be disposed of under NPDES permits common for storm-type discharges.

Disadvantages include higher capital costs than for comparative alternatives. However, life cycle costs for a system that produces equal quality finished water are less for NF treatment systems with suitable pretreatment.

Dual Membrane Filtration – (Option 5)

The WTP may be upgraded with a dual membrane filtration system comprised of a microfiltration (MF) membrane followed by NF membrane. The process would produce a permeate that would either be combined with the filtrate from the direct filtration treatment plant, or may be considered for a new treatment facility.

Advantages of the dual membrane treatment process are that it does not rely upon gravity separation of solids, or the formation of a floc particle for removal of source water contaminants. It is a pressure driven membrane filtration process that produces a high quality permeate, generally without operator intervention, regardless of daily or seasonal changes in the source water characteristics. Because the process does not use coagulants, there is no requirement for close operator monitoring and/or control of the source water quality or the effectiveness of the coagulation process.

There is no sedimentation basin and, therefore, no need to check for settled sludge inventory or to waste chemical sludge from the process.

No granular media filter is used in the process; therefore, there is no requirement to monitor for filtrate turbidity and/or color breakthrough. MF filter backwashes occur approximately once every 15 to 20 minutes for a period of less than 2 minutes in an unattended, automated control sequence. NF concentrate is generated continuously as the system operates.

There is no waste chemical sludge generated by a sedimentation process.

Existing, full-scale, dual membrane MF/NF systems at three different North Slope water sources, all of which operate on cold, colored, low turbidity, low alkalinity surface waters produce final treated water with a quality that exceeds current and proposed future drinking water regulatory requirements.

If required, waste MF backwash water and waste NF concentrate flow streams can be disposed of under NPDES permits common for storm type discharges.

Disadvantages include higher capital costs than for comparative alternatives. However, life cycle costs for a system that produces equal quality finished water are less for the dual membrane treatment system.

Desalination (Reverse Osmosis) – Option 6

A new treatment system may consider the purification of seawater for a potable water supply. Reverse osmosis (RO) and desalination is the method of producing pure water by forcing seawater through a semi-permeable membrane across which salts or impurities cannot pass.

The main advantage of a desalination treatment system is the abundant supply of ocean that water may be utilized. Water shortages would not be anticipated and water quality may be relatively stable, depending on the location of the intake. RO treatment is also capable of producing high quality water. The main disadvantages of RO treatment are: capital and operations costs are relatively high, generally due to high pressure operation and potential cleaning requirements; pre- and post-treatment may be required; and potable water recovery rates are relatively low (50% to 75%).

Appurtenant Systems

Depending on the upgrade alternatives and progression of implementation, the City will have to periodically assess the capacity and performance of existing appurtenant systems. Examples of such systems include disinfection and sludge handling.

The current chlorine contact tanks are impacted by parameters variable to each of the treatment systems recommended. In particular, disinfection byproducts precursors may be substantially eliminated by a particular treatment system, thus allowing higher chlorine concentrations and higher flow throughput, such that future capacity expansion may not be required. As such, upon treatment system selection and subsequent water quality determination, the disinfection treatment system, and potable water storage and distribution system, should be re-evaluated to verify system sizing, chlorine dose and residual parameters, and regulatory compliance. New chlorine contact volume was not included in any of the future upgrade costs. ***Section 7*** includes an estimate of cost of a 500,000-gallon chlorine contact tank for inclusion in estimates that eliminate the existing wood stave tank to facilitate WTP upgrades. Cost will vary depending on the location of the replacement tank, its design, and design parameters used.

Water treatment waste waters and sludge management will be impacted by parameters variable to each of the treatment systems recommended. Quality and quantity of waste waters and sludge will be different for each system, and will vary depending on raw water quality fluctuations. As such, upon treatment system selection and subsequent quality and quantity determination, the sludge handling and disposal methods currently used by the City should be re-evaluated to verify system sizing, dewatering methods, disposal options, and regulatory compliance. The City has had previous plans that include a design for a sludge thickening pond. The area identified previously may be used as such, or modified as a sludge drying bed. Depending on disposal options available to the City, the City may wish to develop acquired land adjacent to the WTP to include sludge handling/disposal. Treatment system options include in-plant sludge handling equipment, but do not include costs for a sludge thickening pond/drying bed, on-site sludge handling and disposal area, or on-site sludge handling and disposal equipment.

Section 7: Recommendations for Upgrades

The following sections describe recommendations for short and long-term upgrades. Should the City so choose, on-site assessments, bench scale testing, and/or pilot testing are recommended and would confirm practicality, benefit, and cost of implementation of these upgrade options.

Control of Algae Blooms at Source

Though many options exist to control source water algal blooms, watershed control to limit nutrient entry from sources such as runoff and septic systems, and reservoir circulation may impact the watershed and source water in a manner to yield most benefit with little or no adverse impact.

Some inline filtration systems have been successfully used to filter algae from source waters. One such system uses disc filtration technology and short backwash cycles. Often algae may be removed with a single 130-micrometer filtration stage, or as necessary, include a second stage of a finer filtration grade. If determined viable, this filtration system could be deployed at the raw water pumping station or, with upgrades, at the WTP.

Treatment of Algae-Laden Water with Existing Direct Filtration Process

Implementation of an alternative oxidative pretreatment, alteration and/or optimization of the filtration media, and optimization of the coagulation program may yield the most benefit by reducing use of compounds contributing to DBP formation and increasing the overall efficiency of the filtration process.

Treatment Plant Upgrades

Options suitable for long-term upgrades, which may be implemented to meet estimated requirements for the year 2025, include:

- Upgrading the existing treatment system to conventional filtration (Option 1),
- Upgrading the existing treatment system to a variation of conventional filtration by implementing dissolved air flotation clarification (Option 2),
- Upgrading the existing treatment system to a variation of conventional filtration by implementing

microsand enhanced flocculation clarification (Option 3),

- Upgrading the existing treatment system by following the existing filtration system with NF (or ultrafiltration) treatment system (Option 4),
- Replacing the existing treatment system with a dual membrane (MF followed by NF or ultrafiltration) treatment system (Option 5), and
- Replacing the existing treatment system with a seawater desalination (reverse osmosis) treatment system (Option 6).

Depending on the needs of the City, these options may also be suitable for short-term upgrade consideration, as well as phased construction. On-site assessments, bench scale testing, and/or pilot testing are recommended and would confirm the practicality, benefit, and cost of implementation of these options.

A scoring matrix developed to rank the treatment alternatives discussed in this report. The matrix uses a point system to evaluate both the quantitative and qualitative elements of each treatment alternative. A point system ranging from 1 through 3 was used, and the higher the score, the better the alternative ranks relative to the other alternatives.

As example, for the cost criteria, a score of “3” indicates that the costs are low relative to the other treatment alternatives (see Section Comparative Costs for Alternative System Upgrades). Alternatives with a score of “2” have average costs relative to the other alternatives. Alternatives with a score of “1” have above average costs relative to the other alternatives.

For the qualitative criteria, a score of “3” represents the preferred alternative of above average performance. A score of “1” indicates below average performance in that category. Scores for each category were assigned based on cost estimates, technical considerations, and experience.

Each treatment alternative was scored based on the following criteria:

- Land Acquisition – Land acquisition required for construction of buildings for upgrades.
- Capital Cost – The cost of construction based on the project team’s opinion.
- Operations and Maintenance Cost – The estimated cost of labor, chemicals, and power.
- Ease of Operation – The amount of training required to successfully and consistently operate the facility. A score of “3” indicates the plant requires less operator training and attention.
- Reuse of Existing Equipment – The extent to which the existing infrastructure may be integrated into the upgraded system. A score of “3” indicates the majority of the existing equipment will be integrated into the upgrade.
- Ease of Expansion – The ability of the WTP to be expanded in the future. A score of “3” indicates the treatment alternative can be expanded easily to treat increased water demands in the future.
- Treated Water Quality – The quality of the treated water (DBP precursors and turbidity removal) that can be achieved consistently relative to the other alternatives. A score of “3” indicates the technology will consistently produce a high quality effluent.
- Waste Waters Generation/Recovery – The amount of waste waters (including backwash, filter to waste, cleaning, and concentrates) generated, and the recovery of treated water compared to raw water reflect on the efficiency of the water treatment process though not the quality of the treated water produced. A score of “3” indicates the technology creates little waste waters and has high recovery.

- Waste Solids/Sludge Production – The amount of waste solids and sludge generated by the technology and disposal method. A score of “3” indicates a relatively low volume will be generated by the treatment alternative, with an associated ease of disposal. A score of “1” indicates a relatively high volume will be generated by the treatment alternative, with associated difficulties of handling, storage and disposal.

In addition to the scoring described above, a weighting system was applied in order to reflect the City’s prioritization or emphasis on importance of the quantitative and qualitative elements of the ranking system. A weighting factor of “1.3” indicates the City applies a higher priority or emphasis on that element, relative to the base weighting of “1”. The scoring matrix for the various upgrade options is presented in Table 16.

Table 16: Upgrade Options - Decision Matrix

Technology	City Weight	CF Upgrade Option 1	DAF Upgrade Option 2	MEF Upgrade Option 3	DF/NF Option 4	MF/NF Option 5	RO
Land Acquisition	1	1	3	3	3	1	1
Capital Cost	1	2	3	3	3	2	1
O&M Cost	1	2	2	1	1	2	1
Ease of Operation	1.3	1	1	1	1	2	2
Reuse of Existing Equipment	1.2	3	3	3	3	1	1
Ease of Expansion	1.1	1	2	2	2	3	3
Treated Water Quality	1.3	1	1	1	3	3	3
Waste waters generation/Recovery	1.1	3	3	3	2	2	1
Waste solids/Sludge Production	1.3	2	2	1	2	3	3
TOTAL		16	20	18	20	19	16
Total with City Weighting		18.2	22.3	20	22.8	22.1	19

The land acquisition category above includes land assumed to be available adjacent and to the east of the existing WTP parcel. In the case of a desalination plant, a parcel near the ocean would be necessary. It does not include land the City may wish to acquire for all possible treatment alternatives, or for future land disposal of water treatment sludge.

As can be determined from **Table 16**, the treatment options with the highest scores (based on the evaluation of both quantitative and qualitative elements and including City weighting) include:

- Upgrading the existing treatment system to a variation of conventional filtration implementing DAF clarification (Option 2),
- Upgrading the existing treatment system by following the existing filtration system with NF (or ultrafiltration) treatment system (Option 4), and
- Replacing the existing treatment system with a dual membrane (MF followed by NF or ultrafiltration) treatment system (Option 5).

These three options above are identified by option number and schematically illustrated on the WTP site plans, **Sheets 31A, 31B, and 31C** respectively, in the Water and Wastewater Master Plan.

As detailed in the following paragraphs, selection of any of the treatment recommendations or treatment system upgrade options will be best determined by conducting on-site assessments, bench scale testing, and/or pilot testing.

On-site Assessments, Bench Scale Testing, and Pilot Testing

Pre-design on-site assessments, bench scale testing, and/or pilot testing of any major alteration or addition to the existing treatment process is a recommended precautionary practice. Items to consider in the protocol of these investigations are discussed below for some of the treatment options presented in this section.

Algae Treatment

Depending on treatment systems selected and phasing developed, pilot testing of algae treatment or removal alternatives should be conducted. Data collected will be used to determine:

- Effectiveness of treatment method at source water reservoir, such as reduction of algae counts, and
- Filtration micrometer exclusion requirements.

Existing Filtration System

If an expansion of the existing direct filter system is to be pursued, pilot testing should be based on the recommendations of the American Water Works Associations (AWWA) for bench scale coagulant evaluation of surface waters like Homer's. Data collected will be used to determine:

- Mixing method and detention time,
- Optimum chemicals (e.g. coagulant, coagulation/filtration aid, pH/alkalinity adjustment), dosages, and demand,
- Coagulation and flocculation detention time,
- Settling characteristics,
- Filtration hydraulic loading rates verified in filter column evaluations, and
- Settled and filtrate water quality.

If the DAF or MEF clarification systems are to be considered in a conventional treatment plant, replacing flocculation and gravity sedimentation, the following items should be investigated in a pilot testing effort:

- Verification of chemical and ballast dosage and required detention times,
- Air flow and subnatant recycle flow rates for optimum flotation,
- Hydraulic loading rates,
- Filter hydraulic loading rates using subnatant from the DAF reactor or clarified effluent from the MEF, and
- Clarified and subnatant water quality.

Membrane Treatment

Pilot testing efforts for any membrane treatment alternative should include identification of the following:

- Fouling potential and modes,
- Effective CIP regimens,
- Transmembrane pressures for colder water operations,
- MF flux rates and associated CIP intervals,
- Recovery rates,
- Waste production,
- MF filtrate quality including turbidity and silt density index,
- Net permeate production as a function of feed water temperature, and
- Confirmation of membrane integrity testing methods.

Comparative Costs for Alternative System Upgrades

Rough order of magnitude opinions of project costs for the upgrades reviewed in this report are presented in **Attachment A-1** of this Appendix. These costs are not based on preliminary design, include a significant contingency factor, and represent a rough order of magnitude accuracy range of +30% to -15% of the actual cost of the work. All cost data presented are in year 2006 dollars.

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ATTACHMENT A-1

Water Treatment Plant Cost Estimates

HOMER WTP - EXPANSION FOR YEAR 2006
ROUGH ORDER OF MAGNITUDE COST OPINION
(ACCURACY RANGE: +30%/-15%)

SUMMARY

OPTION	DESCRIPTION	TOTAL COST	Notes
1	Expansion of Existing Filtration Plant to Conventional	$6,393,114	
2	Dissolved Air Flotation (DAF) - Option for Conventional Filtration	$4,431,837	
3	Microsand Enhanced Flocculation (MEF) - Option for Conventional Filtration	$6,494,003	
4	Expansion of Existing Filtration Plant - Direct Filters Followed by NF	$6,353,033	
5	Dual Membrane: MF / NF (or UF)	$7,714,180	

SUMMARY

HOMER WTP - EXPANSION FOR YEAR 2025
ROUGH ORDER OF MAGNITUDE COST OPINION
(ACCURACY RANGE: +30%/-15%)

OPTION	ITEM DESCRIPTION	TOTAL COST	Notes
1	Expansion of Existing Filtration Plant to Conventional	$10,431,765	
2	Dissolved Air Flotation (DAF) - Option for Conventional Filtration	$7,673,371	
3	Microsand Enhanced Flocculation (MEF) - Option for Conventional Filtration	$11,653,260	
4	Expansion of Existing Filtration Plant - Direct Filters Followed by NF	$6,900,200	
5	Dual Membrane: MF / NF (or UF)	$8,624,526	

APPENDIX B

Homer Wastewater Treatment Plant Review

HOMER
WASTEWATER TREATMENT PLANT REVIEW

Prepared for
Bristol Environmental and Engineering Services, Corporation

for the
City of Homer

July 2006

GV Jones & Associates, Inc.
WATER AND WASTEWATER PROCESS ENGINEERS

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Final

Table of Contents

Figures

Tables

Attachments

Wastewater Treatment Plant Review

Section 1: Introduction

The City of Homer (City) operates a variation of an activated sludge wastewater treatment plant (WWTP) called a deep shaft reactor. It is configured with pretreatment including screenings and grit removal, floatation clarification, and sludge digestion and dewatering. Clarified effluent is ultraviolet radiation (UV) disinfected prior to ocean discharge.

Within the last two years, operators reported influent waste strength increases and corresponding increases in effluent parameters five-day biochemical oxygen demand (BOD_5), temperature, and fecal coliform. Wet weather is also reported to affect the treatment plant periodically and is dependent on conditions during the wet weather event, such as freeze-up or break-up conditions in addition to intensity and duration of rainfall. During such events, operators note high flow, low temperature, and dilute influent waste strength; with resulting high effluent levels of BOD_5 and total suspended solids (TSS). Operators indicate during some wet weather events the outfall is not able to adequately convey flow causing effluent to overflow into the empty clarifier.

The wastewater treatment plant was designed to treat an annual average flow of 730,000 gallons per day (gpd) up to a peak month average of 880,000 gpd, and a maximum of 1,430,000 gpd at average and maximum influent BOD_5 and TSS waste strength of 225 and 326 mg/L; and 317 and 488 mg/L, respectively.

The objective of this report is to review options for upgrading the existing WWTP to provide the City with wastewater treatment capacity for future flow conditions, and to meet current and future regulatory requirements.

This *Wastewater Treatment Plant Review* section is structured with an initial descriptive review of the existing system followed by a discussion of the operational problems noted by the City. Based on data collected during a site visit and subsequent data provided by the operations staff, an evaluation of individual unit processes is presented. A section is also included on the regulatory outlook for the facility. Finally, a section is included that addresses alternative upgrade programs to meet future flow treatment requirements, and current and future regulatory requirements.

Section 2: Description and Operation of the Existing Wastewater Treatment Plant

The Alaska Department of Environmental Conservation (ADEC) and the U.S. Environmental Protection Agency (EPA) have authority to regulate and permit the discharge of treated wastewater from a community treatment facility. Under federal law, communities discharging to the land or waters of the state or the ocean are obligated to meet secondary effluent standards. The City of Homer operates the wastewater treatment plant under EPA National Pollution Discharge Elimination System (NPDES) permit number AK-002124-5 (expiration date August 1, 2005). Effluent is discharged to Kachemak Bay through one outfall identified as 001.

The EPA has not yet granted the State of Alaska the authority to administer federal pollution discharge regulations (primacy); though the State has authority to grant mixing zones through the process of issuing Certification of Reasonable Assurance for NPDES permits. The State of Alaska has adopted the federal requirements into the Alaska Administrative Code (AAC) contained in Wastewater Disposal (18 AAC 72), Water Quality Standards (18 AAC 70) and Water and Wastewater Operator Certification and Training (18 AAC 74). More details may be found in the Clean Water and Water Quality Acts.

A summary of the effluent limitations and monitoring requirements specified in the City's NPDES permit are shown in **Table 1** and **Table 2** respectively.

Table 1: Current NPDES Effluent Limitations (Expires August 1, 2005)

Parameter	Monthly Average	Weekly Average	Daily Maximum
Flow, mgd	--	--	0.88
BOD_5, mg/L (lb/day)	30 (220)	45 (330)	60 (440)
BOD_5, percent removal	≥ 85	--	--
TSS, mg/L (lb/day)	30 (220)	45 (330)	60 (440)
TSS, percent removal	≥ 85	--	--
Fecal Coliform, #/100mL	200	400	800
pH (standard units)	--	--	6.5 - 8.5
Total Chlorine Residual, mg/L	--	--	<0.1

Table 2: Current NPDES Monitoring Requirements (Expires August 1, 2005)

Parameter	Sample Location	Sample Frequency	Sample Type
Flow, mgd	Effluent	Continuous	Recording
Biochemical Oxygen Demand (BOD_5)	Influent & Effluent	2 per month	24 hour composite
Total Suspended Solids (TSS)	Influent & Effluent	2 per month	24 hour composite
Fecal Coliform	Effluent	4 per month	Grab
Fecal Coliform (may be discontinued after 2 years acceptable results)	Mixing Zone	1 per month for May, June, July, August, September	Grab
pH	Effluent	5 per week	Grab
Temperature, degrees C	Effluent	5 per week	Grab
Total Chlorine Residual (when in use)	Effluent	Daily	Grab
Ammonia as Nitrogen, mg/L	Effluent	1 per month	24 hour composite

Grab – a grab sample, for monitoring purposes, is a single sample or measurement taken at a specific time.

24-hour composite – a 24-hour composite sample is a mixture of not less than 8 discrete aliquots. Each aliquot shall be a grab sample of not less than 100 ml.

The current NPDES permit further required the City to complete the following: update of the Quality Assurance Plan, update of the Sludge Application, and place outfall/mixing zone signs. The City reports the completion of these items.

The following paragraphs describe the configuration and operation of the wastewater treatment plant.

Current and Future Wastewater Flows

Current and future wastewater flow rates are are discussed in detail in other sections of this master plan and are summarized here for purposes of this wastewater treatment plant review.

Over the period of 2001 through July of 2005, the maximum flow to the wastewater treatment plant was 1,739,000 gpd recorded on October 24, 2002. Flow to the wastewater treatment plant averaged approximately 409,000 gpd. Projected flows for 2025 are summarized in **Table 3**.

Table 3: 2025 Projected Flows (1/01-7/05)

Parameter	Value
Minimum day flow (winter)*	750,000 gpd
Minimum day flow (summer)*	1,100,000 gpd
Average day flow*	1,200,000 gpd
Average day flow (includes I/I**)	1,350,000 gpd
Maximum day flow	3,900,000 gpd
Peak hour flow	(~2000 gpm)

*flows include baseline sewage and infiltration
** I/I – Infiltration and Inflow

Performance of Wastewater Treatment Plant

The performance of the existing wastewater treatment facility was evaluated using data collected by plant personnel from January 2001 through July 2005. The results summarized in **Table 4** show that the WWTP generally produced an effluent within permit limits. The daily fecal coliform limit was exceeded three times at values of 2,000; 6,200; and 1,380 cfu/100 mL. The two highest fecal coliform values appeared to correspond to upset high flow events.

Table 4: WWTP Performance Summary (1/01-7/05)

Parameter	Average	Median	Maximum	Minimum	90^{th} Percentile*
Flow, mgd	0.408	0.393	1.739	0.244	0.483
Influent BOD_5, mg/L	262	247.5	749	55	385
Effluent BOD_5, mg/L	15.6 (94% Removal)	14	39	6	24
Influent TSS, mg/L	285	267	1,331	106	387
Effluent TSS, mg/L	15 (95% Removal)	13	46	6	21
Effluent pH	7.2	7.2	7.7	6.8	7.3
Effluent Fecal Coliform (cfu/100 mL)	92.1	30	6,200	1	118

* Indicates that 90% of the data recorded during the analysis period were less than this amount.

Table 5 summarizes the average daily flows by month from January 2001 through July 2005. The peak month for each year is highlighted in yellow. Over the 4-1/2-year period, the average daily flow for the peak month ranged from a low of 0.42 mgd (February/July 2003) to a high of 0.58 mgd (January 2001). The average daily flow for the peak month over the 4-1/2-year period was approximately 0.5 mgd. The highest peak instantaneous flow measured was 2.88 mgd in October and November of 2002.

Table 5: Average Daily Wastewater Flow (by Month) from 2001-June 2005

Year	Jan.	Feb.	March	April	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
2001	0.58	0.44	0.46	0.39	0.39	0.38	0.42	0.42	0.39	0.35	0.35	0.41
2002	0.43	0.39	0.40	0.43	--	0.40	0.41	0.41	0.39	0.50	0.55	0.41
2003	0.41	0.42	0.36	0.39	0.37	0.40	0.42	0.41	0.37	0.39	0.35	0.40
2004	0.32	0.42	0.40	0.51	0.41	0.42	0.44	0.39	0.35	0.42	0.42	0.42
2005	0.35	0.32	0.41	0.42	0.38	0.41	0.43					
Ave.	**0.42**	**0.4**	**0.41**	**0.43**	**0.39**	**0.4**	**0.42**	**0.41**	**0.37**	**0.41**	**0.42**	**0.41**

Figures 1 and 2 plot the influent BOD_5 and TSS concentration as a function of flow rate using data collected between 1/1/01 and 7/31/05. **Figure 3** plots the BOD_5 loading as a function of time for the same period.

The scatter apparent in the flow versus TSS and BOD_5 loading data make identification of trends in these three parameters difficult. There does appear to be a general decreasing trend in TSS concentration as the flow rate increases. A similar relationship is not apparent for the influent BOD_5 concentration. The BOD_5 loading data shown in **Figure 3** is also scattered but a visual inspection of these data does indicate an increasing trend in the extreme high values over time.

Figure 1: Influent BOD_5 vs. Flow

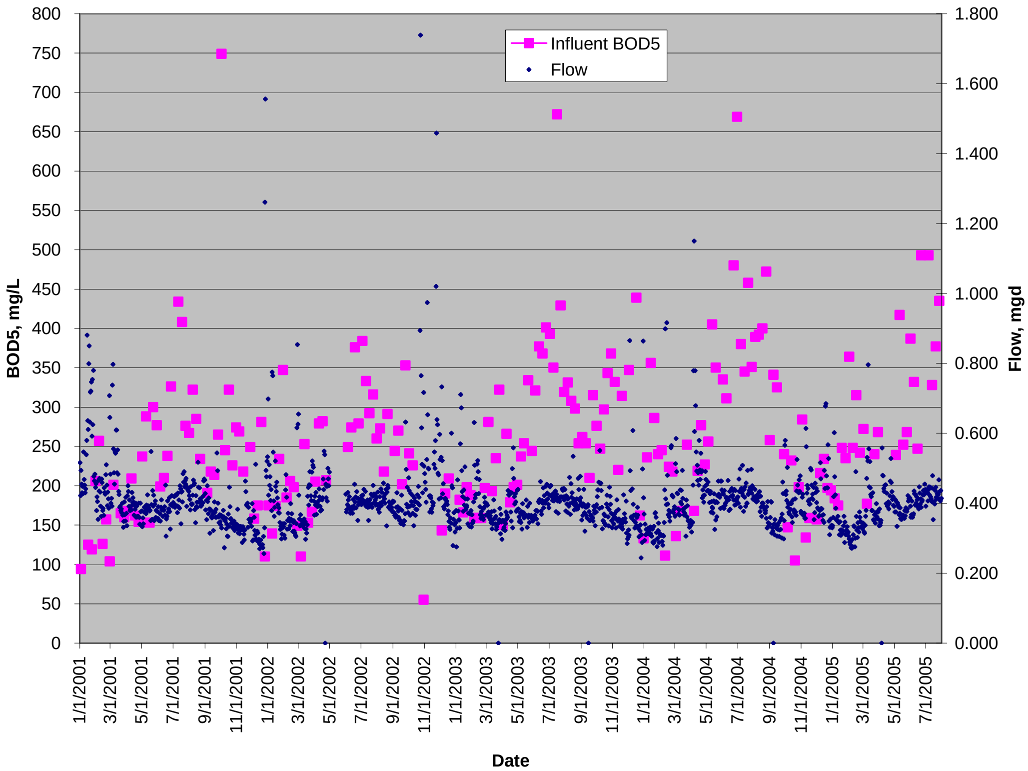


Figure 2: Influent TSS vs. Flow

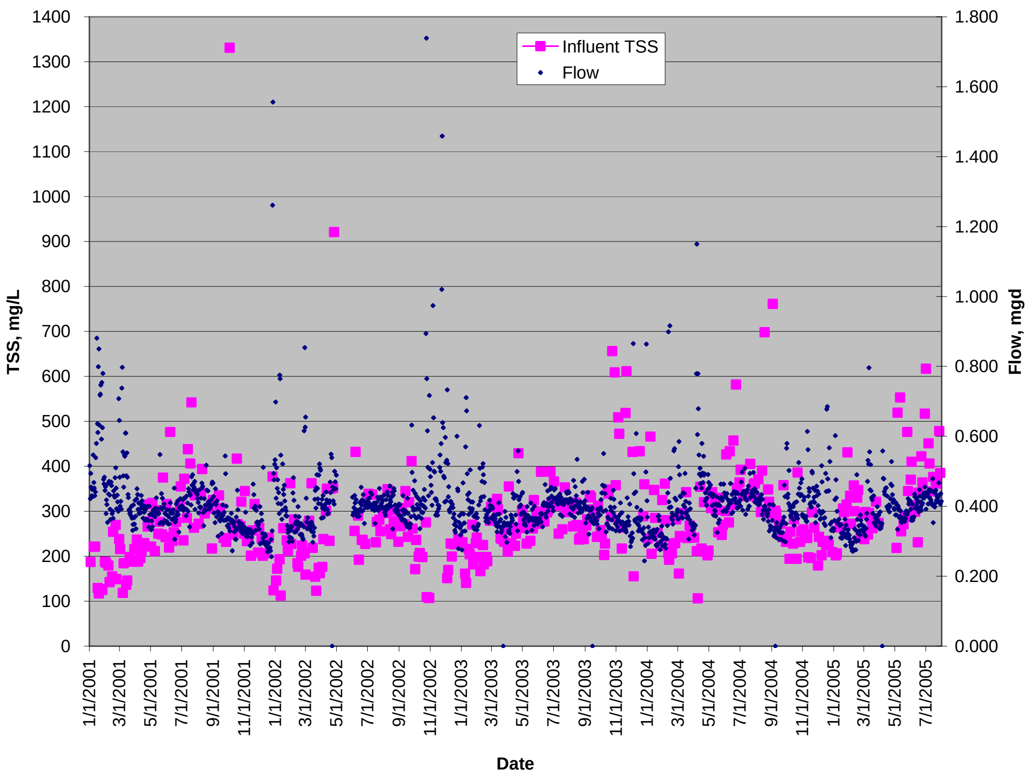
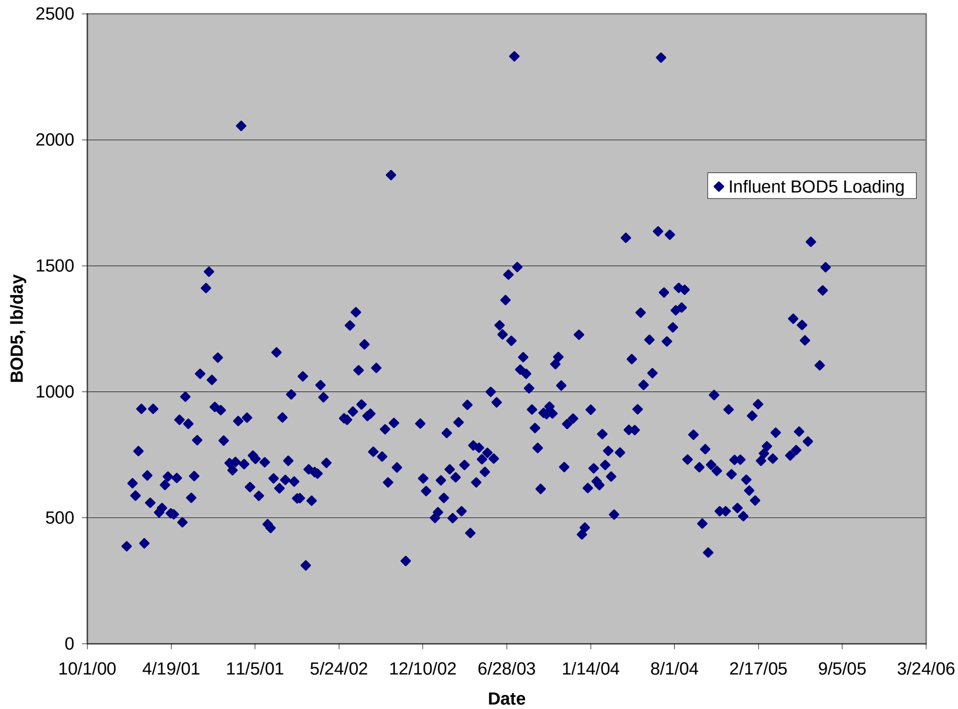


Figure 3: Influent BOD_5 Loading over Time



To help discern any trends in the data, values for the yearly average and 90th percentile value for flow, TSS, influent BOD_5 concentration, and BOD_5 loading were calculated for each year from 2001 through 2004. The results are summarized in **Table 6**.

Comparison of the yearly average values for flow rate and 90th percentile flow rate indicate no significant change in either from 2001 to 2004. The yearly average values for influent TSS and BOD_5 concentration have increased less than 10% from 2001 to 2004. Likewise, the 90th percentile value of TSS has varied less than 10% over the 2001-2004 period.

In contrast, the 90th percentile BOD_5 concentration has steadily increased about 20% from 2001 to 2004. Yearly average BOD_5 loading increased about 12% and the 90th percentile BOD_5 loading increased about 20% from 2001 to 2004. The increases in 90th percentile data for these two parameters appear significant, and suggest more frequent higher BOD_5 concentrations and loadings are being received at the plant.

Table 6: Yearly Average and 90th Percentile Values for the Homer WWTP

	Flow (mgd)		Influent TSS (mg/L)		Influent BOD_5(mg/L)		BOD Loading (lb/day)	
Year	average	90th Percentile*	average	90th Percentile*	average	90th Percentile*	average	90th Percentile*
2001	0.41	0.49	267	370	235	322	784	1097
2002	0.43	0.51	253	349	234	336	839	1113
2003	0.39	0.45	293	388	277	379	898	1231
2004	0.41	0.49	293	387	258	403	882	1410
2005**	0.39	0.45	338	476	302	430	962	1467

* Indicates that 90% of the data recorded during the analysis period were less than this amount.

** Data through July 2005. Does not represent yearly average.

Speculation as to the sources contributing to the increase in influent BOD_5 values may include but are not limited to recreational vehicle dumping corresponding to high summer tourism, increasing contribution of high strength landfill leachate, and/or variation of wastewater quality as Homer grows (restaurant, industry, and commercial).

Existing Plant Configuration and Operation

Influent Lift Station

The WWTP raw wastewater influent lift station is located outside of and adjacent to the main wastewater treatment plant and is configured with four submersible, variable frequency driven pumps, each rated at 625 gpm at 46-feet total dynamic head. Air from the lift station is processed through an odor control scrubber. The lift station is intended to process a peak instantaneous flow of 2.7 mgd with three pumps in operation and one as a backup.

Influent Flow Meter

Raw wastewater is metered with a magnetic flow meter installed within the building at the headworks. The magnetic flow meter is a 10-inch Sparling Waterhawk, Model No. FM621.

Screening

Large wastewater solids are removed to reduce equipment maintenance and improve treatment process performance by means of 0.625-inch opening size headworks bar screen. The screen is manufactured by John Meunier, Model No. RCR-21XGAA, 1 Hp. Design operating flow conditions are: 1.735 mgd average and 2.7 mgd maximum. Screenings are dewatered, compacted, and collected in a hopper prior to incineration and final disposal.

Air from the headworks screening room is processed through an odor control scrubber.

Grit Removal

Discharge from the screenings process is direct to a vortex-style grit removal system, manufactured by Eutek, including a 62-inch TeaCup grit separator and a decanter dewatering unit. The unit is rated for 150-micron removal at 1.7 mgd flow with 18-inch headloss. Peak capacity of the unit is 2.7 mgd with 46-inch headloss.

Degritted wastewater is discharged to the deep-shaft reactors. Collected grit is pumped by means of two grit pumps through a grit grinder to the aerated sludge lagoons. Grit Pumps are WEMCO

TorquFlow, Model 4x11 CF, rated at 132/150 gpm at 32/23 total dynamic head and 810/960 rpm. Grit grinder is a Volgesang RotaCut Model RC3000S E1; Motor: 1,720 rpm and 2.5 kW.

Wastewater Treatment

The wastewater treatment system consists of two activated sludge deep shaft reactors and floatation clarifiers. As the name implies, a 500-foot deep, 30-inch diameter vertical shaft fitted with an 18-inch diameter concentric pipe (downcomer) forming an annular reactor, replaces the standard aeration basin. The configuration is such that mixed liquor circulates through the reactor with injected air, down the center of the shaft (downcomer) and upward through the annulus (space between shaft and downcomer).

Clarification

Reactor effluent is dosed with polymer prior to floatation clarification for sludge removal. The current polymer is Hydrofloc 1665. Target dose of polymer is 5 mg/l. Polymer feed equipment consists of both dry and liquid feed equipment by Polyblend, Stranco and Accurate.

The plant is constructed with two clarifiers each of dimensions 13.92-feet wide by 75-feet long and 15-feet deep. The design average overflow rate is 420 gallons per square foot per day (gpsfpd) and 820 gpsfpd at peak hour flow. Design solids loading rate is 21 pounds per square foot per day. The clarifier drive is a Rex, 3 Hp. Currently only one clarifier is on line per operators preference.

Sludge is wasted to the aerobic digester by means of four waste sludge pumps rated at 10 to 40 gpm at 18-foot total dynamic head.

UV Disinfection

Treated effluent is disinfected with ultraviolet (UV) continuous wave irradiation (radiation at a wavelength of 254 nm). UV irradiation causes the inactivation of microorganisms by the photochemical breakdown of cellular nucleic acids (DNA). A typical UV dosage for an activated sludge treatment system effluent is 15 – 140 $mW\cdot s/cm^2$ for 7 to 14 seconds of contact time. UV disinfection produces no chemical residual and it is currently believed that no toxic UV-produced compounds are formed. Operation and maintenance of UV units primarily consist of periodic bulb replacement and quartz-sleeve cleaning.

The current design provides a UV dose of 30 $mW\cdot s/cm^2$ at a peak flow of 1,200 gpm.

Aerobic Digestion

Sludge is digested in an indoor aerobic digester and stored in an outdoor aerated lagoon prior to dewatering and disposal.

Septage Receiving Station

No septage is currently received at the wastewater treatment plant. A septage receiving station was constructed at the treatment plant but is not used due to concern of organic overloading of the existing treatment plant. The record drawings do indicate the ability to pump septage to the aerated sludge lagoon. Two recreational vehicle dump stations are provided on the City's sewer system, one upstream of the WWTP and the other near the Spit fishing hole.

Currently, septage from the pumped effluent septic tank system in Kachemak City is received every two years and processed in the digesters at the WWTP. Other septage and holding tank wastes are collected and processed by private haulers.

Sludge Lagoon and Sludge Drying Beds

An outdoor aerated lagoon provides septage and sludge storage in addition to aerobic digestion. The lagoon is periodically dredged and polymer conditioned sludge is pumped to outdoor, covered sludge drying beds. After drying, sludge is removed from the beds and beneficially land applied. On average, the City has beneficially land applied approximately 275 cubic yards per year (1996-2004) of treated sludge.

Section 3: Operational Problems

The following paragraphs review the performance problems identified in our discussions with operations staff and the City. These issues do not include the results of the plant capacity analyses addressed in a subsequent section of this report.

As discussed in the introduction of this section, operators reported influent waste strength increases and corresponding increases in effluent parameters five-day biochemical oxygen demand (BOD_5), temperature, and fecal coliform. In addition, during wet weather flow events, operators note high flow, low temperature, and dilute influent waste strength; with resulting high effluent levels of BOD_5 and total suspended solids (TSS).

Operators report on occasion, the outfall is not able to convey wet weather flow events adequately causing overflow to the currently empty clarifier. Operators note the old landfill is connected to the sewer system and may substantially contribute to wet weather flow events.

Operators report sludge handling has become more burdensome due to sludge handling and maintenance costs associated with the sludge drying beds. In addition, future options for land application, and disposal at a solid waste landfill or monofill have become more limited and expensive.

Section 4: Unit Process Evaluation

An evaluation was performed on each of the major unit processes that comprise the existing WWTP and a summary presented. Any assumptions required to estimate unit process capacity were based on manufacturer’s information, manuals of practice, peer-reviewed journal articles, and past experience.

Wastewater Treatment

To evaluate the deep shaft wastewater treatment facility, a hydraulic and loading analysis was conducted using data provided by the treatment plant operations staff for the last 4 years. **Table 7** summarizes the results of this analysis.

Based on the wastewater flow rates reported from 2001-2005, the Homer WWTP appears to be well within its design constraints from an average hydraulic standpoint. The current average annual daily flow and average daily flow for the peak month are well below the values specified in the original design documents for the years 1997 and 2007.   As a result, the deep shaft reactors are operating within their design hydraulic residence time. And, with the operation of <u>both</u> floatation clarifiers, surface overflow rates are within the design parameters.

The treatment facility also appears to be within the design limits for organic loading. Even though the strength of the waste received at the facility appears to be increasing (see **Table 6**), the BOD_5 loading to the facility has remained below the design values.

Table 7: Design Criteria and Current Operating Parameters for the Deep Shaft Reactors

Parameter	Design Value	Current Operation
Average Day Peak Month Flow, mgd	0.62 (for 1997) 0.88 (for 2007)	0.5 mgd (1/01-7/05)
Average Annual Daily Flow, mgd	0.52 (for 1997) 0.73 (for 2007)	0.409 mgd
BOD_5 Loading – Average Day Peak Month, lb/day	1,180 (for 1997) 1,645 (for 2007)	586-1,464 (1/01-7/05)
Influent Pumping 3 pumps on, gpm	1,875	--
Screening, average & peak flow, mgd	1.7 / 2.7	--
Grit Removal, average & peak flow, mgd	1.7 / 2.7	--
Hydraulic Detention Time, hour	1.0 hr at average daily flow in the peak month	1.1 hr at average flow rate of 0.408 mgd, 0.79 hr at average day peak month (based on deep shaft volume alone)
Mean Cell Residence Time, days	2.0 days	2 days (as reported by operators on 1/19/05)
Food to Microorganism Ratio, mg/L BOD_5 / lb MLVSS	1.0	0.64 (average in 2001) to 0.78 (avg. in 2004)
Mixed Liquor Suspended Solids, mg/L	6,600	5,000 mg/L (as reported by operators on 1/19/05)
Mixed Liquor Volatile Suspended Solids, mg/L	5,300	4,000 mg/L (assumed 80% of MLSS is volatile)
Downcomer Velocity, ft/s	3.5 to 4	
Riser Velocity, ft/s	2 to 3	
Air Required per Shaft at 100 psi, scfm	130	15 downcomer, 60 riser
Clarifier Surface Overflow Rate, gal/ft^2/day at Ave. Day of Peak Month	420	480 (one clarifier operating)
Digester Loading at 3.5% solids, gpd	6,930	--
Disinfection Capacity at 30,000 mW sec/cm^2 , gpm	1,200	--

Summary of Individual Major Unit Process Capacities

Figure 4 summarizes the capacity of the major unit processes used in the wastewater treatment plant as well as various design and daily log flow rates and compares them to predicted 2025 flows.

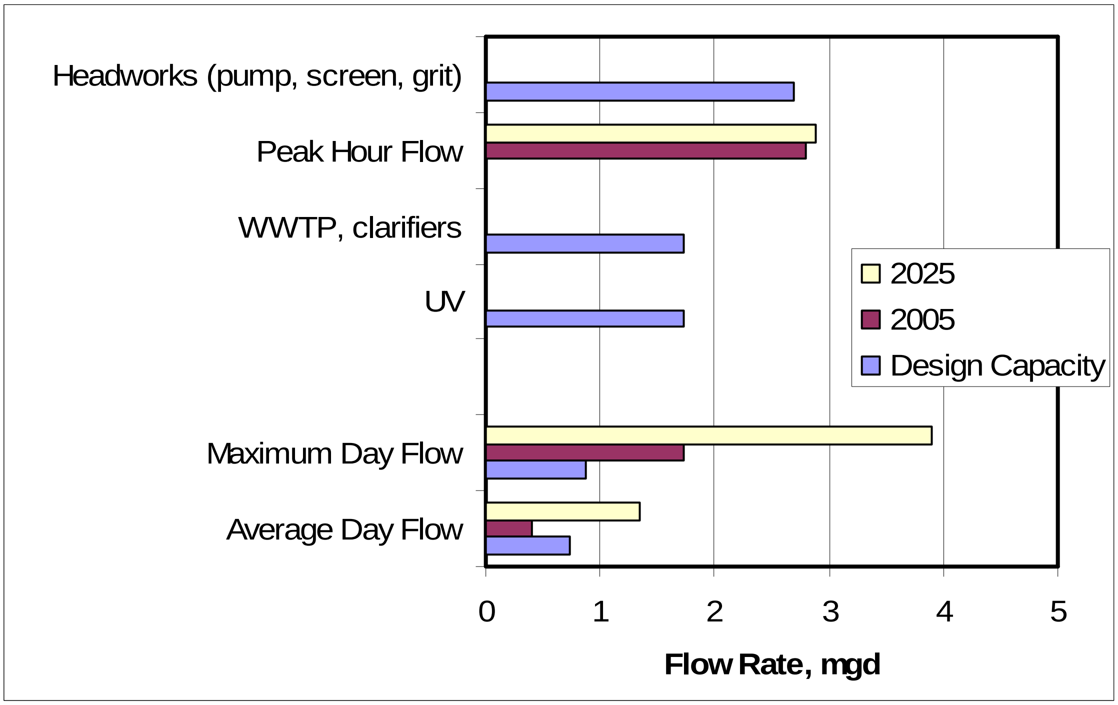
As the graph indicates, the wastewater treatment plant design maximum day flow has been exceeded based on 2001-2005 data. Peak hour flows from the 2001-2005 data are excess of the clarifier and UV system designs.

Though the 2005 average day flow is within the design capacity of the wastewater treatment plant, the maximum day and peak hour flows are contributing to reduced overall performance of the treatment system during times of excessive infiltration and/or wet weather flow events.

Upgrades to the facility are likely required to process maximum day and peak hour flows while meeting current and future regulatory requirements. Substantial I/I reduction or elimination could significantly extend the capability of the existing treatment plant to process future wastewater flow.

If I/I reduction is not significant nor elimination possible, then both treatment process and capacity upgrades will be required to meet projected 2025 wastewater and I/I flow to the wastewater treatment plant as well as current and future regulatory requirements.

Figure 4: Wastewater Treatment Plant Capacity



Section 5: Regulatory Outlook for Wastewater Treatment Systems

NPDES Renewal Issues

The current NPDES permit limits maximum day flow to 0.88 mgd. The WWTP sees periodic exceedance of this parameter as associated with wet weather flow events. The maximum daily flow limit is imposed by ADEC through its 401 Certification of the NPDES permit. ADEC reports EPA does not have authority to set or enforce flow limits in the NPDES permit, but still, has recently included it in many renewed permits. Although ADEC indicates exceeding the daily flow limit may not result in a permit violation, it would be in the City's best interest to peruse I/I mitigation strategies or flow attenuation projects with the goal of remaining below the ADEC set flow limit stated in the NPDES permit.

According to 2001-2005 data, the WWTP has consistently achieved greater than 85% monthly average removal of BOD_5 and TSS. However, if BOD_5 and TSS data collection is coincident with wet weather flow events, there is a chance that the monthly average removal rates may be impacted to the extent that 85% removal is not achieved. The lowest record single day percent removal for both BOD_5 and TSS was approximately 76%. The BOD_5 data was recorded on 2/11/2004 with a daily flow of 0.899 mgd and the TSS data recorded on 1/19/2001 with a daily flow 0.85 mgd. Only 2% of the recorded data indicated removal rates of 85% and less for BOD_5 and TSS.

The City should take note of potential draft permit language requiring sampling of non-routine discharges. Per EPA proposed policy on NPDES permit requirements during wet weather conditions (40 CFR Parts 122 and 133) the percent removal standard may be modified for facilities with wet weather flow impacts to separate sewer systems causing very dilute influent provided that 1) effluent concentration limits are consistently met, 2) the facility would have been required to meet significantly more stringent limitations than would otherwise be required for concentration-based standards, and 3) dilute influent is not caused by excessive I/I as defined by 40 CFR 35.2005(b)(16). Excessive I/I is defined as 1) quantity of I/I which can be economically eliminated, and 2) flows to the treatment plant in excess of 275 gallons per capita per day. It is not currently known how EPA intends to implement and apply the proposed policy.

1. Whole Effluent Toxicity Testing may become a proposed requirement in the City's draft NPDES permit.
2. Mixing Zone Monitoring may become a requirement by the State of Alaska in their certification of the EPA's draft NPDES permit
3. The EPA may require monitoring of parameters of interest known to be associated with landfill leachate discharge to the sanitary sewer.

NFPA 820, Standard for Fire Protection in Wastewater Treatment and Collection Facilities

The National Fire Protection Association, Inc. (NFPA) 820, Standard for Fire Protection in Wastewater Treatment and Collection Facilities, establishes minimum requirements for protection against fire and explosion hazards in wastewater treatment plants and associated collection systems. The requirements of the NFPA 820 are applicable to new facilities. The requirements of the NFPA 820 are to be reflected in additions or modifications to existing facilities, but are generally not applicable to facilities that existed prior to the effective date of the standard. The standard further explains: "In existing facilities, it is not always practical to apply the provisions of this standard strictly. Physical limitations could necessitate disproportionate effort or expense with little increase in fire protection. In such cases, the authority having jurisdiction should be satisfied that reasonable fire protection is ensured. In existing facilities, it is the intent that any condition that represents a serious threat to fire protection should be mitigated by application of appropriate safeguards. It is not the intent to require modification for conditions that do not represent a significant threat to fire protection, even though such conditions are not literally in conformance with the fire protection requirements."

Any upgrades to the Homer WWTP will require a thorough review of the requirements of NFPA 820 and the associated fire and explosion hazards. Generally, in areas requiring classification, ventilation of 12-air changes per hour is applied and equipment suitable for use in the classified space is installed. Design and construction, as well as capital and operational costs to meet NFPA 820 standards may represent significant increases in cost for all categories.

Certified Operators

ADEC classified Homer's wastewater treatment system as a Level 3. The Homer wastewater treatment system is operated by staff with certification levels as listed in **Table 8**.

Table 8: Certified Wastewater Operators

Operator Name	Certification/Number	Certification Expiration
Paul Barcus	Wastewater Collection 2 / 2170	12/31/06
David Bolt	Wastewater Treatment 3 / 911	12/31/07
Kenneth Frazier	Wastewater Collection 2 / 10222 Wastewater Treatment OIT/ 5001	12/31/06 12/31/05
Jim Hobbs	Wastewater Treatment 3 / 493	12/31/07
Gerald Lawver	Wastewater Treatment 2 / 1671	12/31/06
Steven Martin	Wastewater Treatment 3 / 7567	12/31/06

Section 6: Plant Upgrade Options

This section provides two separate sets of options, the first focusing on short-term options to be applied over the next five-year period, and the second on treatment plant upgrade options intended for a 20-year planning horizon.

Short-term Options for Next 5-years

The following paragraphs present options for enhancing the performance of the existing WWTP, as may be suitable as short-term options for implementation within the next five years, though some options may be suitable for continued use with future upgrades.

For some of these options, on-site assessments, bench scale testing, and/or pilot testing could be used to further assess the practicality, benefit, and cost of implementation.

Wet Weather Flow Control and Treatment Options

I/I Control and Elimination

This topic is addressed in the *I/I Reduction* section of this master plan. Not only is landfill leachate a potential large contributor to wet weather flow, it may also contain contaminants and impart loading that could adversely affect treatment processes at the WWTP. In particular, UV disinfection may be fouled with metals in the leachate, and the metals may also impact loading limits imposed on the receiving water body and sediments.

Other communities in Alaska have been reviewing the removal of leachate contribution to sewer systems, or are contemplating installation of leachate treatment systems. The EPA, in reviewing NPDES permits, has recently been taking note of leachate contributions to sanitary sewer systems and have required testing of constituents that may be found in leachate, not only at the source, but in the receiving waters.

Off-Line Equalization

Off-line flow equalization to dampen wet weather-induced wastewater flow is one option for handling maximum day and peak hour flows. In off-line equalization, flow above a predetermined rate is diverted from the sewage collection system to an equalization basin for short-term storage. The stored wastewater is then returned to system over time at a rate determined by the capacity of the wastewater treatment system. Ideally, equalization basins are sized using detailed and accurate flow rate data collected during peak flow events. Since these data are not available for Homer, an estimate of required equalization volume of approximately 3.0 million gallons (mgal) was used for purposes of this plan, and represents the difference between the calculated 2025 severe storm peak day of 3.9 mgd and the maximum day design capacity of the existing WWTP of 0.88 mgd. This is a conservative estimate based on the sewer system modeling. More detailed flow data and future treatment capacity upgrades may justify the use of a smaller basin.

A 3.0 mgal lined earthen equalization basin 12-feet deep with 3:1 (run:rise) side slopes would require enough land to accommodate an approximately 220 by 220-foot (top dimension) basin. The basin, associated access roads, and appurtenant structures would require approximately 2.0 acres total. A basin mixing and aeration system (e.g., aspirating jet mixer/aerator) would be required to prevent solids from settling in the basin and to reduce the potential for odor generation. A water cannon system to clean the equalization basin after storm events would also be required.

The equalization basin may be located east of the existing treatment and public works facilities along the main gravity sewer pipe feeding the treatment plant lift station. The City would have to acquire this land or land at any other suitable location proximate to the sewage collection/conveyance system.

The facility would be configured with a concrete splitter box and adjustable weirs installed in the gravity sewer line that would direct to the equalization basin wastewater flow exceeding the capacity designated for the treatment plant. Once the peak flow event subsided, a duplex, variable speed pump system would return the stored wastewater to the gravity conveyance system for subsequent treatment at the plant.

By locating the equalization basin at a lower hydraulic elevation than the gravity sewer, only one pumping system would be required to deliver stored wastewater to the plant. If site conditions do not allow for gravity flow, then an additional pumping system would be required to convey wastewater flow from the splitter box to the equalization basin.

This option is identified as “A” and schematically illustrated on a WWTP site plan, **Sheet No. 33 B**.

Use of an equalization basin would potentially eliminate the future need to upgrade existing headworks structures. Installation of an equalization basin would require little or no modification to the existing wastewater treatment facility to control maximum day and peak hour flows.

The primary disadvantage of the equalization basin option is additional land will have to be purchased. In addition, the equalization basin will have the potential to generate odors, though lessened with proper maintenance and cleaning after each event. Operation and maintenance requirements for the splitter box and equalization basin, and maintaining the pump station and aeration/mixing equipment will increase the time required to operate the treatment facility.

Wet Weather Flow Treatment

The wet weather flow treatment options presented here would require upgrades to the wastewater treatment plant lift station, headworks structures and appurtenances, preliminary treatment systems (grit and screening), and possibly the effluent outfall, to process the wet weather induced flows. The capacity required of each would be approximately 3 mgd.

Raw sewage would be lifted to expanded capacity screen and grit removal equipment with discharge to a hydraulic diversion structure. Preliminary treated flows exceeding the capacity designated for the biological treatment system would be diverted through a wet weather flow treatment. Treated flows may also require disinfection prior to being recombined with treated effluent from the biological treatment system.

Two wet weather flow treatment systems which may have applicability to this project are clarifiers and microscreens. The capacity required of each would be approximately 2.2 mgd assuming the treatment system would receive a baseline flow of approximately 0.88 mgd. Depending on upgrades phasing, the sizing of the system may be adjusted as applicable.

Clarifier

The clarifier system uses chemical coagulation, weighted flocculation and lamella plate settling that was originally developed for drinking water applications, but has recently been used for wastewater treatment for BOD_5 and TSS removal beyond that provided by preliminary treatment for dilute wet weather induced flows.

The clarifier would be sized to achieve BOD_5 and TSS removal efficiencies of up to 60%, though pilot studies may be required for confirmation.

Preliminary treated wastewater first flows into an injection chamber where polymer and microsand are added. In the maturation chamber, flocs are allowed to form around microsand particles. These dense flocs are then removed in a lamella plate clarifiers and the microsand recovered using hydrocyclones. Sludge from the clarifier is returned to the existing plant’s biological treatment process or digester.

The clarifier system can potentially remove more BOD_5 and TSS than the microscreens thus improving the systems ability to meet NPDES treated effluent limits. However, this option is more complex than the microscreens and will require more operator attention to adjust the polymer and coagulant feed pumps to ensure removal efficiencies. Although the clarifier option has a small footprint compared to conventional clarification systems and the equalization basin option presented, the space occupied by this unit will be greater than the microscreen option. Treatment of chemically coagulated sludge from the clarifier (or microscreens) may also impact the existing sludge handling system due to added volume and possible remnant presence of coagulant.

This option is identified as “A1” and schematically illustrated on a WWTP site plan, **Sheet No. 33 A**.

Microscreen

Microscreens have been adapted for use in wastewater treatment to remove additional BOD_5 and TSS beyond that provided by preliminary treatment for what is normally very dilute wet weather flows.

The units consist of microscreen mesh of 20 to 60 micrometers (µm) mounted on a center feed rotary drum assembly. Chemical coagulant may be applied to aid filtration. Screenings collected by the microscreens are returned the existing plant’s biological treatment process or digester.

Since preliminary treatment and microscreen removal efficiencies can vary depending on actual wastewater characteristics, pilot studies are required to determine design parameters and need for coagulant. Depending on the results of the pilot studies, a range of performance values may be between 30% to 60% total BOD_5 and TSS removal. This data would be used to determine the ratio of biological treatment capacity at secondary quality or better, and wet weather flow treatment capacity at achievable levels so when combined, effluent meets NPDES limits.

This option is identified as “A2” and schematically illustrated on a WWTP site plan, **Sheet No. 33 A**.

Wastewater Contributions and Quality Determinations

The *Performance of Wastewater Treatment Plant* section of this report showed a marked increase in the BOD_5 received at the wastewater treatment plant over the last four years. Continued increase in BOD_5 loading could exceed the capacity of the existing plant. In order to best respond to this concern and determine impacts to future upgrades and expansions, the City should pursue options to define the contributions of high strength waste to the sewage collection system.

Specifically, field studies may be conducted to determine wastewater quality, flow rate, and quantity including but not limited to septic tank effluent, landfill leachate; recreational vehicle discharges; and restaurant, industry, and commercial dischargers. Also, a detailed study is required to predict future septic tank effluent contribution and septage/holding tank discharges quantity, quality, pre-treatment and/or treatment needs, and associated capital and operational costs.

Control and treatment of high strength wastewaters, including any future septage receiving may be required to ensure maintenance of effluent and sludge quality produced by the WWTP. Control methods imposed on customers may include but is not limited to enforcement of grease trap ordinances, and pretreatment prior to discharge to City sewers. Septage treatment at the WWTP may include solids removal, aeration and/or digestion, and dewatering. Aeration may decrease BOD_5 prior to normal processing at WWTP, or septage may be incorporated into the WWTP’s sludge digestion and processing systems.

Options for Year 2025

Options more suitable for long-term upgrades which may be implemented to meet estimated requirements for the year 2025 are provided below. Depending on the needs of the City, these options may also be suitable for short-term upgrade consideration.

Waste Solids Management Options

Regulations require disposed sewage solids (screenings, grit, and sludge) meet pathogen reduction requirements and/or meet vector reduction requirements. Land applied sewage sludge must also meet certain pollution concentration limits. The City currently meets the requirements for land application through their existing aerobic digestion processes (digesters and lagoon), lagoon storage, bed dewatering, and a freeze-thaw cycle.The following summarizes sludge dewatering and disposal alternatives, and recommendations for a solids management program.

Sludge Dewatering

Mechanical dewatering prior to placing sludge in the existing beds for a freeze-thaw cycle would allow a greater volume of sludge to be placed each fall. Options for dewatering sludge removed from the lagoon for placement in the freeze-thaw dewatering beds include equipment such as centrifuges, gravity belt thickener, and belt filter, plate and frame, and screw presses.

Each of these mechanical dewatering options requires polymer addition to the sludge to facilitate release of water from the sludge in processing. The gravity belt thickener, screw press, and centrifuge may each achieve dewatered sludge solids in the range of 4 to 12 % percent dry solids (ds). Belt filter, and plate and frame presses may achieve dewatered sludge solids in the range of 14 to 20% ds, and up 40% ds. Vacuum and heat assisted systems may achieve up to 90% ds. Performance of each individual system is dependant on the equipment design and operation, feed quality and solids content, and conditioning methods. The sludge dewatering option is identified as “B” and schematically illustrated on a WWTP site plan, **Sheet No. 33 A**.

As the treatment facility expands in the future, sludge digestion unit processes may be added and existing modified to increase the efficiency of digestion thus eliminating the current requirement of one freeze-thaw cycle to achieve stabilization necessary for land application. Stabilized sludge can be either stored or placed on the dewatering beds. Future expansions of this system will require careful review to determine potential impacts from septage receiving and processing operations.

Options for Sewage Solids Disposal

Options for ultimate disposal of sewage solids include incineration, landfill, and beneficial land application. These are discussed briefly below.

Incineration

Incineration of sewage solids is comparatively more expensive than the other alternatives, even with the availability of natural gas. Natural gas is not routed to Homer, but extension of service from Ninilchik is planned for the future. Incineration is energy intensive, as it requires the conversion of liquid in sludge to steam, sending it into the atmosphere along with the other products of combustion. Ash may be disposed in a solid waste landfill. The incineration option is typically viable when no other disposal alternatives are readily available within the vicinity of the treatment plant.

Landfill

Disposal of sewage solids in a landfill is an option only at sites permitted for this activity. The Kenai Peninsula Borough’s Class 2 landfill (ADEC 0023-BA004 expiration August 6, 2008) is currently listed for receiving only construction debris and municipal waste. And, there are no known permitted sludge-only landfills (monofills) operating within the Borough at this time.

In order to co-dispose sewage solids with municipal solid waste in a permitted landfill (Municipal Solid Waste Landfill, MSWLF), the following requirements must be met:

- The sewage solids must be free of hazardous wastes and polychlorinated biphenyls (PCB) in accordance with 40 CFR 261, 18 AAC 62 and 40 CFR 761.
- The sewage solids must not contain "free-liquids" as defined by EPA Method 9095 (Paint Filter Test) as described in Test Methods for Evaluating Solid Waste, Physical/Chemical Methods, Third Edition, November 1986 (SW-846).
- The sewage solids meet the vector reduction requirement in accordance with 40 CFR 503.33(b)(11); OR must be treated and stabilized to meet Class A or Class B pathogen reduction requirements in accordance with 40 CFR 503.32, AND vector attraction reduction requirements of 40 CFR 503.33 (b)(1)-(10), as adopted by reference in 18 AAC 60.505.

Land Application

Biosolids (sludge) may be disposed by beneficial application to agricultural or municipal lands. Land application of sludge may increase the organic and nutrient content of the soil and enhance its water retention character. To be eligible for land application, the regulations cited above also apply as described further below.

- The soils to which sludge shall be applied must have concentrations of metals below, and will remain below, limits established in Table 1 of 40 CFR 503.13.
- The sludge to be applied to agricultural land or public contact sites must have concentrations of metals lower than the limits established in Table 1 of 40 CFR 503.13.
- The sludge applied to agricultural land or public contact sites must satisfy one of the following conditions:
  - The concentration of metals at the application sites shall not exceed the cumulative limits for metals established in Table 2 of 40 CFR 503.13, OR
  - The maximum concentrations of contaminants in the applied sludge shall not exceed the limits listed in Table 3 of 40 CFR 503.13.
  - The sludge cannot be applied to the land designated as endangered species critical habitat, or adversely affect endangered species.
  - The sludge cannot be applied to the land during periods when the land is frozen, snow covered or flooded.
  - The sludge cannot be applied to land within 10 meters of navigable waterways.
  - The sludge cannot be applied to land in excess of the rate at which the nutrients in the sludge are used by the vegetative cover for the area.
  - The sludge must be treated and stabilized to meet Class A or Class B pathogen reduction requirements in accordance with 40 CFR 503.32, AND vector attraction reduction requirements of 40 CFR 503.33 (b)(1)-(10), as adopted by reference in 18 AAC 60.505.

The options recommended for Homer include either disposal at a MSWLF and beneficial land application.

Wastewater Treatment Capacity Upgrades

The average day flow to the wastewater treatment plant is predicted to double to approximately 1.4 mgd by the year 2025. As such, the existing treatment facility would require an upgrade of approximately double its current annual average day flow capacity of 0.73 mgd.

Options available to the City for the next 20-year planning horizon include expanding the existing wastewater treatment plant and constructing a new treatment plant. Given the relatively good condition and age, as well as the successful implementation and operation of the existing facilities, construction of a new treatment facility is not being considered in this master plan. Depending on implementation of this plan, the City may wish to reconsider this option with future master plan updates.

The existing treatment system may be replicated to the west and adjacent to the existing treatment building. Some of the infrastructure, such as the polymer feed system, may be housed at the same location with potentially minimal upgrades in capacity required to serve the expanded system. This option is identified as “C” and schematically illustrated on a WWTP site plan, **Sheet No. 33 A**.

The upgrade of the wastewater treatment plant will require careful consideration of maximum day and peak hour flows anticipated in the future. Depending on wet weather flow control and treatment options implemented, as well as consistent I/I control achieved, the design requirements to handle maximum day and peak hour flows may be substantially reduced and anticipated to occur on a less frequent basis. Control of the maximum day and peak hour flows will allow for many of the unit processes in the treatment system to be designed more economically with reduced foot print and power requirements.

The following lists the potential upgrades required for implementing a capacity upgrade to the existing treatment facilities:

- Raw sewage lift station capacity increase with related piping and metering upgrades;
- Screening and grit removal capacity increase with related piping and diversion structure upgrades;
- Construction of similar deep shaft reactor(s), clarification, UV disinfection and aerobic sludge digestion treatment trains with potential phasing of infrastructure upgrades;
- Additional aerobic sludge digestion and thickening capacity to maximize utilization of sludge storage lagoon and covered existing sludge dewatering freeze-thaw beds, while incorporating requirements imposed if septage is to be received in the future;
- Mixing zone, and receiving water and sediment modeling; and
- Effluent outfall capacity upgrade.

Recommendations for Upgrades

The following sections describe recommendations for short and long-term upgrades. Should the City so choose, on-site assessments, bench scale testing, and/or pilot testing are recommended and would confirm practicality, benefit, and cost of implementation if these upgrade options.

Wet Weather Flow Control

Though many options exist to control I/I, elimination of sources of I/I often yields the most benefit to sewer systems and treatment plants. Options for I/I control and elimination are presented in the *I/I Reduction* section of this master plan.

Since I/I elimination and control are often difficult to maintain consistently, options to either equalize or treat wet weather flow events are often contemplated in conjunction with I/I control and elimination. Though many options exist to equalize or treat wet weather flow events, installation of off-line equalization may yield the most benefit to the WWTP for this 20-year planning horizon.

Wastewater Contributions and Quality Determinations

Complete studies to define potential high strength wastes and impacts to the WWTP. The City may complete much of this work in house. The City may wish to solicit for consultation on a septage receiving feasibility study and pretreatment programs.

Sludge Management Program

The City should secure agreements with the Borough for landfill disposal and local agricultural interests for beneficial land application. In addition, should agreements with the Borough or agricultural interests not be negotiable, the City may wish to acquire lands for potential sludge landfill operations and/or land application of stabilized sludge.

Ultimate disposal of waste solids may be best accomplished by supplementing the existing sludge stabilization and gravity dewatering process with mechanical dewatering. Lagoon sludge may be chemically conditioned and mechanically dewatered to approximately 6 to 12 percent solids prior to being directed to the covered freeze-thaw dewatering beds. Future expansions may integrate increased digestion capability and/or elevation of the pH of waste sludge. Additional digestion will allow for disposal of stabilized sludge not requiring a freeze-thaw cycle. Increasing pH levels may be used as a method of stabilization and, as applicable, may be practiced for benefit to soils with naturally low pH levels.

Treatment Plant Upgrades

The most suitable long-term upgrade for the WWTP includes the expansion of the existing activated sludge deep shaft reactor. Depending on the needs of the City, the upgrades may be phased and planned to utilize full capacity of existing infrastructure such as the headworks and polymer feed system.

Comparative Costs for Alternative System Upgrades

Pre-design, rough order of magnitude opinions of project costs for the upgrades reviewed in this report are presented in **Attachment B-1** of this Appendix. These costs include a significant contingency factor, and represent a rough order of magnitude accuracy range of +30%/-15% of the actual cost of the work. All cost data presented here are in year 2006 dollars.

Estimates do not include costs for land acquisition. The actual cost of construction depends on the final project scope, site location and construction plans; actual labor and material costs; material delivery; shipping time; actual site and weather conditions; productivity; market conditions; and other variable factors. As such, the actual construction cost will vary from the rough order of magnitude estimates provided herein and we recommended the City update cost estimates at one or more stages of preliminary design and design phases of selected upgrades.

References

1. USEPA (February 1982) Technology Assessment of the Deep Shaft Biological Reactor, EPA-600/2-82-002.
2. Great Lakes – Upper Mississippi River Board of the State and Provincial Public Health and Environmental Manager (1997) Recommended Standards for Wastewater Facilities. (Commonly referred to as the Ten States Standards).
3. Metcalf and Eddy, Inc. (1991) Wastewater Engineering: Treatment, Disposal, Reuse, McGraw Hill, NY.
4. USEPA (September 1992) Manual: Wastewater Treatment/Disposal for Small Communities, EPA/625/R-92/005.
5. Water Environment Federation (1988) Manual of Practice FD-13: Aeration: A Wastewater Treatment Process, WEF/ASCE.
6. Water Environment Federation (1991) Manual of Practice 8: Design of Municipal Wastewater Treatment Plants, Volume I and II, WEF/ASCE.
7. Water Environment Federation (1994) Manual of Practice FD-6: Existing Sewer Evaluation & Rehabilitation, WEF/ASCE.
8. Water Environment Research Foundation, J. Darby, et al. (1995) Comparison of UV Irradiation to Chlorination: Guidance for Achieving Optimal UV Performance, Alexandria, VA.

ATTACHMENT B-1

Wastewater Treatment Plant Cost Estimates

HOMER WWTP - UPGRADE OPTIONS
ROUGH ORDER OF MAGNITUDE COST OPINION
(ACCURACY RANGE: +30%/-15%)

SUMMARY

OPTION	DESCRIPTION	TOTAL COST
A	Wet Weather Flow - Off-line Storage	$3,196,985
A1	Wet Weather Flow Treatment - Clarifier	$7,158,538
A2	Wet Weather Flow Treatment - Microscreen	$7,236,108
B	Sludge Dewatering	$1,123,022
C	Expansion of Existing Deep Shaft Activated Sludge Plant	$15,000,432

APPENDIX C

Hydrology Report
for the City of Homer

Final

Hydrology Report for the City of Homer

July 2006

TABLE OF CONTENTS

TABLES

FIGURES

ACRONYMS AND ABBREVIATIONS

°F	degrees Fahrenheit
ADF&G	Alaska Department of Fish and Game
cfs	cubic feet per second
City	City of Homer
gpd	gallons per day
Report	Hydrology Report for the City of Homer
USGS	U.S. Geological Survey

CONVERSION FACTORS AND DATUM

Multiply	By	To obtain
inch (in)	2.54	centimeter (cm)
foot (ft)	0.3048	meter (m)
foot per mile (fpm)	0.1894	meter per kilometer
mile (mi)	1.609	Kilometer (km)
square mile (mi^2)	2.590	square kilometer
cubic foot per second (cfs or ft^3/s)	0.02832	cubic meter per second
cubic foot	7.480519	U S gallon liquid

Temperature in degrees Fahrenheit (ºF) may be converted to degrees Celsius (ºC) as follows:

$$°C = (°F-32)/1.8$$

DATUM

- **Vertical coordinate information** was referenced to the National Geodetic Datum of 1929 (NGVD 29) unless otherwise stated.
- **Horizontal coordinate information** was referenced to North American Datum of 1927 (NAD 27).

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1.0 INTRODUCTION

The purpose of this Hydrology Report (Report) is to provide the City of Homer (City) with a preliminary assessment of the ability of the existing Bridge Creek Reservoir, as well as potential future surface water supplies, to meet future potable water demands. Because of the limited amount of hydrologic data in the form of mean daily flows for the immediate area, it was necessary to use the U.S. Geological Survey (USGS) crest gauging data to supplement the limited information regarding mean daily flows for the area.

2.0 METHODOLOGY

There are no long-term streamflow records available. Although relatively good meteorological data is available in the vicinity of the study area, there is little or no actual data available specific to the streams that are being considered for this study. A series of statistical equations developed by the USGS, and national standards developed by the USGS Water Resources Branch, in cooperation with the Alaska Department of Transportation and Public Facilities (ADOT&PF) were used. These publications are:

- *Guidelines for Determining Flood Flow Frequency*, Bulletin #17B, U.S. Geological Survey, Interagency Advisory Committee on Water Data.
- *Flood Characteristics of Alaska Streams*, U.S. Geological Survey, Water-Resources Investigations Report 78-129.
- *Magnitude and Frequency of Floods in Alaska and Conterminous Basins of Canada*, U.S. Geological Survey Water-Resources Investigations Report 93-4179.
- *Estimated Annual High-Flow Statistics and Monthly and Seasonal Low-Flow Statistics for Ungaged Sites on Streams in Alaska and Conterminous Basins in Canada*, U.S. Geological Survey, Water-Resources Investigations Report 03-4114.
- *Estimating the Magnitude and Frequency of Peak Streamflows for Ungaged Sites on Streams in Alaska and Conterminous Basins in Canada*, U.S. Geological Survey, Water-Resources Investigations Report 03-4188.
- *Water Resources of the Cook Inlet Basin*, Alaska U.S. Geological Survey, Geoffrey W. Freethey, and David R. Scully.

2.1 PRECIPITATION AND TEMPERATURE DATA

For the purposes of this Report, the mean January temperature (in degrees Fahrenheit [F°] – Figure 1), and mean annual precipitation (in inches – Figure 2) information were taken directly from the Magnitude and Frequency of Floods in Alaska and Conterminous Basins of Canada, USGS, WRI 93-4179, by Stanley H. Jones, and Charles B. Fahl, 1994. These independent variables from the 1993 publications are used to develop the current regression equations.

Figure 1. Mean January Minimum Temperature

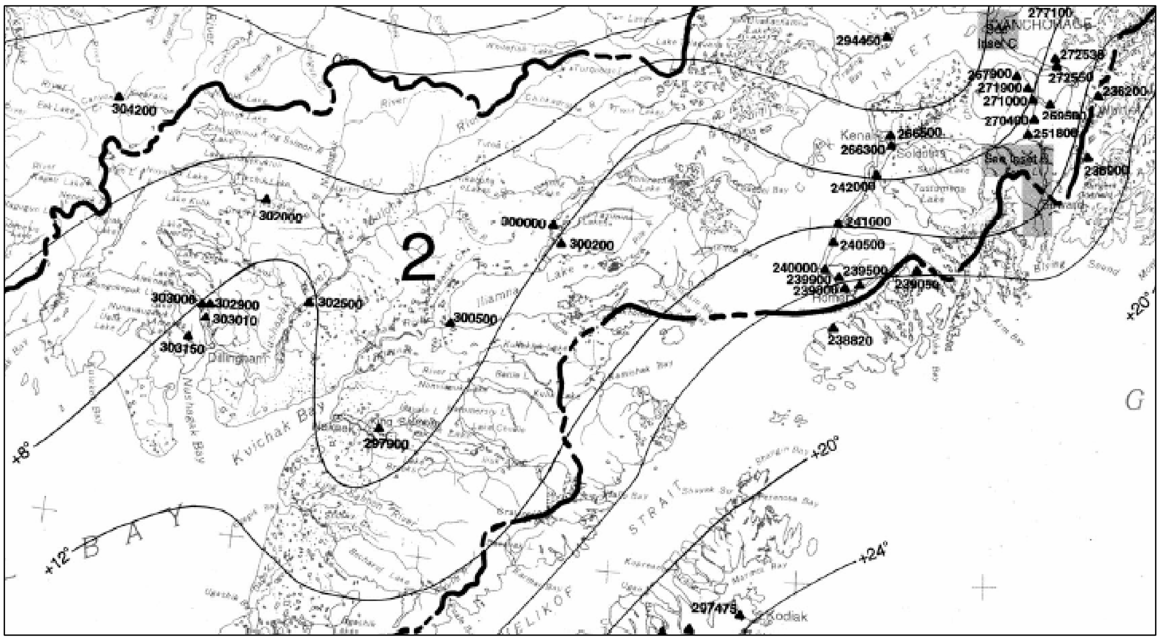
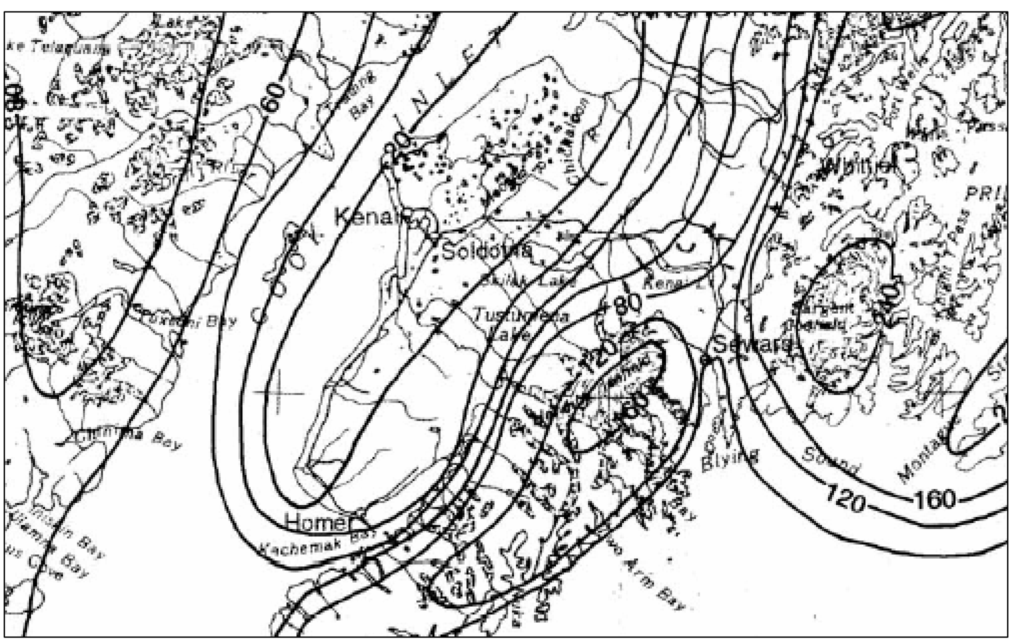


Figure 2. Mean Annual Precipitation



Using this information, the following was determined:

- Mean January temperature of +15°F, and
- Mean annual precipitation of 25 inches.

2.2 STREAMFLOW DATA

Table 1 provides a listing of all of the surface-water data from the USGS, including surface-water, water quality, and miscellaneous records. The table provides the basin characteristics as reported by the USGS Water Resources Branch, with no modifications, for each gaging site.

Table 1 USGS Surface Water Data

No	Station Name	Gage #	Latitude	Longitude	Da (mi^2)	El (ft)	St (mi^2)	Gl (mi^2)	Fr (mi^2)	Pr (in)	Te (ºF)
1	Anchor River @ Anchor Point	15240000	59°46'21"	151°50'05"	224	970	0	0	118.72	25	14
2	Anchor River above Beaver Creek nr Homer	15239810	59°45'10"	151°30'31"	63.2	802	0	0		25	14
3	Anchor River above Twitter Creek nr Homer	15239840	59°43'06"	151°38'31.3"	105	913	0	0		25	14
4	Anchor River nr Anchor Point	15239900	59°44'50"	151°45'11"	137	1120	0	0	82.2	25	14
5	Anchor River nr Homer	15239805	59°48'32"	151°23'51"	28.8	1434	0	0		25	14
6	Anchor River trib. at mouth near Homer	15239807	59°48'35"	151°23'56"	20.1	1124	0	0		25	14
12	Beaver Creek at mouth nr Homer	15239822	59°45'09"	151°30'29"	19.8	1005	0	0		25	15
13	Beaver Creek near Bald Mountain nr Homer	15239818	59°44'59"	151°19'44"	5.41	1197	0	0		25	15

Table 1 USGS Surface Water Data (continued)

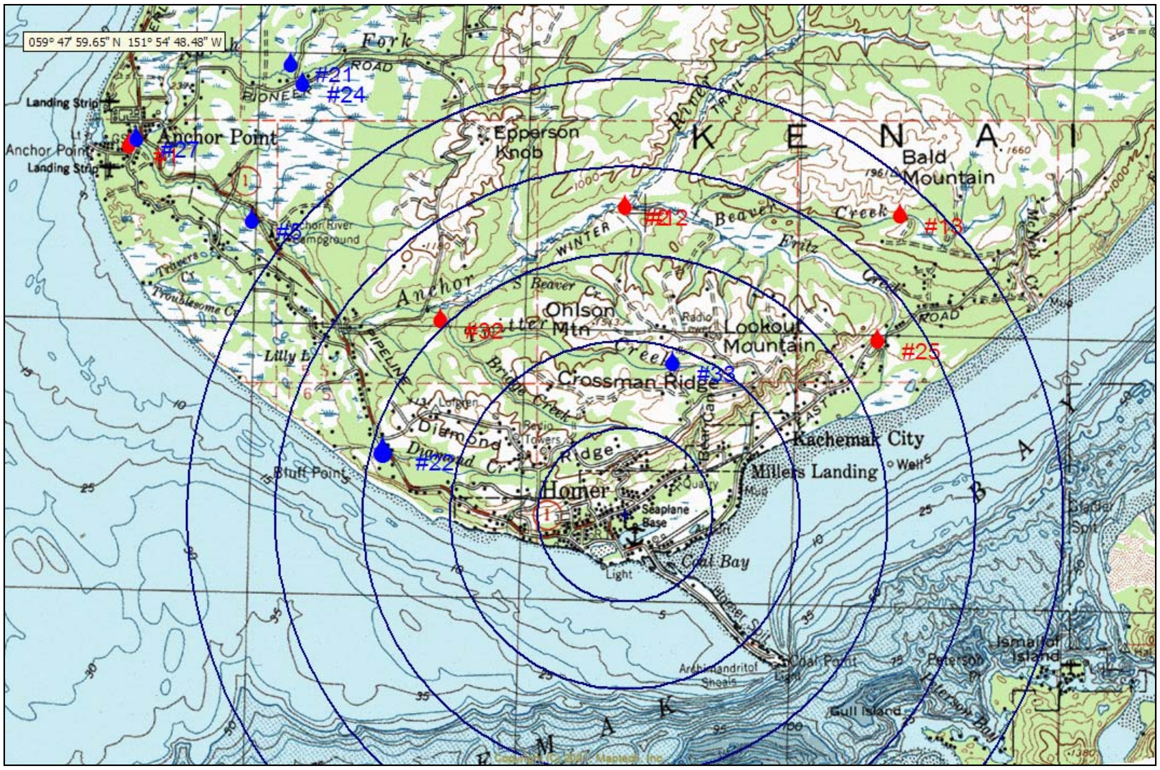
No	Station Name	Gage #	Latitude	Longitude	Da (mi^2)	El (ft)	St (mi^2)	Gl (mi^2)	Fr (mi^2)	Pr (in)	Te (ºF)
20	Chakok River nr Anchor Point	15239980	59°48'03"	151°43'51"	38.7						
21	Diamond Creek nr Homer	15239800	59°40'10"	151°40'00"	5.35	890	0		1.9795	25	15
22	Falls Creek nr Homer	15239300	59°47'23"	151°07'14"	2.84						
23	Fork Anchor River (North Fork) above Chakok River nr Anchor Point	15239970	59°47'41"	151°43'46"	18.4	417					
24	Fritz Creek nr Homer	15239500	59°42'30"	151°20'35"	10.4	880	0		7.072	25	15
25	Ninilchik River @ Ninilchik	15241600	60°02'54.31"	151°39'53.7"	135	670	1.35	128.25		20	11
26	Nuka River (Upper) nr Park Boundary nr Homer	15238648	59°41'04"	150°42'12"	3						
27	Nuka River nr Tidewater nr Homer	15238653	59°39'59"	150°40'40"	38						
29	Tutka Lagoon Creek nr Homer	15238860	59°25'59"	151°34'36"	10.8						
30	Twitter Creek nr Homer	15239880	59°42'54"	151°36'46"	16.1	982				25	15
31	Twitter Creek nr Lookout Mountain nr Homer	15239845	59°42'00"	151°28'38"	1.63	1368				25	15

Notes:

°F = degrees Fahrenheit
Da = drainage area in square miles
El = mean basin elevation in feet MLLW
Fr = forested area in square miles
ft = feet
Gl = glacier area in square miles
in = inches
mi^2 = square miles
MLLW = mean lower low water
No = location of site on location map
Pr = mean annual precipitation in inches
St = storage area in square miles
Te = mean January minimum temperature °F

Figure 3 shows the location of the USGS stations in relation to Homer.

Figure 3. Area Map Showing Distance in 2-Mile Increments from Homer



3.0 RESULTS

This Report concurs with the general conclusions of the 1980 CH2M Hill *Homer Area Water Reservoir Study* and the 1983 Olympic Associates *Water Improvement Study*, and concentrated on the water sources listed in Table 2.

Table 2 Proposed New Water Sources

Name Creek	Location	Drainage Area (mi^2)
South Fork Beaver Creek	Just upstream of Beaver Creek Flats	9.80
Fritz Creek	Northeast of Homer	9.60
South Fork Beaver Creek	0.5 miles upstream of confluence of Anchor River	2.40
Twitter Creek	North of Bridge Creek Drainage	3.50
Bridge Creek	Current Reservoir	3.31

Note: The above drainages are approximately the same locations discussed in the 1980 (CH2M Hill) Homer Area Water Reservoir Study.

These five potential water sources are shown on Figure 4.

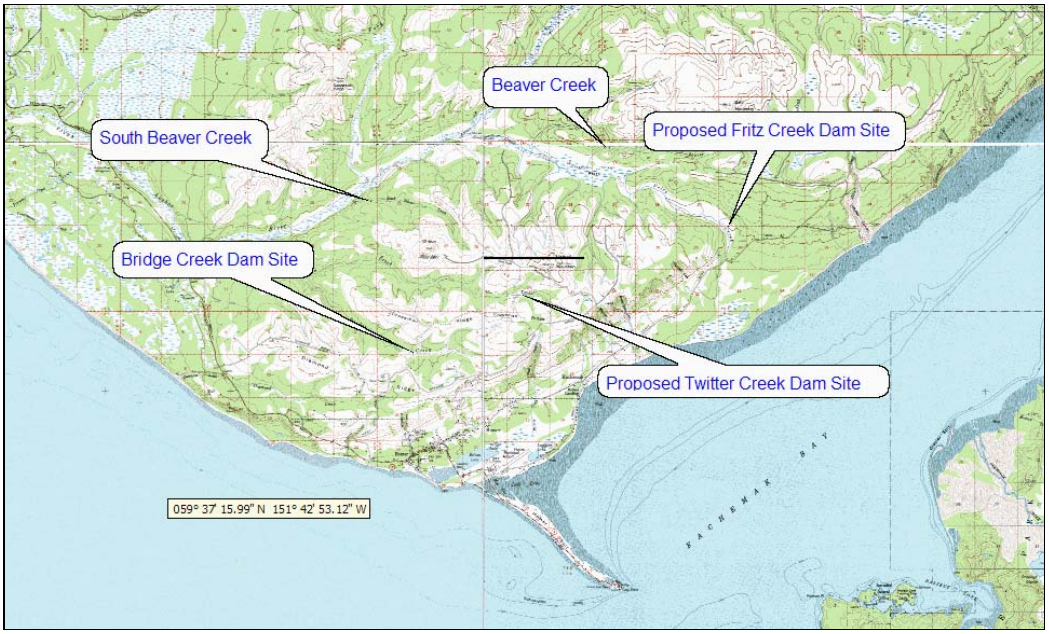


Figure 4. Existing Bridge Creek Reservoir and Potential Water Sources

Using the statistical regression equations described above for each of the potential water sources, the following information was computed:

- Estimated floods of magnitude and frequency of peak stream flows (in cubic feet per second [cfs]) – Q_2, Q_5, Q_{10}, Q_{25}, Q_{50}, Q_{100}, Q_{200}, and Q_{500}.
- Estimated monthly magnitude and frequency (mean daily discharges in cfs) – $Q_{1.02}$, $Q_{1.05}$, $Q_{1.11}$, $Q_{1.17}$, $Q_{1.25}$, $Q_{1.42}$, $Q_{1.66}$, and Q_2 for the months of July, August, and September. These months represent the period of time when there is the highest demand for water from the community.
- Estimated annual (yearly) flow mean daily discharge (cfs) – $Q_{6.66}$, Q_{10}, $Q_{1.11}$, $Q_{12.50}$, $Q_{14.28}$, $Q_{16.66}$, Q_{20}, Q_{25}, $Q_{33.33}$, Q_{50}, and Q_{100}.

Using the equations published by USGS *Water-Resources Investigations Report* 03-3114, a mean daily annual flow computation was generated for each of the basins. Table 3 shows the mean low flows and annual flows for each basin. Table 4 shows projected water demands for the City of Homer.

Table 3 Projected Mean Daily Low Flow and Annual Flows

Name of Creek	Mean Daily Low Flow 7 Day Duration (cfs)	Mean Daily Annual Flow (cfs)	Mean Daily Low Flow 7 Day Duration (gpd)	Mean Daily Annual Flow (gpd)
Beaver Creek	4	20	2,585,000	12,926,000
Fritz Creek	3	20	1,939,000	12,926,000
South Fork Beaver Creek	1	5	1,293,000	3,232,000
Twitter Creek	1	7	646,000	4,524,000
Bridge Creek	1	6	646,000	3,878,000

Note: 1 cubic foot per second (cfs) = 646,316 gallons per day (gpd)

Table 4 Projected Water Demand

Year	2006 (gpd)	2011 (gpd)	2016 (gpd)	2021 (gpd)	2025 (gpd)
Summer Average Demand[1]	799,000	1,039,000	1,330,000	1,604,000	1,862,000
Yearly Average Demand	574,000	789,000	1,059,000	1,331,000	1,592,000

Note: [1]Summer is defined as the months of June, July, and August. Assumes future leakage and bleeding are proportionate to existing flowrates.

gpd = gallons per day

Table 5 shows the Low Flow 7-Day Duration, less the projected summer average demand in gallons per day (gpd) for the various water supply scenarios. These numbers do not account for loses due to evaporation, groundwater flow; or leakage through, around, and under the dam. For the Bridge Creek Reservoir, an unknown quantity of water is lost from the basin through seepage and evaporation, and is not available to the water system. Most of the water loss is probably through the groundwater column.

Table 5 Mean Daily Low Flow Less the Projected Summer Average Demand

Name of Creek	2006 (gpd)	2011 (gpd)	2016 (gpd)	2021 (gpd)	2025 (gpd)
Beaver Cr + Bridge Cr.	2,430,000	2,190,000	1,900,000	1,630,000	1,370,000
Fritz + Bridge Cr.	1,790,000	1,550,000	1,250,000	980,000	720,000
South Fork Beaver + Bridge Cr.	490,000	250,000	-40,000	-310,000	-570,000
Twitter Cr + Bridge Cr.	490,000	250,000	-40,000	-310,000	-570,000
Bridge Cr.	-150,000	-390,000	-680,000	-960,000	-1,220,000

Notes:

Cr = creek

gpd = gallons per day

It is evident from Table 5 that, where there is an extended low flow event during the summer months when demand is high and flows are low, the drawdown of the Bridge Creek Reservoir could exceed inflow rather dramatically. However, the annual recharge compared to consumption for Bridge Creek still remains positive, as shown in Table 6.

Table 6 Annual Mean Daily Flow Less the Projected Annual Average Demand

Name of Creek	2006 (gpd)	2011 (gpd)	2016 (gpd)	2021 (gpd)	2025 (gpd)
Beaver Cr + Bridge Cr.	16,230,000	16,020,000	15,750,000	15,470,000	15,210,000
Fritz Cr + Bridge Cr.	16,230,000	16,020,000	15,750,000	15,470,000	15,210,000
South Fork Beaver + Bridge Cr.	6,540,000	6,320,000	6,050,000	5,780,000	5,520,000
Twitter Cr + Bridge Cr.	7,830,000	7,610,000	7,340,000	7,070,000	6,810,000
Bridge Creek	3,300,000	3,090,000	2,820,000	2,550,000	2,290,000

Notes:

Cr = creek
gpd = gallons per day

Table 7 shows the annual mean daily low flow for a 28-day low flow event (drought). It is evident that the Bridge Creek reservoir could experience a significant drawdown over time for projected water demands from approximately 2016 to 2025.

Table 7 Annual Mean 28 Daily Low Flow Less the Projected Annual Average Demand

Name of Creek	2006 (gpd)	2011 (gpd)	2016 (gpd)	2021 (gpd)	2025 (gpd)
Beaver Cr + Bridge Cr.	4,530,000	4,290,000	4,000,000	3,730,000	3,470,000
Fritz Cr + Bridge Cr.	3,730,000	3,490,000	3,190,000	2,920,000	2,660,000
South Fork Beaver + Bridge Cr.	1,620,000	1,380,000	1,090,000	820,000	560,000
Twitter Cr + Bridge Cr.	2,110,000	1,870,000	1,580,000	1,300,000	1,050,000
Bridge Cr	490,000	250,000	-40,000	-310,000	-570,000

Notes:

Cr = creek
gpd = gallons per day

4.0 ENVIRONMENTAL CONSIDERATIONS

Twitter Creek is very important for silver salmon spawning. The minimum streamflow to support resident fish populations would need to be protected by the Alaska Department of Fish and Game (ADF&G) before water from Twitter Creek could be used. This requirement could impact the amount of water that could be made available from Twitter Creek.

Fritz Creek does not have a migratory fish population. However, it does contain some land-locked dolly varden. Silver and king smolt are released into the creek just above East End Road in May and June. Adult fish return in August and September, but they are blocked from returning upstream by a 6-foot waterfall about 100 yards from Kachemak Bay. ADF&G estimates a required flow of 3.5 to 4.0 cfs in Fritz Creek during the smolt release (May and June) and fish return (August and September) periods.

The South Fork of Beaver Creek has a strong fish population and the ADF&G would likely resist the development of a reservoir on that stream.

5.0 SUMMARY OF RECOMMENDATIONS

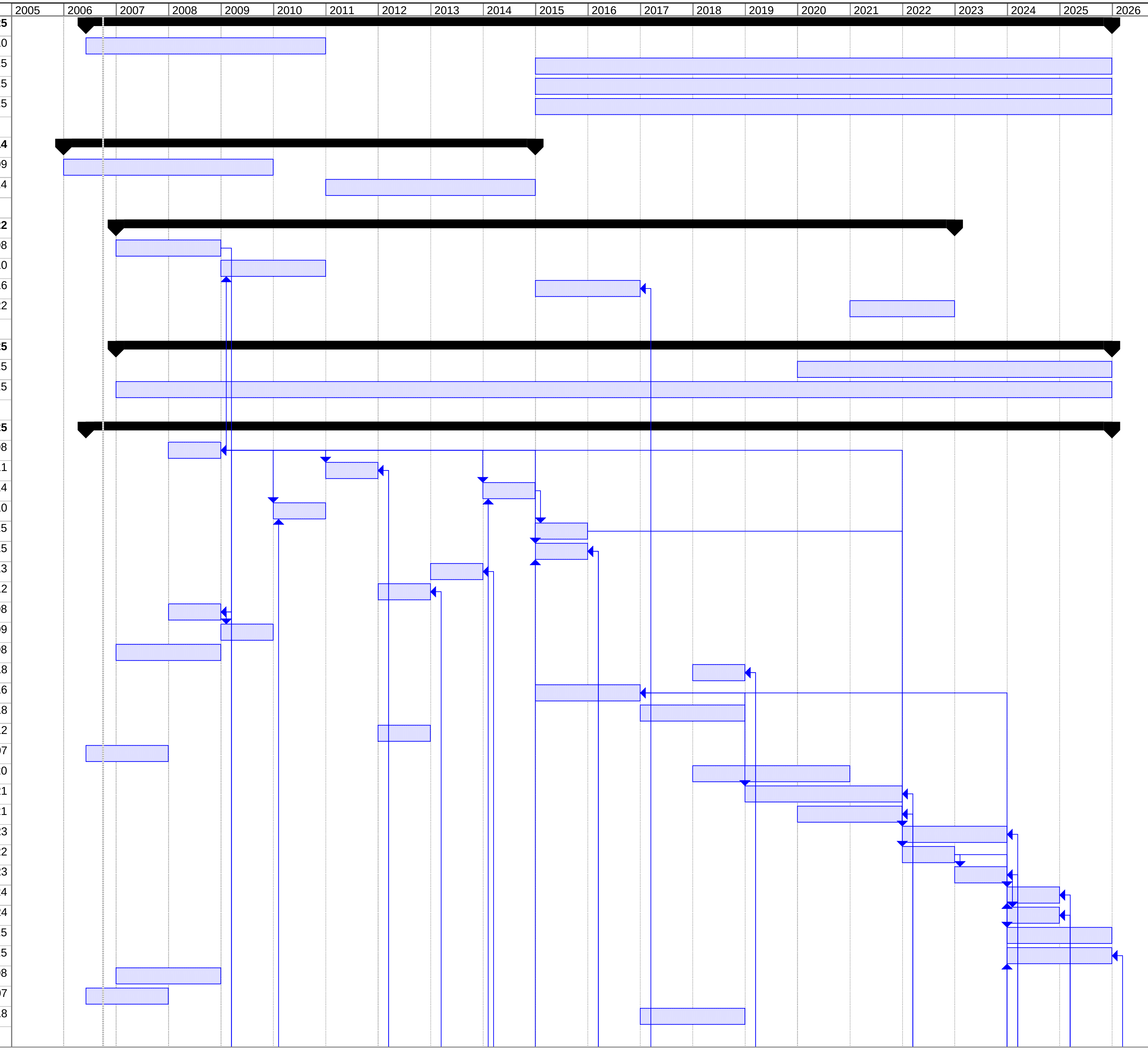
Based on the projected demand, it appears that the City will have sufficient capacity in the Bridge Creek Reservoir to meet anticipated year 2025 projected demands. If water usage increases significantly from expected demands, or flows into the Bridge Creek basin are much less than the projected flows described in this Report, the development of an alternative water source will be necessary. One potential option that could be developed in the future is a cross-basin diversion with a small empoundment reservoir and a raw water pump to transfer water from Twitter or Fritz Creeks to the upper Bridge Creek drainage basin.

APPENDIX D

Proposed Capital Improvement Program Schedule

City of Homer
Conceptual CIP Schedule

ID	Phase	Task Name	Duration	Start	Finish
1	1	**Water Supply**	**7149 days?**	**Tue 6/6/06**	**Wed 12/31/25**
2	1.1	Twitter Creek/Bridge Creek hydrology study	1670 days?	Tue 6/6/06	Fri 12/31/10
3	1.2	Twitter Creek impoundment	4018 days?	Thu 1/1/15	Wed 12/31/25
4	1.3	Twitter Creek pump station (and power requirements)	4018 days?	Thu 1/1/15	Wed 12/31/25
5	1.4	Twitter Creek water main (16,000 to 18,500 LF)	4018 days?	Thu 1/1/15	Wed 12/31/25
6					
7	2	**Water Treatment Plant**	**3287 days?**	**Sun 1/1/06**	**Wed 12/31/14**
8		Upgrades to meet Year 2005 Demands	1461 days?	Sun 1/1/06	Thu 12/31/09
9		Upgrades to meet Year 2025 Demands	1461 days?	Sat 1/1/11	Wed 12/31/14
10					
11	3	**Water Storage**	**5844 days?**	**Mon 1/1/07**	**Sat 12/31/22**
12	3.1	New 1 MG Storage Tank	731 days	Mon 1/1/07	Wed 12/31/08
13	3.2	New 1 MG Storage Tank	730 days?	Thu 1/1/09	Fri 12/31/10
14	3.3	New 1 MG Storage Tank	731 days?	Thu 1/1/15	Sat 12/31/16
15	3.4	New 1 MG Storage Tank	730 days?	Fri 1/1/21	Sat 12/31/22
16					
17	4	**Water R&R**	**6940 days?**	**Mon 1/1/07**	**Wed 12/31/25**
18	4.4	West Trunk Replacement (WTP to Meadows Ave)	2192 days?	Wed 1/1/20	Wed 12/31/25
19	4.5	4-inch and 6-inch main replacements	6940 days?	Mon 1/1/07	Wed 12/31/25
20					
21	5	**Water System Expansions**	**7149 days?**	**Tue 6/6/06**	**Wed 12/31/25**
22	5.1	West Hill Road / Jeffery Avenue Extension	366 days	Tue 1/1/08	Wed 12/31/08
23	5.2	West Hill Road Extensions	365 days	Sat 1/1/11	Sat 12/31/11
24	5.3	Eagle View Drive and Surrounding	365 days	Tue 12/31/13	Tue 12/30/14
25	5.4	East Highland Drive	365 days	Fri 1/1/10	Fri 12/31/10
26	5.5	Garden Park Drive & Highland Drive (Partial)	365 days	Wed 12/31/14	Wed 12/30/15
27	5.6	Foothills Subdivision, Lillian Walli Estate, & Saltwater Dr.	365 days	Thu 1/1/15	Thu 12/31/15
28	5.7	W. R. Benson's Subdivision	365 days	Tue 1/1/13	Tue 12/31/13
29	5.8	Virginia Lyn Subdivision	365 days	Mon 1/2/12	Mon 12/31/12
30	5.9	East Hill Road to Mountain View Drive Extension	365 days	Wed 1/2/08	Wed 12/31/08
31	5.10	Shellfish / Kallman	365 days	Thu 1/1/09	Thu 12/31/09
32	5.11	Forget Me Not Lane / Paintbrush	731 days	Mon 1/1/07	Wed 12/31/08
33	5.12	N. East Hill Road / Cottonwood Dr.	365 days	Mon 1/1/18	Mon 12/31/18
34	5.13	Mission Avenue	731 days?	Thu 1/1/15	Sat 12/31/16
35	5.14	Scenic View Dr. & Other	730 days?	Sun 1/1/17	Mon 12/31/18
36	5.15	Sterling Highway (Lake St. to Greatland St.) & Misc.	365 days	Sun 1/1/12	Sun 12/30/12
37	5.16	Kachemak Drive	574 days	Tue 6/6/06	Mon 12/31/07
38	5.17	Parson / Cape Douglas	1096 days	Mon 1/1/18	Thu 12/31/20
39	5.18	Tietjen Subdivision	1096 days	Tue 1/1/19	Fri 12/31/21
40	5.19	Spencer Drive / Larry Lane	730 days	Wed 1/1/20	Thu 12/30/21
41	5.20	Tulin Terrace Subdivision	730 days	Sat 1/1/22	Sun 12/31/23
42	5.21	W. Highland Drive / Sprucewood Dr.	365 days	Sat 1/1/22	Sat 12/31/22
43	5.22	Sterling Highway	365 days	Sun 1/1/23	Sun 12/31/23
44	5.23	W. Sterling Highway / W. Rogers Loop	366 days	Mon 1/1/24	Tue 12/31/24
45	5.24	Mount Augustine Subdivision	365 days	Mon 1/1/24	Mon 12/30/24
46	5.25	Kachemak City	731 days	Mon 1/1/24	Wed 12/31/25
47	5.26	Skyline Drive	731 days	Mon 1/1/24	Wed 12/31/25
48	5.27	South Peninsula Hospital Main	731 days?	Mon 1/1/07	Wed 12/31/08
49	5.28	Install Additional Hydrants	574 days?	Tue 6/6/06	Mon 12/31/07
50	5.29	Fire Flow Improvements - High School	730 days?	Sun 1/1/17	Mon 12/31/18
51					

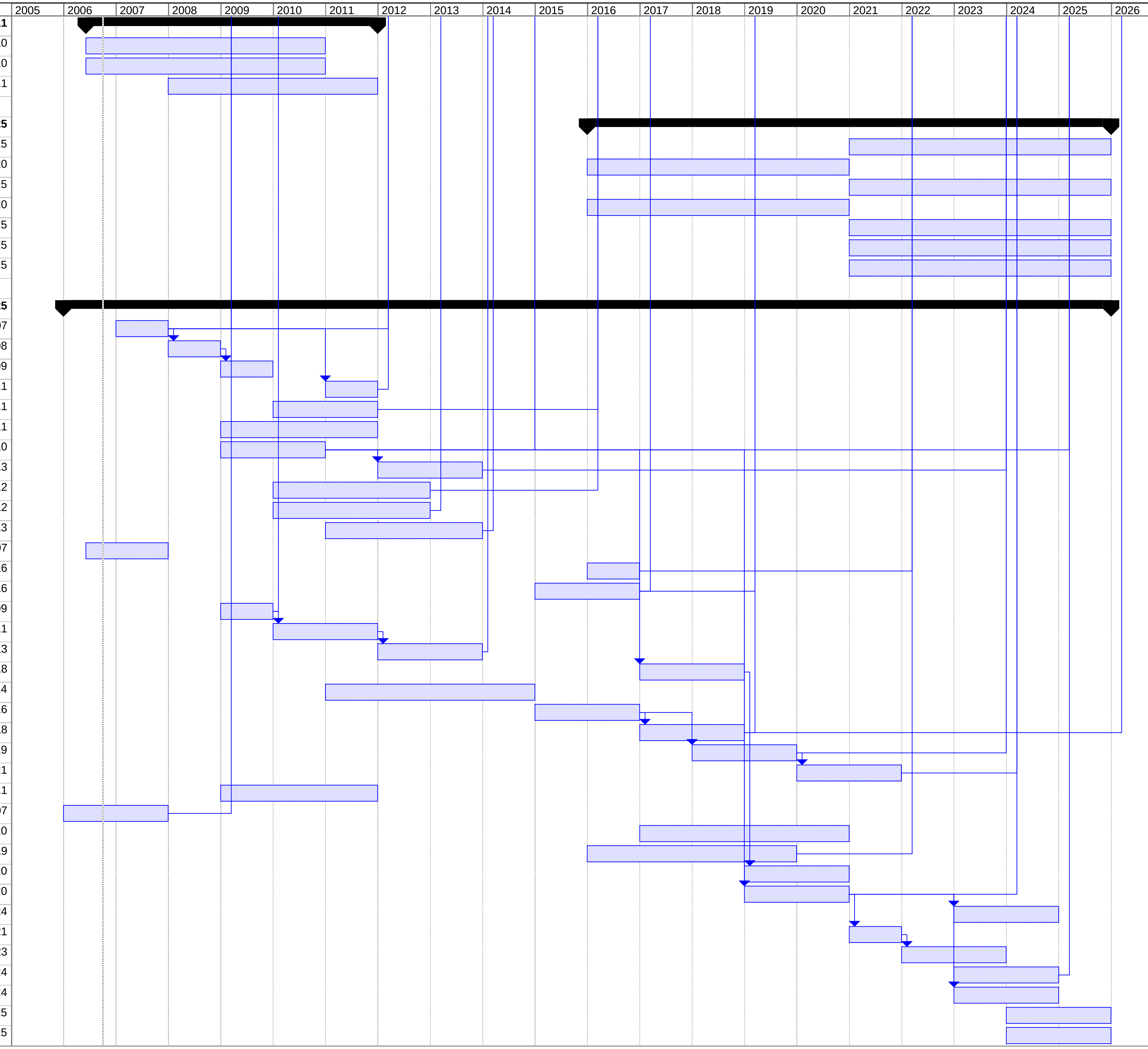


Project: Proposed CIP Schedulev3
Date: Thu 9/28/06

Task | Milestone | Summary

City of Homer
Conceptual CIP Schedule

ID	Phase	Task Name	Duration	Start	Finish
52	6	**Wastewater Treatment Plant**	**2035 days?**	**Tue 6/6/06**	**Sat 12/31/11**
53		Offline Equilization Basin	1670 days?	Tue 6/6/06	Fri 12/31/10
54		Sludge Dewatering	1670 days?	Tue 6/6/06	Fri 12/31/10
55		Capacity Upgrade	1461 days?	Tue 1/1/08	Sat 12/31/11
56					
57	7	**Sewer R&R**	**3653 days?**	**Fri 1/1/16**	**Wed 12/31/25**
58	7.2	Sterling Highway (Soundview / Thomas)	1826 days?	Fri 1/1/21	Wed 12/31/25
59	7.3	Sterling Highway (Thomas St. to Waddell)	1827 days?	Fri 1/1/16	Thu 12/31/20
60	7.4	Bunnell Ave. (Greatland to Beluga)	1826 days?	Fri 1/1/21	Wed 12/31/25
61	7.5	East line entering WWTP	1827 days?	Fri 1/1/16	Thu 12/31/20
62	7.6	Mattox Road to Lake Street	1826 days?	Fri 1/1/21	Wed 12/31/25
63	7.7	East trunk to WWTP	1826 days?	Fri 1/1/21	Wed 12/31/25
64	7.8	Ocean Drive	1826 days?	Fri 1/1/21	Wed 12/31/25
65					
66	8	**Sewer Expansions**	**7305 days?**	**Sun 1/1/06**	**Wed 12/31/25**
67	8.1	West Hill Road (Middle)	365 days	Mon 1/1/07	Mon 12/31/07
68	8.2	Tundra Rose Road to West Hill Rd. / Highland Dr.	365 days	Tue 1/1/08	Tue 12/30/08
69	8.3	West Hill Road (Upper)	365 days	Wed 12/31/08	Wed 12/30/09
70	8.4	West Hill Road Extensions	365 days	Sat 1/1/11	Sat 12/31/11
71	8.5	Bell Subdivision	730 days	Fri 1/1/10	Sat 12/31/11
72	8.6	East Hill Road / Christensen Tracts	1095 days	Thu 1/1/09	Sat 12/31/11
73	8.7	Mount Augustine Dr.	730 days	Thu 1/1/09	Fri 12/31/10
74	8.8	Sterling Highway (Phase I)	730 days	Sun 1/1/12	Mon 12/30/13
75	8.9	Lillian Walli Estate Subdivision	1096 days	Fri 1/1/10	Mon 12/31/12
76	8.10	Mattox Subdivision	1096 days	Fri 1/1/10	Mon 12/31/12
77	8.11	Benson Subdivision	1096 days	Sat 1/1/11	Tue 12/31/13
78	8.12	Kachemak Drive (Phases I-III)	574 days?	Tue 6/6/06	Mon 12/31/07
79	8.13	Paradise South Subdivision / Scenic View Estates	365 days	Fri 1/1/16	Fri 12/30/16
80	8.14	Mission Road / Eker Estates Subdivision	731 days	Thu 1/1/15	Sat 12/31/16
81	8.15	Highland Drive	365 days	Thu 1/1/09	Thu 12/31/09
82	8.16	Natalie Woods / Emerald Highland Estates	730 days	Fri 1/1/10	Sat 12/31/11
83	8.17	Eagleview Subdivision	730 days	Sun 1/1/12	Mon 12/30/13
84	8.18	Mountain Park Subdivision	730 days	Sun 1/1/17	Mon 12/31/18
85	8.19	Willow Drive / Spruce Circle to Larkspur Ct.	1461 days	Sat 1/1/11	Wed 12/31/14
86	8.20	Bayview Gardens Subdivision	731 days	Thu 1/1/15	Sat 12/31/16
87	8.21	East Hill Road	730 days	Sun 1/1/17	Mon 12/31/18
88	8.22	Tulin Terrace	730 days	Mon 1/1/18	Tue 12/31/19
89	8.23	West Terrace Blvd.	730 days?	Wed 1/1/20	Thu 12/30/21
90	8.24	Vineyard Estates	1095 days	Thu 1/1/09	Sat 12/31/11
91	8.25	South Slope Rd.	730 days	Sun 1/1/06	Mon 12/31/07
92	8.26	East End Road Extensions	1461 days	Sun 1/1/17	Thu 12/31/20
93	8.27	Tiejen Subdivision	1460 days	Fri 1/1/16	Mon 12/30/19
94	8.28	West Highland Drive	731 days	Tue 1/1/19	Thu 12/31/20
95	8.29	Sterling Highway (Phase II)	731 days	Tue 1/1/19	Thu 12/31/20
96	8.30	East Rogers Loop	731 days	Sun 1/1/23	Tue 12/31/24
97	8.31	Sprucewood Drive	365 days	Fri 1/1/21	Fri 12/31/21
98	8.32	Bayridge Road	730 days	Sat 1/1/22	Sun 12/31/23
99	8.33	Sterling Highway Phase III	731 days	Sun 1/1/23	Tue 12/31/24
100	8.34	Lake St. / Sterling Highway Force Main	731 days	Sun 1/1/23	Tue 12/31/24
101	8.36	Scenic Place/Horizon Court	731 days?	Mon 1/1/24	Wed 12/31/25
102	8.37	Garden Park Drive (North end)	731 days?	Mon 1/1/24	Wed 12/31/25



Project: Proposed CIP Schedulev3
Date: Thu 9/28/06

Task | Milestone | Summary

APPENDIX E

Council Resolution Adopting the Water and Sewer Master Plan

City Clerk

CITY OF HOMER
HOMER, ALASKA

RESOLUTION 06-121

A RESOLUTION OF THE CITY COUNCIL ACCEPTING THE JULY 2006 FINAL HOMER WATER AND SEWER MASTER PLAN BY BRISTOL ENVIRONMENTAL AND ENGINEERING SERVICES CORPORATION AS THE OFFICIAL PLAN FOR THE CITY OF HOMER.

WHEREAS, Council awarded the Professional Planning and Engineering Services Contract for a Water and Sewer Master Plan to the Firm of Bristol Environmental and Engineering Services in the amount of $291,126.00 via Resolution 03-75 on August 25, 2003 during a Regular City Council Meeting; and

WHEREAS, There were some public meetings held regarding the Water and Sewer Master Plan; and

WHEREAS, The Public Works and Bristol Environmental gave a presentation to the Council on the Water and Sewer Master Plan during the Council's Committee of the Whole meeting on January 23, 2006; and

WHEREAS, The Final Water and Sewer Master Plan was submitted to the City July, 2006.

NOW, THEREFORE BE IT RESOLVED that the City Council hereby accepts the July 2006 Final Homer Water and Sewer Master Plan by Bristol Environmental and Engineering Services Corporation as the as the Official Plan for the City of Homer.

PASSED AND ADOPTED by the City Council of Homer, Alaska this 28th day of August, 2006.

CITY OF HOMER

James C. Hornaday

JAMES C. HORNADAY, MAYOR

ATTEST:

Mary L. Calhoun

MARY L. CALHOUN, CMC, CITY CLERK

Fiscal Note: NA

APPENDIX F

Public Meetings(s)/Participation

Homer Water and Sewer Master Plan
65% Submittal
Meeting of the Whole – Presentation

March 29, 2005

Background: This project consists of the preparation of a 20-year master plan for the water and sewer systems. The overall objective is to evaluate the existing systems, identify growth, analyze impacts to the existing systems, and recommend improvements. The water system includes the water source, water treatment plant facilities, storage tanks, transmission mains, and the distribution network. The sewer system includes the sewer collection piping and interceptor sewer mains, and the sewer treatment plant.

In addition to the overall study, the master planning effort includes detailed mapping of the sewer collection system and the water distribution system using GIS software. This software is being used by the City independently to map other important City resources, development considerations, and geographic features. Combining the work already completed (collecting aerial photography, topographic contours, floodplain information, zoning districts, beach erosion characteristics, etc.) with infrastructure mapping will provide a valuable tool in evaluating growth and development in our community. The project installed a GIS base station inside the City limits (roof of City Hall). With the handheld GIS data collector, staff has precisely located (within centimeters) all water valves, fire hydrants, sewer manholes, and cleanouts. Other features can be located in the future (ie: service lines, building projects, storm drain systems, and road projects, etc. The maps prepared using this information will be incorporated into the City's overall GIS program. A computer model is being created that will allow staff to evaluate effects of growth on the existing water and sewer systems and help staff understand the impacts of growth. Future councils and commissions can use this information to make good decisions regarding development and long term planning efforts.

The 65% draft plan reflects the review of the existing infrastructure and quantifies the capacities of the systems. It projects where the systems will expand and identifies alternatives to solving capacity constraints. No specific recommendation will be made until City staff, City Council and area residents have had an opportunity to review and comment. These comments will be incorporated into the final report containing specific recommendations regarding future water source development, water and sewer treatment plant improvements, and water distribution and sewer collection system capital project requirements. The final master plans are expected to be complete by the end of 2005.

Talking Points:

Population Growth Projections (page 10)
Projected Water Production Rates (see page 40)
Impacts of New Water Treatment Regulations
Water and Sewer Piping Expansions (see grid map inserts)
Water and Sewer Treatment Plant Capacities (see appendix A and B)
Water System Complexities (see pressure zone map in clear plastic folder)

Bristol
ENVIRONMENTAL & ENGINEERING
SERVICES CORPORATION

2000 W. International Airport Road, #C-1
Anchorage, AK 99502-1117
907-563-0013 Phone
907-563-6713 Fax

MEMO

DATE: May 23, 2005

TO: File No. 24047

FROM: Kyle Petersen

RE: Homer Water and Sewer Master Plan
65% Review Meeting on 5/05/05 and 5/06/05

In attendance:

Homer: Carey M. and Jim H.
BEESC: Jim V. and Kyle P.
GVJ: Greg J. and Lisa W. (5/05/05 only)

WATER TREATMENT

- Modify WTP versus build a new WTP
- Have $2.9 million available for improvements to Bridge Creek.
- The existing filters were adequate for 1 NTU treatment, even 0.5 NTU, but the 0.3 NTU limit is causing operational problems. Surface water quality too much for filters. The filters are being overloaded. The filter media may not be right.
- Combination of high water use and high algae this summer will further compound problems. Can raise and lower intake at Dam. Inlet in 30' of water was set at 5' (below surface) last summer.
- Alternate water supply: Wells are not a good option. Twitter Creek probably most feasible option. Fritz Creek is further away. If Fritz Creek is added, there would be more potential people served. Cross basin pipeline with small impoundment at either source is a potential option. May be benefit to having WTP at both Bridge and Fritz Creek, but will have high O&M costs.
- Filter backwash (water loss): 2-10% is normal. 30% was rate for last summer. Estimated that 25% of backwash goes to ponds – some water lost due to percolation and evaporation.
- What is build out potential for Bridge Creek?
- "Pretreatment" options at WTP include package plants, but DBPs can still be a problem.

- The City has tried new polymers, but not necessarily the latest blends.
- How to reduce the volume of solids entering the WTP (i.e. treat at reservoir)?
- DAF:
    - Footprint size
    - Good at removing algae.
- Need to ID algae, organics, and metals. Consensus to remove organics. Algae & Surface water turbidity are overloading the filters/media. Is reservoir treatment possible for algae removal? Killing algae would convert the algae to sediment.
- The City wants to prepare an RFP for "pretreatment" ASAP.
- WTP designed under 1983 regulations – overloaded filters can't get to 0.3 NTU.
- DPBs are a concern.
- The City will be looking at a higher population growth in Homer than what is shown in the 65% report (i.e. more than 3%).
- Possible to supplement Bridge Creek Reservoir with Twitter or Fritz Creek water [small impoundments may be necessary]. Permitting issues may be more of a problem for Twitter (Anchor River drainage).
- Funding issues may prevent major WTP improvements for the present.
- Better to build 1 WTP with redundant operations than two separate plants.
- Water loss due to backwashing is 10% of production. Had a loss of 25% of water to backwash last summer. City is looking at purchasing bigger decant pumps for backwash @ WTP.
- Define logical expansion area (to east) for new WTP. Room on other (east) side of section line for WTP expansion. Additional treatment may be possible at the pump station.
- Room is available for a coagulation / flocculation / sedimentation process. A better coagulant than alum (a derivative perhaps) may be possible. May need a bench scale test. Conventional treatment may not be possible with existing funds.
- Pretreatment with organics removal.
- Membrane filtration would have a large footprint, could remove all DBP in conjunction with conventional filtration; would have a 15% water loss for membrane system.
- Any upgrades to the existing filtration system that include nanofiltration will have capacity problems if the existing (unmodified) filtration system is used.
- Need to study treatment for algae, organics, color, and metals. Unknown if algae treatment with copper sulfate would increase LCR problems. Chemical treatment of algae may cause issue with USFW regarding Anchor River fish populations.

- DBP samples were better during the last testing.
- Sediment build up at Bridge Creek Reservoir. Will have to be removed, possibly by dredging.
- Discussed reasonable level of effort that RFP should address.
- Treatment Options
  - In Reservoir Option: Copper Sulfate, drops organics to bottom as sediment.
  - Pretreatment of raw water prior to direct filtration: Remove organics and enhance Cl_2 level situation.
  - Put polishing filter downstream of direct filtration.
  - Look at conventional treatment versus direct filtration. Build a whole new conventional filtration plant (east of section line). Maybe use membrane filtration.
- If a new WTP is built, can the raw water be sent directly to the WTP?
  - Tube membrane filters have 15% water loss.
  - Nanofiltration (spiral round membrane) have 15% water loss, and needs granular filtration upstream.
- City: How big is footprint for membrane technology?
- City: Is potassium permanganate (greensand) an option?
- City: Need to revisit population projections and water demands.
- Discuss options with "Lo Press" (existing WTP)
- City wants memo from GVJ&A recommending bench and pilot testing, as well as other testing to determine viable options.
- Mention desalination as an option.
- Reservoir turnover is an issue.

WASTEWATER TREATMENT

- Impact of I/I. Contributing source (roof drains, foundation drains, sump pumps, etc.)?
- WSMP showed I/I as a real issue of concern. Need to show projection for I/I and future wastewater flow. Show cost/timeline to treat I/I versus cost to (partially) remove I/I. Roofs and sump pumps are maybe the maximum contributors. Need ordinances to cover removal of I/I from system.
- Wet weather flowrates – outfall line: high tide pushes flow back into WWTP due to absence of air relief valves. Have installed one air relief valve, but need more.
- Highest daily flowrate has been 2.8 MGD.

- Show flowrate for last 4 years.
  - I/I reduction versus increase plant capacity as needed.
  - Show two dates (for implementation) and costs for WWTP upgrades – one with I/I and one with reduced I/I.
- City would like to optimize the drying bed / dewatering process. Need a faster sludge drying process such as a screw press, belt press, centrifuge, etc. Have a dredge that can process 500-600 gallons per minute with chopper pump (Vaughn). The City produces 1,200 cy of dried sludge annually. Currently dispose of sludge on private property on an annual basis. Need future disposal location for sludge solids.
- Condition the pumped sludge with polymer.
- Freeze drying part of process to produce Class A sludge. Freeze drying – 10% to 12% water content.
- Have put in an air relief at the WWTP outfall. Need more air relief valves. Outfall was designed for current peak flows.
- Leachate collection system for old landfill (ballfield) included in I/I, and can be detrimental to system depending on composition. Leachate can damage UV system. Leachate has very high BOD and iron. Look at discharging someplace other than the WWTP.
- Need to acquire property for future WWTP and WTP improvements. Logical area of expansion for WWTP is to the west.
- Discussed BOD and TSS trends. Definite increase in concentration.

WATER DISTRIBUTION / WASTEWATER COLLECTION

- The City and BEESC discussed and modified preliminary layouts for the water distribution and sewer collection systems.
- Notes on Sheets 1, 2, and 3 on Bridge Creek watershed.
- Discussed West Hill transmission line.
- Move 1 MG tank & WTP details to separate sheet.
- Need better designation symbols for PRVs.
- Get Hillside acres Plat (plans).
- W&S plans or as-builts.
- Get As-builts for Forest Glen.

PRIORITY LISTING

1. WTP Upgrades
    a. Alternate source water – future strategy
    b. Finalize Population (growth estimates)
2. Connect Transmission Main
    a. West Hill Transmission Line
    b. East Hill Transmission Line
    c. Tie to new tanks
3. Storage Requirements
    a. East Hill
    b. West Hill
4. West Hill Transmission Main
5. Sludge Management (Dewatering) + Acquire Easement/Land
6. I/I Detention Basin
7. Third Deep Shaft & Clarifier
8. Septage Receiving (pretreatment / conditioning)
9. Upsize Sewer (Ocean Drive and West Main)
10. W. Transmission Line – replace WTP to Hilltop PRV with 12'' main and one new PRV (Ridgeline)
11. Upsize water line at High School
12. Paintbrush Booster Station

Attachment: Water Treatment Options

Bristol
ENVIRONMENTAL & ENGINEERING
SERVICES CORPORATION

2000 W. International Airport Road, #C-1
Anchorage, AK 99502-1117
907-563-0013 Phone
907-563-6713 Fax

MEMO

DATE: November 8, 2004

TO: File No. 24047

FROM: Jim Vogel

RE: Homer WSMP
Teleconference on 11/04/04 at 10:00 a.m.

In attendance:

Homer: Carey M. and Jim H.
BEESC: Jim V. and Kyle P.
GVJ: Greg J. and Lisa W.

WATER TREATMENT

1. Flows – use percentage of flow model versus projected population densities in areas of expansion (what about impact of water haulers?).
2. Discussed WTP with Greg J. and Lisa W. – can we continue to meet ESWTR with direct filtration?
3. Carey – Overview – discussed GVJ scope letter.
    a. The objectives are good (1st page).
    b. Most WSMP improvements are out 15+ years.
    c. City feels something needs to be done with WTP.
    d. Look at alternative water source – where will it be? Is Bridge Creek adequate?
    e. City is working on $2.9 million loan for WTP improvements; there may be additional ADEC Matching Grant funding.
    f. Should money go to pretreatment of Bridge Creek raw water; or just consider a new WTP?
    g. Surface water versus groundwater.

        Make assumptions:

- Jim H. – WTP operations are as good as it gets; can we pretreat raw water?
- Reservoir water quality is degrading – can't keep up with demand with existing WTP, given increased backwash frequency.
- Look ASAP at pretreatment of reservoir water.
- What is groundwater (GW) quality of existing wells in vicinity of WTP? Can we blend?

4. Greg Jones
   a. Options include GW reserves
   b. Low rainfall and algae growth (due to high water temperatures) in reservoir this summer.
   c. No real good place to get algae-free and less turbid water by changing withdrawal point in reservoir.
   d. Can we pre-treat reservoir?
      - $CuSO_4$
      - Potassium permanganate pretreatment by dosing.
      - Algae (EPS) gets sticky.
      - Chlorine oxide.
      - After coagulant program – automated instruments for coagulant dosing.
      - Streaming current detector (Homer already has one).
      - Ferric-based coagulants.
      - Newer polermeric blends of ferric and aluminum.
      - Minimize formation – mechanical stirring and aeration (in reservoir).
      - Nutrient controls.
      - Put fish species in reservoir that consume algae.
      - Dissolved air flotation to remove algae.
      - Micro-sand enhanced flocculation.
      - Membrane filter systems.
5. BEESC and GVJ working together
   a. BEESC – Bridge Creek source (reservoir levels dropped in July and August 2004).
   b. Reservoir level approximately 1 foot below "Flower" (outlet/overflow structure).
   c. What is future capacity of reservoir? BEESC to do hydrology.

- When do we need new source? Where and how much?

6. Alternative Water Sources
   a. Recommendation for:
      - Short-term fix
      - Long-term fix
        - raise dam
        - new source?
7. Look at well(s)
   a. Review casings/wells near WTP and reservoir
      - Twitter Creek Drainage
      - Fritz Creek Drainage
      - Well capacity of 200 gpm+ is needed
      - Can we blend well water?
      - What extra work can City do to look at well options?
8. Performance goals
   a. Include additional/alternative water supply
9. WTP
   a. "Conventional Wisdom" for pretreatment of raw water- drop algae and turbidity, to reduce filter runs. Is this possible?

WASTEWATER TREATMENT

1. "Conventional Wisdom" is to drill a second deep shaft aeration cell.
2. With anticipated growth the STP volume will be "pushed" or maxed out.
3. Address handling of sludge and disposal.
   a. Want proper emphasis on sludge handling, including status changes.
      - Sludge lagoon,
      - Sludge drying beds,
      - Capacity and disposal – include problems, and
      - Future: go to mechanical dewatering? (Still have disposal issues)

COSTING AND BUDGETING

1. Estimated cost of alternatives – main focus.

SEWER PIPING

1. Discussed AC pipe – is it holding up okay?
2. Jim H. – when bedded and backfilled with native materials – pipe is protected.

FOLLOW-UP DISCUSSION
November 5, 2004

1. Carey M. called to confirm areas of emphasis in WSMP.
2. Don't spend a lot of time looking at and analyzing existing WTP and processes to solve future problems. Look at pretreatment – what is long-term solution?
3. The source water is key.
   a. What is long-term solution?
      - Bridge Creek capacity?
      - Stay at WTP site or new?
      - Twitter creek (to Bridge Creek reservoir)?
      - Fritz Creek?
   b. Look at existing source – how to improve or find new one?
   c. Pretreatment – with existing WTP – is this possible?
4. Task 3 of GVJ Scope Letter: focus on looking at supplemental processes and pretreatment. Again – don't spend too much time analyzing existing WTP data thus concentrating effort on "juicing up" existing WTP. Look at new alternatives, what is best for future. How can WTP work more efficiently in future.
5. Task 4 of GVJ Scope Letter: less effort on No. 1 and No. 3 more on No. 4 and No. 5. No. 4: use wording – upgrades/supplemental, pretreatment, and alternative treatment(s). No. 5: more costing and budget focus on alternatives and less on recommended upgrades or processes.

DRAFT WSMP TABLE OF CONTENTS (TOC)

1. We need to prepare an outline of the TOC for Carey to review and comment. This includes our first cut on proposed attachments.

PROJECT MEETING

First half of December in Homer.

APPENDIX G

Tsunami Inundation Scenarios for Homer, Alaska

ALASKA DIVISION OF GEOLOGICAL & GEOPHYSICAL SURVEYS
FY04 Project Description

TSUNAMI INUNDATION MAPPING

With funding from Congress, the National Oceanic & Atmospheric Administration (NOAA) and Federal Emergency Management Agency (FEMA) initiated the National Tsunami Hazard Mitigation Program in 1997 to assist Pacific states in reducing losses and casualties from tsunamis. The program includes funding for five states (Alaska, Hawaii, Washington, Oregon, and California) to address four primary issues of concern: (1) quickly confirm potentially destructive tsunamis and reduce false alarms, (2) address local tsunami mitigation and the needs of coastal residents, (3) improve coordination and exchange of information to better utilize existing resources, and (4) sustain support at state and local level for long-term tsunami hazard mitigation.

As part of this program, DGGS is participating in a cooperative project with the Alaska Division of Emergency Services (ADES) and the University of Alaska Geophysical Institute (UAGI) to prepare tsunami-inundation maps of selected coastal communities. Kodiak was the first community selected for this project. During FY02, we completed and published maps of the Kodiak area to show estimated extent of inundation from tsunamis generated by seven hypothetical distant and nearby earthquakes. As a result of a meeting of local, state, and federal representatives in 1999, nine additional communities were selected and prioritized for future inundation mapping based on population, tsunami exposure, community interest, and data availability. Homer and Seldovia are the next communities for which we are currently preparing inundation maps based on two earthquake scenarios (see figure below). Maps and a report for Homer and Seldovia will be completed in FY2004. We have begun data compilation and inundation-modeling work for the next community, Seward, for which maps will be completed in FY2005.

In this program, we are developing inundation maps using complex numerical modeling of tsunami waves as they move across the ocean and interact with the seafloor and shoreline configuration in shallower nearshore water. UAGI is conducting the wave modeling using facilities at the Arctic Region Supercomputing Center. DGGS imports the results of this modeling to a Geographic Information System (GIS) database for use in depicting projected inundation limits on suitable base maps. DGGS, UAGI, and ADES meet with community leaders to communicate progress and results of the project, discuss format of resulting maps, and obtain community input regarding past tsunami effects and extent. DGGS publishes the final maps along with explanatory text, which are available in both hardcopy and digital formats. DGGS also makes the GIS files of inundation-limit lines available to the local communities for use in preparing their own tsunami-evacuation maps.

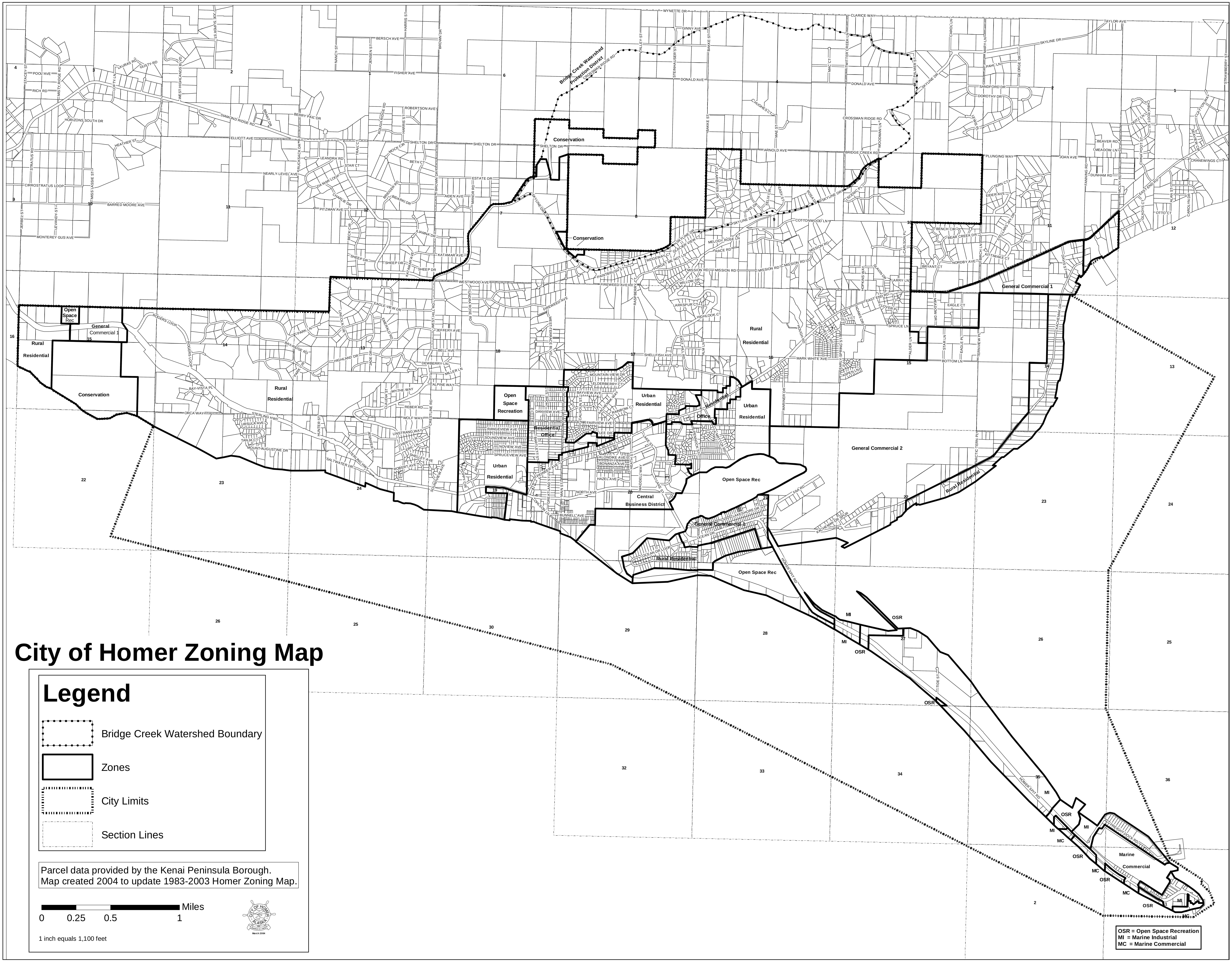
Draft tsunami-inundation map for Homer, Alaska. Red line: Estimated inundation limit for a repeat of the 1964 magnitude 9.2 earthquake. Yellow line: Estimated inundation limit for a hypothetical magnitude 7.5 earthquake on the Border Ranges fault.

During preparation of the Kodiak maps, comparison of the modeled 1964 inundation with the observed wave run-up in 1964 showed that the model produced comparable inundation. The maps also show that the modeled 1964 inundation nearly everywhere exceeds the inundation from all other credible source earthquakes. We have presented results of this project at international tsunami symposia in Istanbul, Turkey, and Seattle, Washington in 2001, at the Tsunami Society symposium in Honolulu, Hawaii, in 2002, and at the American Geophysical Union meeting in December 2003. This project was the subject of articles in *Geotimes* and *Tsu-Info Alert Newsletter in 2002*.

Contact: Rod Combellick, Chief, Engineering Geology section, 907-451-5007, rod@dnr.state.ak.us

APPENDIX H

City of Homer Zoning Map (2004)


City of Homer Zoning Map
Legend
Bridge Creek Watershed Boundary
Zones
City Limits
Section Lines
Parcel data provided by the Kenai Peninsula Borough.
Map created 2004 to update 1983-2003 Homer Zoning Map.
Miles
0
0.25
0.5
1
1 inch equals 1,100 feet
OSR = Open Space Recreation
MI = Marine Industrial
MC = Marine Commercial
Bridge Creek Watershed Protection District
Conservation
Rural Residential
Urban Residential
Residential Office
Open Space Recreation
Open Space Rec
General Commercial 1
General Commercial 2
Central Business District
Marine Commercial

APPENDIX I

Water and Sewer Fee Schedule

Uplands Storage:
Land for Gear Storage-
First come-first served basis; approved by Harbormaster; primarily for fishing related gear.

Open areas, fishing gear	0.10/ sq. ft.
Open areas, non-fishing gear	0.14/ sq. ft.
Fenced storage yard	0.20/ sq. ft.

Boat Trailers-
Short term storage, up to 7 days - space available basis - no fee.
Long term storage, 8 days or more - not available May 1 to Oct 1

Up to 30 feet	$ 75.00/month Oct 1 to May 1
Over 30 feet	$100.00/month Oct 1 to May 1

TIDAL GRIDS:
The City of Homer operates two tidal grids. The wooden grid is for vessels of less than 60 feet in length. The steel grid is only for use by vessels of 60 feet or greater in length. Vessels that remain on either grid after their scheduled tide may be assessed a 50% surcharge for each unscheduled tide. Use of the steel grid shall be charged at the minimum rate applicable for a 60' boat if a boat of less length is allowed to use this grid.

The rate per foot per tide is $0.82 for vessels 0' - 59'
The rate per foot per tide is $1.99 for vessels 60' - 80'
The rate per foot per tide is $2.50 for vessels 81' - 100'
The rate per foot per tide is $2.93 for vessels 101' - 120'
The rate per foot per tide is $3.30 for vessels 121' - 140'

ATER: Potable water furnished to vessels at the Deep Water Dock and Main Dock:
Quantity charge - $30.75 per one thousand gallons (minimum five thousand gallons).
Scheduled deliveries will have a minimum charge of eighty ($82.00) dollars for combined connection and disconnection.
Unscheduled deliveries will have a minimum charge of one hundred three ($108.50) dollars for combined connection and disconnection.

ELECTRICITY(per kilowatt):
Reserved stalls having a meter base at the berth shall be charged a meter availability fee.
The meter availability fee - $18.50 per month
Connect/disconnect fee - $22.25

Metered transient vessels will be charged a meter availability fee of $22.25 per month with a one month minimum charge to be applied for shorter connection periods.

Unless other arrangements have been made in writing with the Harbormaster, transient vessels shall be charged the following rates (where metered power is unavailable).

	110 volt	220 volt	208 volt/3-phase
Daily (or part thereof)	$ 8.75	$ 17.25	$38.50
Monthly	$127.50	$268.50	available meter only

Vessels requiring conversion plugs may purchase them from the Harbormaster's office for a nominal fee.

Basic Life Support	$350
Advanced Life Support	$500
Non-Emergency Transport	$350
Standby each half hour	$ 25
Mileage	$7/mi

FIRE:

Type 1 Engines	$200/hr.
Type 1 Tenders	$250/hr.
Brush Patrol	$100/hr.
Command Vehicle	$ 50/hr.
Rescue Medical Unit	$100/hr.
Utility Vehicle	$ 50/hr.

VOLUNTEER PERSONNEL:

Fire Department IC (1) (IC - Incident Command)	$25/hr.
Driver/Engineer 1 each vehicle	$20/hr.
Fire Fighter (Minimum 1/tender and; 2/Engine)	$15/hr.
EMT (Minimum 2 per Rescue Medical Unit)	$15/hr.

PUBLIC WORKS DEPARTMENT

Administrative - 235-3170

(The following fees have been set by legislative enactment Resolution 95-1).

R.V. Station dumping	$2 per dumping
Bluelines, copies minimum	$10 + $2/pg.
Standard Construction Specs	$50

Job Specific Specifications and plans vary in price.

LOCAL IMPROVEMENT DISTRICTS (LID's):

HARP (Homer Accelerated Roads Program) LID's
Assessments are: $30 per front foot for Road Reconstruction
$17 per front foot for Paving

HAWSP (Homer Accelerated Water and Sewer Program) LID's
Assessments are: 75% of the total project cost allocated in equal shares to each participating parcel

OTHER LID's if approved by the Council are at 100% property owner participation.

WATER AND SEWER FEES:

(The following fees have been set by the following legislative enactment HCC Title 14, new fees set forth in Resolution 03-159, Resolution 02-80, Resolution 01-80(A), Resolution 00-123, Resolution 00-34, Ordinance 00-02, Ordinance 97-17(A), amending the rates set forth in Ordinance 97-5(S)(A), with amendments by Ordinance 97-7, Ordinance 97-13 and Ordinance 97-14).

Public Works - 235-3170
City Hall - 235-8121
B'''ing - 235-3107

A 15% admin. fee for replacement parts for water/sewer services, functions, pressure reducing valves, sewer saddles, any Public Works Department stock item for resale to public.

(Contact Planning Department - at City Hall, 235-3106. Planning issues the permits.)
Establishing service includes a one time disconnect - $30
Service calls, inspections, repairs not to exceed one hour - $25 per employee plus equipment and materials.

Service calls, inspections and repairs during normal operating hours in excess of one hour labor: actual labor costs by City plus equipment and materials.

Service calls, inspections and repairs after normal operating hours or on weekends/holidays: $50 minimum plus equipment and materials or actual cost incurred by City, whichever is greater.

SEWER FEES

Sewer Connection and Extension Permit fee
Single Family/Duplex $255
Multi-Family/Commercial/Industrial $330

Sewer Rate Schedule.

, sewer utility services shall be billed according to the following schedule. This schedule is for monthly sewer services and is in addition to any charges for connecting or disconnecting the service, installation of the service or any assessment of the improvements.

Commodity rate per thousand gallons of water	$8.27
Monthly Customer Charge	$3.82

General Monthly Service Charge and Spit Service Charge by meter size.

Meter size	General Service Charge	Spit Service Charge
5/8	$ 16.93	$ 16.93
3/4	$ 25.40	$ 25.40
1	$ 42.33	$ 42.33
1 2	$ 84.66	$ 84.66
2	$135.45	$ 135.45
3	$296.30	$ 296.30
4	$533.34	$ 533.34
6	$1,185.20	$1,185.20

Sewer System Residential or Residential Equivalent Dischargers Who are not Water System Users: Sewer system dischargers who are not water system users shall be charged at the rate of $49.69 (variable ate $28.94) based on 3,500 gallons per month plus general service charge ($16.93) plus monthly customer charge ($3.82) per month. The City reserves the right to adjust this rate based on the characteristics of the service for non-residential or non-residential equivalent users. Customers who receive septic service shall be charged an additional $4.34 per month.

Sewer System Dischargers Who are Members of Kachemak City LID: Kachemak City Local Improvement District (LID) members have contributed to the initial cost of the sewer treatment plant and the collection system. Kachemak City LID dischargers connected within the LID and the City of Homer shall bill Kachemak City in one lump sum at the rate of $50.21 (variable rate $28.94) based on 3,500 gallons per month plus general service charge ($16.93) plus septage cost ($4.34) per month for each residential or residential equivalent discharger. Kachemak City shall be responsible for payment to the City of Homer.

Domestic sewer service customers who use large quantities of City water in addition to their domestic use shall be allowed, with the Public Works Director's approval, to install an additional water meter on the domestic water use line for the purpose of metering and charging for domestic sewer system use. Sewer system use will be billed monthly.

The City will allow, upon approval by Public Works and a permit from the Planning Department, a second water usage meter - called a seasonal sewer meter for each customer that desires to measure the flow of City water that is not discharged to the sewer system during the summer growing season, June 15 through September 15. Rates noted above do not apply.

Seasonal Sewer Meter Fees: $121.10.

WATER FEES

Water connection fee
Single Family/Duplex $300
Multi-Family/Commercial/Industrial $375
HCC 14.08.035(b)

Water or Sewer main extension/private development/subdivision agreement $actual cost.

Commodity rate per 1,000 gallons of water: $4.55

Monthly Customer Charge: $4.75

General Monthly Service Charge and Spit Service Charge by meter size.

Meter Size	General Service Charge	Spit Service Charge
5/8"	$ 20.51	$ 20.51
3/4"	$ 30.77	$ 30.77
1"	$ 51.28	$ 51.28
1-1/2"	$ 102.56	$ 102.56
2"	$ 164.10	$ 164.10
3"	$ 358.98	$ 358.98
4"	$ 646.16	$ 646.16
6"	$1,435.91	$1,435.91

When the customer uses more than 20,000 gallons per month the following service charges are:

Meter Size	Excess Use Exceeds		General Surcharge
5/8"	20,000 gallons		$ 28.70
[illegible]	70,000 gallons	$ 47.83	
3/4"	70,000 gallons	$ 47.83	

One time service deposit:

Meter Size Deposits (Inches)	Residential Users	Nonresidential Users
5/8	$ 75.00	$ 220.00
3/4	$ 80.00	$ 230.00
1	$ 90.00	$ 250.00
1-1/2	$115.00	$ 310.00
2	$150.00	$ 370.00
3	$220.00	$ 525.00
4	$310.00	$ 730.00
6	$520.00	$1,225.00

BULK WATER FEES:

A commodity charge of $4.55 per 1,000 gallons of water shall apply to bulk water sales provided by tanker truck or fire hydrant plus an additional surcharge of $3.47 per 1,000 gallons of water in order to offset debt service and capital replacement and customer service costs.

A $750 meter deposit shall apply to metered fire hydrant connections. The deposit will be returned when meter is returned undamaged. This deposit may be waived upon the recommendation of the Public Works Superintendent.

If a bulk water customer purchases a meter from the City for measuring the quantity of water purchased, it shall be exempt from the monthly meter service charge. It is the responsibility of the bulk water customer to maintain that meter so the City can accurately determine the amount of water being purchased. In the event the meter fails, it is the bulk water customer's responsibility, at its expense, to repair it or purchase a replacement meter from the City. The City may at any time test the meter for accuracy.

RESIDENTIAL HOLDING TANK FEES

(Resolution 02-23)

City of Homer will bill property owner/customer monthly for City service, not pumping contractor charge.

Each property owner/customer will be billed once each month, regardless of number of pumpings, 1 [one] Customer Charge $3.69 + 1 [one] General Service Charge $12.72 + Commodity Charge [$12.00 per pumping]

Property owner/customer is responsible for payment to pumping contractor.

City of Homer monthly billing examples based upon number of pumpings per month:

Type of Charge	No pumpings	1 mon. pumping	3 mon. pumpings
Customer Charge	$ 3.69	$ 3.69	$ 3.69
Gen. Svs. Charge	$ 12.72	$12.72	$12.72
Commodity Charge	$ 0	$12.00	$36.00
Total monthly bill	$ 16.41	$28.41	$52.41

LEGISLATIVE BODY
MAYOR
Jack Cushing – 2004

COUNCILMEMBERS
Doug Stark – 2004
John Fenske - 2004
Michael Yourkowski - 2005
Rose Beck - 2005
Rick Ladd - 2006
Dennis Novak - 2006

ADVISORY BODIES
Commissions
Parks and Recreation
Planning/Zoning
Port and Harbor
Library Advisory Board

DEPARTMENTS

Located at City Hall
Administration
City Manager's Office
City Clerk's Office
Records and Elections
Personnel

Located at the Fire Hall
Fire Department
EMS
Fire
Rescue

Located at the Public Works Facility
Public Works
Water and Sewer
Maintenance
City Engineer
Inspector

Located at the Public Library
Library

Located at the Police Department
Public Safety
Police Department

Located on the Spit by the Harbor
Port and Harbor

APPENDIX J

Water Rights

STATE OF ALASKA
DEPARTMENT OF NATURAL RESOURCES
DIVISION OF MINING AND WATER MANAGEMENT

PERMIT TO APPROPRIATE WATER

LAS 18902

THE STATE OF ALASKA UNDER AS 46.15, THE ALASKA WATER USE ACT, AND THE REGULATIONS ADOPTED UNDER IT, GRANTS TO:

HOMER, CITY OF
PUBLIC WORKS DEPARTMENT
3575 HEATH STREET
HOMER, AK. 99603

THE RIGHT TO DEVELOP THE FOLLOWING USE OF WATER:

SOURCE:	BRIDGE CREEK	PRIORITY DATE: 03/17/1994
QUANTITY:	900000.0 GAL/DAY	
USE:	PUBLIC SUPPLY	FROM: JAN 01 THRU DEC 31

THE LOCATION OF THIS WATER SOURCE IS:

IDGE CREEK RESERVOIR, LOCATED WITHIN NE/14NE/14 SECTION 7, TOWNSHIP SOUTH, RANGE 13 WEST, SEWARD MERIDIAN.

THE LOCATION TO WHICH THIS WATER RIGHT APPERTAINS IS:

THE CITY OF HOMER'S WATER DISTRIBUTION SYSTEM, WITHIN LOTS 1 THROUGH 6, BLOCK 5, LOTS 1 THROUGH 5 & 35 THROUGH 68, BLOCK 6, LOTS 1 THROUGH 25 & 29 THROUGH 31, BLOCK 7, BAYVIEW SUBDIVISION ADDITION 1, S1/2 SECTION 8, & LOT 18, TRACT A, SE1/4SE1/4 SECTION 10, TOWNSHIP 6 SOUTH, RANGE 13 WEST; LOT 10A, SCENIC VIEW SUBDIVISION, SE1/4 SECTION 10, TOWNSHIP 6 SOUTH, RANGE 13 WEST; S1/2 SECTION 13, S1/2 SECTION 14, & SECTIONS 23 & 24, TOWNSHIP 6 SOUTH, RANGE 14 WEST; S1/2 SECTION 18, SECTIONS 14, 15, 16, 17, 19, 20, 21, 22, 23, 26, 27, 28, 29, 34, 35, & 36, TOWNSHIP 6 SOUTH, RANGE 13 WEST; SECTIONS 1 & 2, TOWNSHIP 7 SOUTH, RANGE 14 WEST, ALL WITHIN SEWARD MERIDIAN; & ALL TIDELANDS LYING BETWEEN THE MEAN HIGH WATER LINE OF COOK INLET AND KACHEMAK BAY AND THE "DIRECTORS LINE", AS SHOWN ON THE PLAT OF ALASKA TIDELANDS SURVEY 612, FILED AS PLAT #77-64, AND FURTHER DESCRIBED IN THE TIDELANDS PATENT #201, RECORDED IN BOOK 80 AT PAGE 171, HOMER RECORDING DISTRICT, THIRD JUDICIAL DISTRICT, STATE OF ALASKA.

PERMIT
TO
APPROPRIATE WATER
LAS 18902

CHANGES IN THE NATURAL STATE OF WATER ARE TO BE MADE IN THE MANNER AND ONLY FOR THE PURPOSES STATED IN THIS PERMIT. THIS PERMIT IS SUBJECT TO THE PERTINENT STATUTORY PROVISIONS IN AS 46.15, ADMINISTRATIVE REGULATIONS IN 11 AAC 93, AND THE FOLLOWING CONDITIONS:

THE HOLDER OF THIS PERMIT SHALL:

FOLLOW ACCEPTABLE ENGINEERING STANDARDS IN EXERCISING THE PRIVILEGE GRANTED BY THIS PERMIT.

DEFEND AND INDEMNIFY THE STATE AGAINST AND HOLD IT HARMLESS FROM ANY AND ALL CLAIMS, DEMANDS, LEGAL ACTIONS, LOSS, LIABILITY AND EXPENSE FOR INJURY TO OR DEATH OF PERSONS AND DAMAGES TO OR LOSS OF PROPERTY ARISING OUT OF OR CONNECTED WITH THE EXERCISE OF THE PRIVILEGE GRANTED BY THIS PERMIT.

COMPLY WITH ALL APPLICABLE LAWS, REGULATIONS, AND CONDITIONS; AND KEEP CURRENT ALL NECESSARY APPROVALS REQUIRED BY OTHER STATE, FEDERAL, AND LOCAL AGENCIES.

NOTIFY THE DIVISION OF ANY CHANGE OF ADDRESS OF THE GRANTEE, TRANSFER OF ANY REAL PROPERTY IDENTIFIED IN THIS DOCUMENT, OR CHANGE IN THE WATER APPROPRIATION.

AS SOON AS YOU BEGIN USING WATER, ESTABLISH A METERING SYSTEM ACCEPTABLE TO THE DIVISION AND SUBMIT MONTHLY WATER USE RECORDS TO THE DIVISION ON A QUARTERLY BASIS FOR THE DURATION OF THIS PERMIT OR UNTIL FURTHER NOTICE FROM THE DIVISION.

GRANTOR: STATE OF ALASKA
DEPARTMENT OF NATURAL RESOURCES
DIVISION OF MINING & WATER MGMT.
SOUTHCENTRAL REGION
3601 C. STREET, SUITE 800
ANCHORAGE, ALASKA 99503-5935

PER 11 AAC 05.010.(A)(8)(M)
AN ANNUAL ADMINISTRATIVE SERVICE FEE SHALL BE ASSESSED ON THIS APPROPRIATION OF WATER.

PERMIT
TO
APPROPRIATE WATER
LAS 18902

THIS PERMIT SHALL EXPIRE ON 12/01/2006

THIS PERMIT TO APPROPRIATE WATER IS ISSUED BY AUTHORITY OF AS 46.15.080 AND 11 AAC 93.120 ON December 2, 1996.

APPROVED: [signature]

TITLE: Chief, Water Resources Section
DIVISION OF MINING AND WATER MANAGEMENT

Certificate No. 4330

Know All Men By These Presents that the State of Alaska, pursuant to A.S. 46.15, as amended and the rules and regulations promulgated thereunder, hereby grants to the CITY OF HOMER, P.O. Box 335, Homer, Alaska 99603

the right to the use of 90,000 gallons per day from the public waters of the State of Alaska for the purposes of public water supply

The location of the water source to which the water right herein granted shall appertain is a dam and impoundment reservoir on Bridge Creek with take point within NE 1/4 NE 1/4 NE 1/4 Section 7, Township 6 South, Range 13 West, Seward Meridian

and the right to said water shall be appurtenant to that certain tract of real property described as follows:

City of Homer municipal
water supply distribution
system

Priority of appropriation began February 28, 1966

To Have and to Hold the said water right with the appurtenances thereof unto the said Grantee and its heirs and assigns forever, subject to the provisions of A.S. 46.15.140-160.

In Testimony Whereof the State of Alaska has caused these presents to be executed by the Director of the Division of Lands pursuant to A.S. 46.15, as amended, this 15th day of February A.D. 19 78

George K. Hollett

Acting Director, Division of Lands/ & Water Manageme

State Record of Water Right Certificates

XVI - 4330. ADL 44449

ww 4-5-78B

CERTIFICATE OF APPROPRIATION OF WATER

Certificate No. 4335

Know All Men By These Presents that the State of Alaska, pursuant to A.S. 46.15, as amended and the rules and regulations promulgated thereunder, hereby grants to the City of Homer, P.O. Box 335, Homer, Alaska 99603 the right to the use of 910,000 gallons per day from the public waters of the State of Alaska for the purposes of public water supply

The location of the water source to which the water right herein granted shall appertain is a dam and impoundment reservoir on Bridge Creek with intake point within NE 1/4 NE 1/4 NE 1/4 Section 7, Township 6 South, Range 13 West, Seward Meridian.

and the right to said water shall be appurtenant to that certain tract of real property described as follows:

City of Homer municipal
water supply distribution
system

Priority of appropriation began July 3, 1972

To Have and to Hold the said water right with the appurtenances thereof unto the said Grantee and its heirs and assigns forever, subject to the provisions of A.S. 46.15.140-160.

In Testimony Whereof the State of Alaska has caused these presents to be executed by the Director of the Division of Lands pursuant to A.S. 46.15, as amended, this 15th day of February A.D. 1978

George K. Hollett

Acting Director, Division of Lands/ & Water Management

State Record of Water Right Certificates

Book XVI Page 4335, ADL 57850

ew 4-5-78

APPENDIX K

Water and Sewer Pump Information

Fir Pump

RAUL PERFORMANCE WARRANTY

GUARANTEED AT DESIGNATED POINT ONLY AND IS CONTINGENT ON

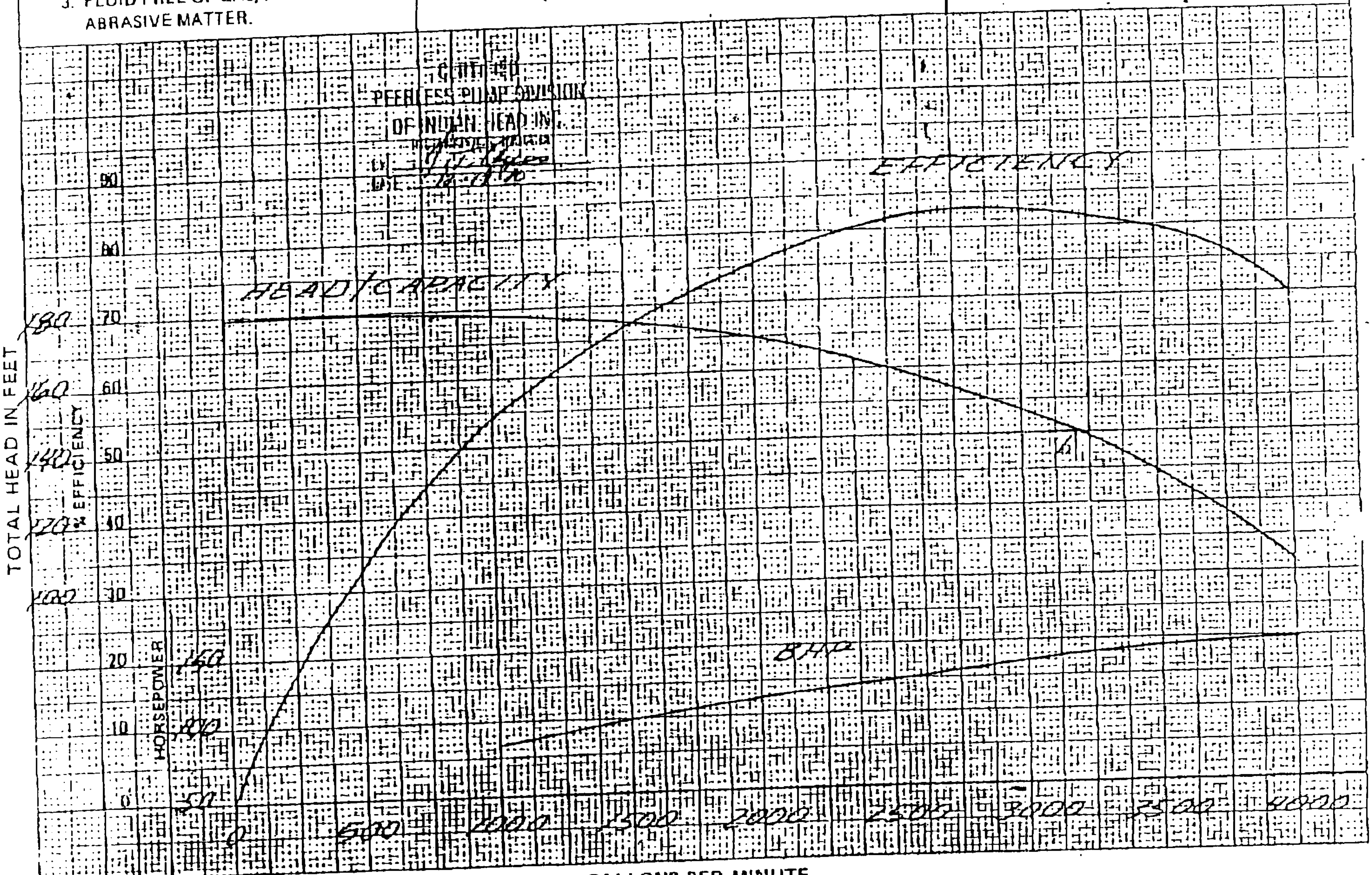
1. PROPER AND ADEQUATE FLOW TO PUMP SUCTION
2. ADEQUATE NPSH
3. FLUID FREE OF GAS, AIR AND ABRASIVE MATTER.

Peerless Pump

An Indian Head Company

2005 Northwestern Avenue Indianapolis, IN 46206
1200 Sycamore Street Montebello, CA 90640

CURVE NO. C15950
PUMP SIZE 8AD13.5 RPM. 50
IMPELLER 2672334 DIA 13.25
PUMP NO. 407713
S/O NO. XH 698978.



D. H. 15 9507 12-15-80

Pub 'orks Lift / Pump Station Data

	Campground Lift	Launch Ramp Lift	30 Acre Lift Station	Kachemak City Lift	Lakeshore Lift Sta.	Bay Ave Lift Sta.	Beluga Lift Sta.	STP Lift Station	Spit Fish Outfall
PW Equip #	L7ST	L8ST	L9ST	L6KK	**Removed**	L3BA	L2BL	L1SP	L5ST
Date On Line	1990	1990	1993	1990	**Oct-02**	1979	Oct-00	Apr-89	**1990/2000
Pump Make	Flygt	Flygt	Hydromatic	Flygt	**Panel, pumps etc.**	Hydromatic	Flygt	Hydromatic	Hydromatic
Pump HP	5hp	23hp	1hp	10hp	**Salvaged**	3hp	5hp	4@ 10hp ea	15hp
Serial No.	715BA-00033	832BA-00734	S14051	832BA-00729			BA-00733		3-3691/3692
Model No.	CP-3102.090	CP-SH-3152.091	S4NVX100CB	CP-3127090.090		S3HRC300M34-2	3102.09	S4LX	S6LX500JB
Impellar No.	432	268	6.4" diam	467		4" diam	432		11.25" diam
Ave GPM	125	235	120	150		190	400	variable 15-450	810
# ftTDH	43	145	9	87		12	31		35
Voltage	208 - 3ph	208 - 3ph	230 - 1ph	230/460 - 3ph		220/460-3ph	240 - 3ph	480 - 3ph	460-3ph
Amperage-full load	15	60	8.2	13/25		13	13	variable 5-21	21
RPM	1700	3500	1150	1750		3500	1715	variable 85-1150	1150
Pump 1	8940033	896073	S1405051	8930100			3102.090.9970174		3-3691
Man. Date P-1	Aug-88	Aug-88	1993			1992	2000		1987
Pump 2	8940034	896074	S1405052	8930099			3102.090.9970174		3-3692
Man. Date P-2	Aug-88	Aug-88	1993			1992	2000		1987
Pump 3 (spare)			S14051			133336	3102.090.0110082		S6LX1500JB
Pump 3 (spare) SN									S27082
Man. Date P-3	April-94	April-94	April-93			1992	2000		March-02
Sensaphone SN.	715BA-00033	832BA-00734	2161104-54999	832BA-00729		832BA-00735	832BA-00733		
Phone No.	235-2117	235-4318	235-3768	235-4316		235-7445	235-6188		235-2212
			Installed			New Control Panel	New Station w/ control		
			Sensaphone			October, 2000	panel,pumps,wet well		
			Jan-03				October, 2000		

** Note: Spit Outfall Station was installed in 1990. Activated in 2000.

Booster Pump

TYPICAL MAINTENANCE SUMMARY FORM

1. EQUIPMENT ITEM........ VARIABLE SPEED DRIVE (Booster Pump)

2. MANUFACTURER........ VS

3. EQUIPMENT IDENTIFICATION NUMBER(S)........ S/N 4526-5558

4. WEIGHT OF INDIVIDUAL COMPONENTS (OVER 100 POUNDS) ....

5. NAMEPLATE DATA (hp, voltage, speed, etc.)........ 1750 RPM

6. MANUFACTURER'S LOCAL REPRESENTATIVE

Name .... Telephone No........

Address........

7. MAINTENANCE REQUIREMENTS

Maintenance Operation	Frequency	Lubricant (If Applicable)	Comments
List briefly each maintenance operation req'd, & refer to specific information mfr's. std. maintenance manual, if applicable.	List required frequency of each maintenance operation.	Refer by symbol to lubricant list as required.	
See MAINTENANCE MANUAL			

8. LUBRICANT LIST

Reference Symbol	Shell	Standard Oil	Gulf	Arco	Or Equa (
List symbols used in Item 7, above.	List equivalent lubricants, as distributed by each manufacturer for the specific use recommended.				
SEE MAINTENANCE MANUAL					

9. SPARE PARTS. Include your recommendations regarding what spare parts, if any, should be kept on the job.

* * * * * *

Fire Pump

TYPICAL MAINTENANCE SUMMARY FORM

1. EQUIPMENT ITEM........ DIESEL ENGINE FIRE PUMP

2. MANUFACTURER........ PEERLESS

3. EQUIPMENT IDENTIFICATION NUMBER(S) MODEL 8AD 13 ½ S/N 407713

4. WEIGHT OF INDIVIDUAL COMPONENTS (OVER 100 POUNDS) ....

5. NAMEPLATE DATA (hp, voltage, speed, etc.)........ 1750 RPM, HP 214 @ 2400 RPM

6. MANUFACTURER'S LOCAL REPRESENTATIVE

   Name .... STANG HYDRONICS .... Telephone No. (206) 641-3867

   Address........ 13251 Northrup Way, Bellevue, Wa. 98005

7. MAINTENANCE REQUIREMENTS

Maintenance Operation	Frequency	Lubricant (If Applicable)	Comments
List briefly each maintenance operation req'd. & refer to specific information in mfr's. std. maintenance manual, if applicable.	List required frequency of each maintenance operation.	Refer by symbol to lubricant list as required.	
See Maintenance Manual 2880549			

8. LUBRICANT LIST

Reference Symbol	Shell	Standard Oil	Gulf	Arco	Or Equal (
List symbols used in Item 7, above.	List equivalent lubricants, as distributed by each manufacturer for the specific use recommended.				
See Maintenance Manual 2880549					

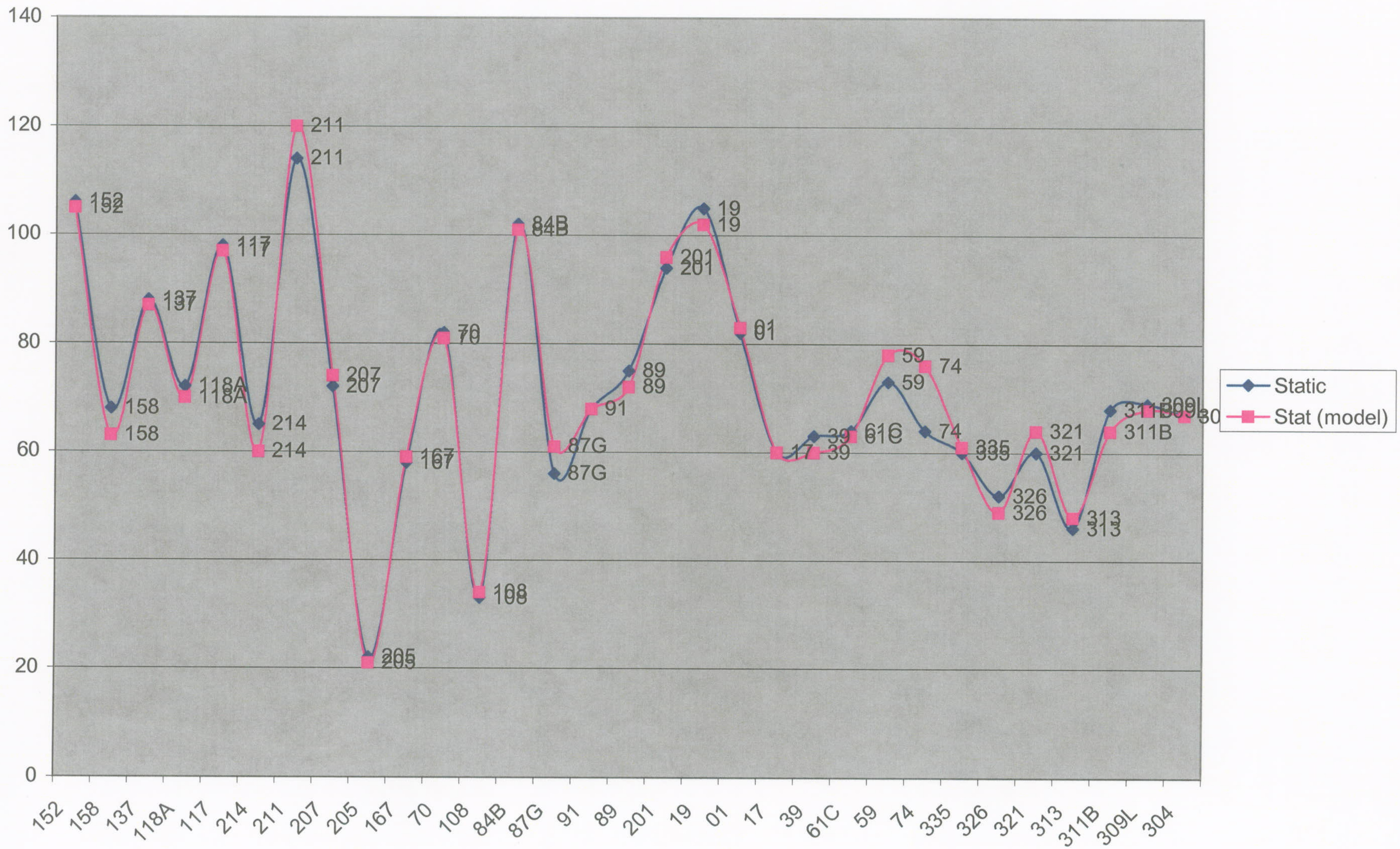
9. SPARE PARTS. Include your recommendations regarding what spare parts, if any, should be kept on the job.

* * * * * *

APPENDIX L

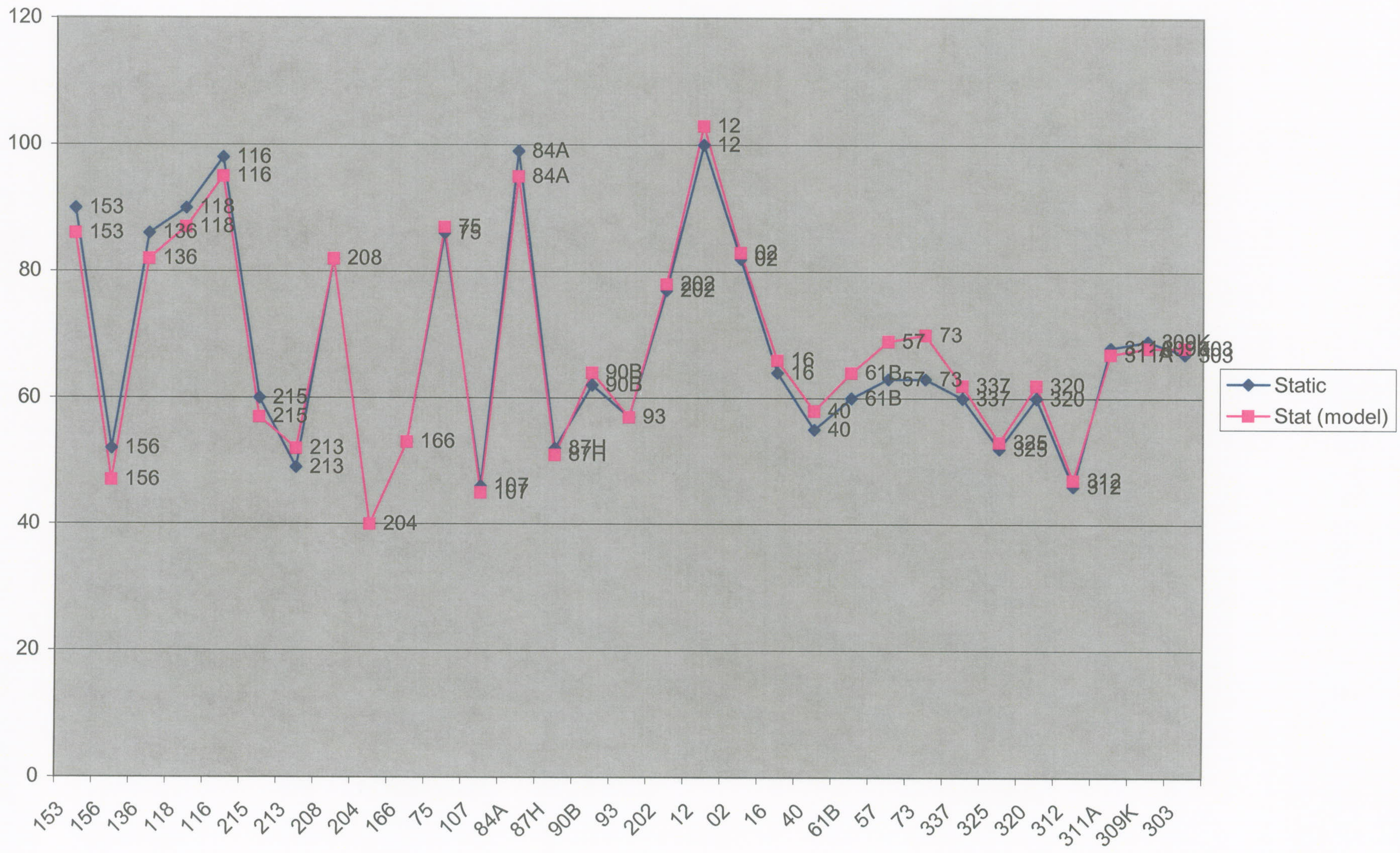
Water Modeling Calibration

Static Flow Hydrant



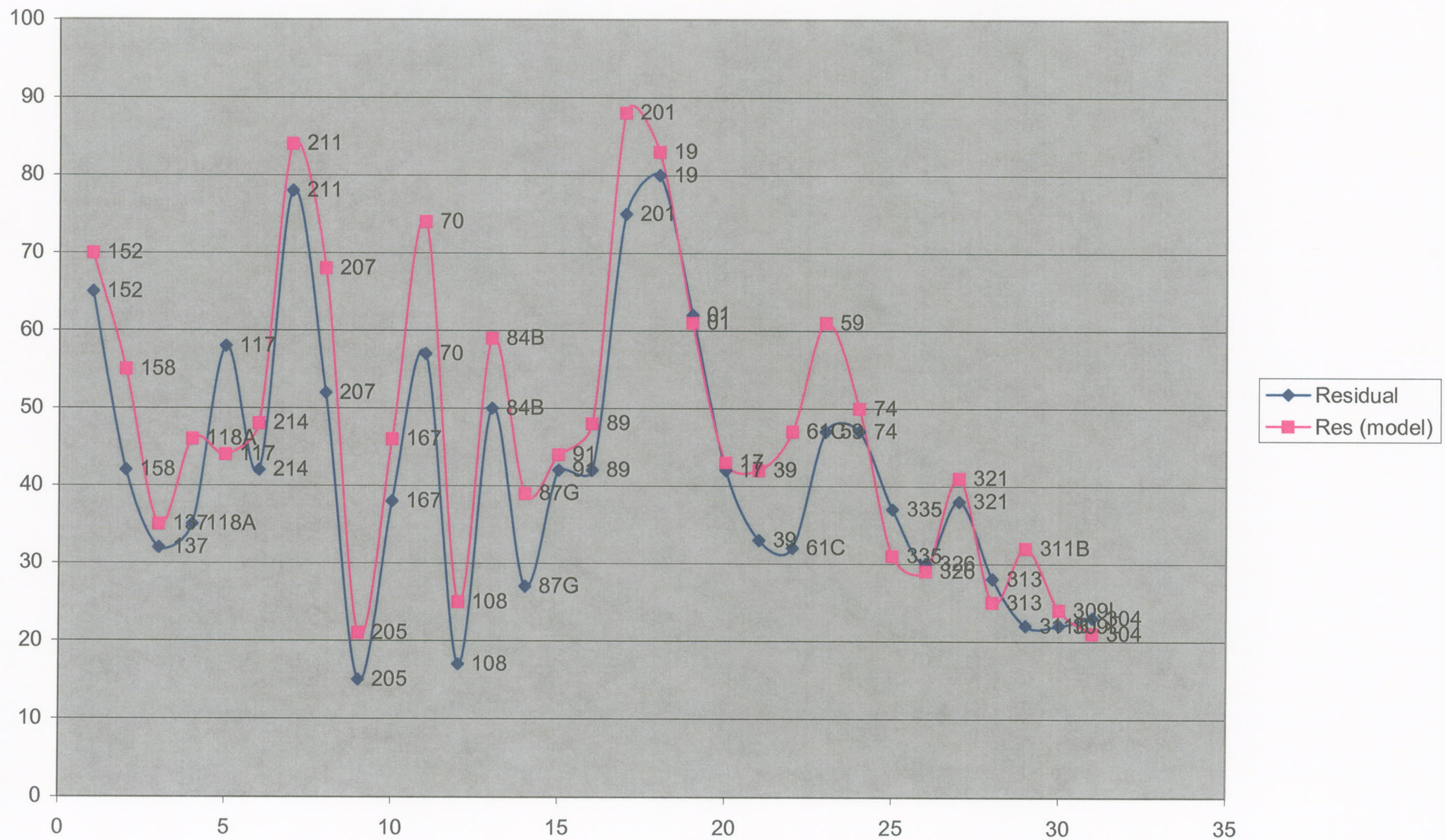
calibration1.xlsSFH

Static Residual Hydrant



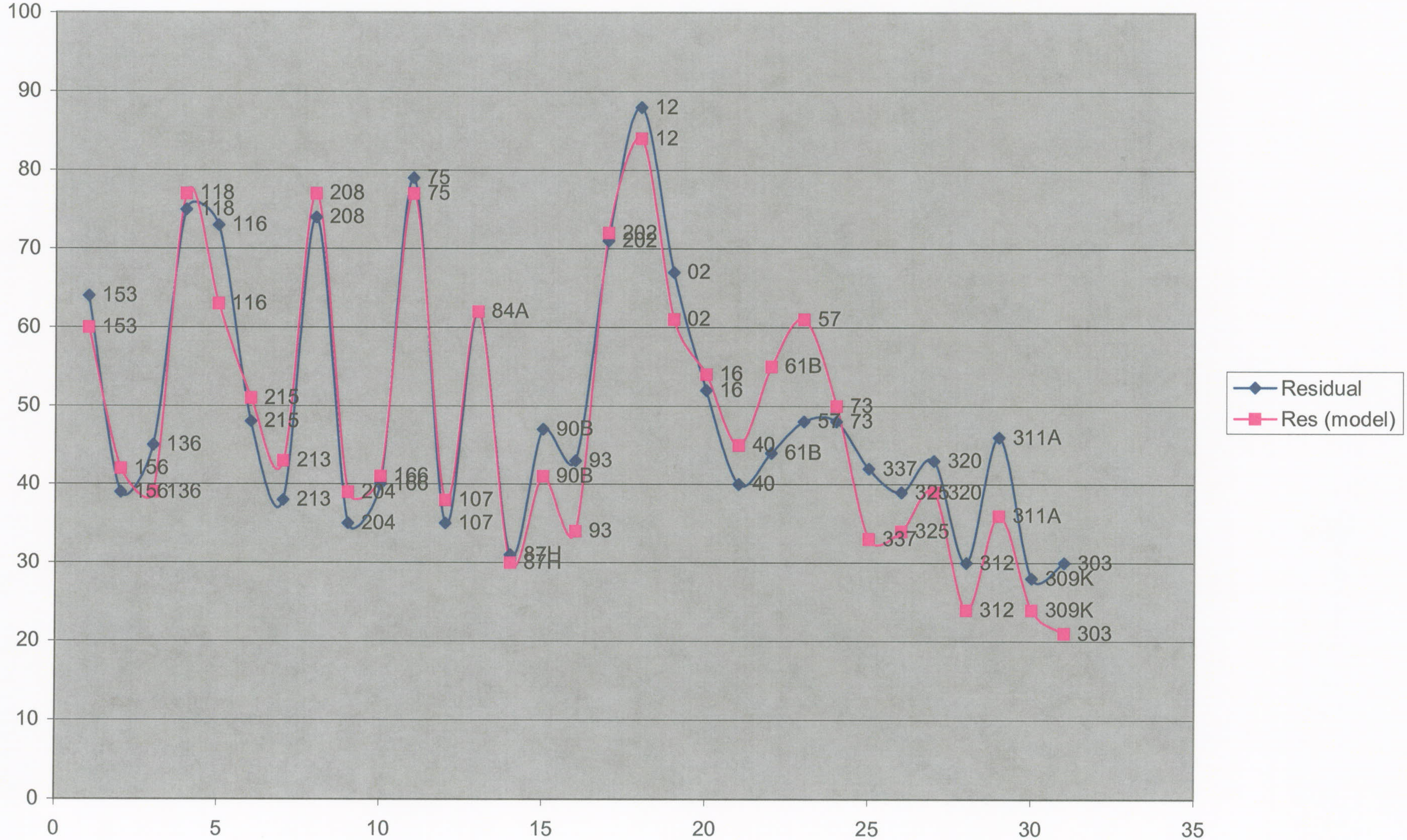
calibration1.xlsSRH

Flow Hydrant Residual Pressure - Calibration 20



calibration1.xlsFc20

Residual Hydrant Residual Pressure - Caibration 20



calibration1.xlsRc20

PRV	Fire Flow FH	GPM	Residual	Res (model)	Difference	Residual FH	Residual	Res (model)	Difference		Diff (Flow H)	Diff (Model H)
MH	152	1360	65	70	5	153	64	60	4	CI/DI	5	-4
AF	158	1090	42	55	13	156	39	42	3	CI/DI	13	3
AF	137	950	32	35	3	136	45	39	6	PVC	3	-6
AF	118A	1010	35	46	11	118	75	77	2	HDPE	11	2
AF	117	1280	58	44	14	116	73	63	10	DI	-14	-10
EF	214	1090	42	48	6	215	48	51	3	DI	6	3
SB	211	1480	78	84	6	213	38	43	5	DI	6	5
BT	207	1210	52	68	16	208	74	77	3	DI	16	3
MD, KK, HL	205	650	15	21	6	204	35	39	4	DI	6	4
MD, KK, HL	167	1040	38	46	8	166	40	41	1	DI	8	1
MD, KK, HL	70	1280	57	74	17	75	79	77	2	CI/PVC	17	-2
MD, KK, HL	108	690	17	25	8	107	35	38	3	PVC	8	3
MD, KK, HL	84B	1190	50	59	9	84A	62	62	0	DI	9	0
MD, KK, HL	87G	890	27	39	12	87H	31	30	1	HDPE	12	-1
MD, KK, HL	91	1090	42	44	2	90B	47	41	6	DI	2	-6
MD, KK, HL	89	1090	42	48	6	93	43	34	9	DI	6	-9
MD, KK, HL	201	1465	75	88	13	202	71	72	1	DI/CI	13	1
MD, KK, HL	19	1500	80	83	3	12	88	84	4	DI/PVC	3	-4
MD, KK, HL	01	1320	62	61	1	02	67	61	6	PVC	-1	-6
BC	17	1090	42	43	1	16	52	54	2	DI/PCV	1	2
LK	39	980	33	42	9	40	40	45	5	DI	9	5
LS	61C	950	32	47	15	61B	44	55	11	DI	15	11
LS	59	1160	47	61	14	57	48	61	13	DI	14	13
BG, JS	74	1160	47	50	3	73	48	50	2	CI	3	2
HE	335	1040	37	31	6	337	42	33	9		-6	-9
HE	326	920	30	29	1	325	39	34	5	CI	-1	-5
HE	321	1040	38	41	3	320	43	39	4		3	-4
HE	313	890	28	25	3	312	30	24	6	PVC	-3	-6
HE	311B	790	22	32	10	311A	46	36	10	CI	10	-10
HE	309L	790	22	24	2	309K	28	24	4	HDPE	2	-4
HE	304	820	23	21	2	303	30	21	9	HDPE	-2	-9

C: PVC=155, HDPE=185, DI=101, CI=99.9	**Sum differences**	**228**	**Sum differences**	**153**
	Average	7		
	Average	5	**TOTAL**	**381**

HDPE	5.8	-3.0	**1.4**
PVC	4.8	-2.2	**0.6**
CI	8.6	-2.1	**3.2**
DI	6.9	1.1	**4.2**

Supply Lines are connected

Homer Water and Sewer Master Plan - Final
July 2006
BEESC Project No. 24047

Prepared for:

Prepared by:

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