City of Albion

Wastewater Facilities Plan

March 2013

Prepared by

J·U·B ENGINEERS, INC.

J-U-B ENGINEERS, Inc.
115 Northstar Avenue
Twin Falls, Idaho 83301
208-733-2414
Project No. 60-11-041

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PROFESSIONAL ENGINEER
REGISTERED
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3-4-13
STATE OF IDAHO
MARK HOLTZEN

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Introduction

1.0 INTRODUCTION

1.1 PURPOSE AND NEED OF FACILITIES PLAN

The City of Albion owns and operates a municipal wastewater collection and treatment system that serves the area in and around the City. They have concerns regarding the age, condition, and capacity of their wastewater infrastructure, including:

- The collection system is comprised primarily of aging, asbestos cement pipe installed in 1975. Leaky and/or cracked service connections, pipe, and manholes has resulted in infiltration and inflow (I&I) flows that are approximately twice as much as would be expected for a city the size of Albion.

- The pumps within the existing sewer lift station are undersized for pumping the influent flow to the treatment lagoons, particularly during high I&I flow periods. As a result, the wet-well has overflowed on several occasions in the past. Additionally, there is no back-up power or flow monitoring and the lift station is over 35 years old, resulting in many components that are corroded and in need of replacement. The lift station pumps also have occasional problems with ragging and clogging.

- The wastewater treatment lagoons and land application system are generally in adequate condition and have sufficient capacity. However, there are several minor improvements needed to optimize their performance (e.g., inlet and outlet valve replacement, influent screening, etc.).

As a result of these concerns, the City authorized J-U-B ENGINEERS, Inc. (J-U-B) to prepare a Wastewater Facilities Plan to analyze the existing wastewater system and to investigate potential improvement alternatives to address their current and future community needs and regulatory requirements. The report will provide the City with the necessary information to make decisions in the future regarding their wastewater collection and treatment system. The Facilities Plan is prepared in accordance with Idaho Department of Environmental Quality (IDEQ) Water Pollution Control State Revolving Loan Fund (SRF) program requirements and with State regulations (IDAPA 58.01.16.410).

1.2 REPORT ORGANIZATION

The report is organized into six chapters, including:

- Chapter 1 - Introduction
- Chapter 2 – Existing Conditions
- Chapter 3 – Future Conditions
- Chapter 4 – Evaluation of Existing Facilities
- Chapter 5 – Development and Screening of Improvement Alternatives
- Chapter 6 – Implementation of Wastewater System Improvements

A further breakdown on the organization of the Facilities Plan is provided in the Table of Contents, Appendices, List of Tables and List of Figures.

1.3 OPINIONS OF PROBABLE COSTS

The cost opinions contained in this report are in 2012 dollars that have been inflated for two years to account for typical delays in obtaining funding and design prior bidding. The costs may need to be inflated further if implementation of the recommended improvements is delayed beyond two years. It should be noted that the costs of most construction materials have been very volatile over the past few years. This volatility in the market and pricing of construction materials makes it difficult to predict what the actual construction costs will be in future dollars.

1.4 ENVIRONMENTAL REVIEW

An Environmental Information Document (EID) will be prepared separately from this report for the specific improvements identified in the Wastewater Facilities Plan. The EID will evaluate potential environmental impacts and mitigation measures for the proposed improvements.

Chapter 2

Existing Conditions

2.0 EXISTING CONDITIONS

2.1 PLANNING AREA

The City of Albion is located in south central Idaho in the north central section of Cassia County (see **Figure 1**). The City falls within Section 6 of Township 12 South, Range 25 East, B.M. It is situated approximately 8 miles southeast of the City of Declo and approximately 17 miles northwest of the City of Malta. The City is located along State Highway 77 in a predominantly agricultural region.

This Wastewater Facilities Plan is based on a specific Planning Area which represents a geographical area and population that the City can reasonably be expected to serve within a 20-year design period from 2012 to 2032. **Figure 2** shows the Planning Area and existing corporate city limits for Albion.

A number of factors were considered in delineating the geographical boundary of the Planning Area including recent developmental patterns, the location of existing wastewater system facilities, expandability of the existing wastewater system, land use designations, topography of the area, and discussions with City staff regarding areas of anticipated growth. Sufficient land is included in the Planning Area to accommodate the forecasted residential and commercial growth, and to allow some flexibility for future development of the community.

2.2 EXISTING PLANNING AREA CONDITIONS

2.2.1 Physiography, Topography, Geology, and Soils

The topography of the Albion planning area is depicted on the U.S. Geological Survey (USGS) topographic map shown in **Figure 3**. As shown on the map, the general configuration of the regional topography is that of a large bowl with an outlet at the north end. The ground surface elevation across the planning area ranges from a low of approximately 4,690 to 4,800 feet above mean sea level. There is a small hill which bounds the north side of the City with a peak elevation of approximately 4,990 feet above mean sea level. The area specific to the treatment lagoons and land application site consists of relatively flat land with a gradual slope to the north.

The regional geology in the vicinity of Albion is characterized by steeply sloping mountain ranges and intervening wide, open valleys. The mountain ranges in the area are primarily composed of rhyolite, mica, schist, quartzite and granodiorite (*Soil Survey of Cassia County, Idaho, Eastern Part, NRCS, 1987*). The intervening valleys include alluvium deposits derived from the surrounding mountains, with later additions of loess and silty alluvium (*Soil Survey of Cassia County, Idaho, Eastern Part, NRCS, 1987*). The City's wastewater facilities are located within one of the intervening valley areas. Soils in the vicinity of the treatment site are very deep, with a depth to bedrock generally greater than 5 feet.

A Natural Resource Conservation Service (NRCS) soil survey map of the planning area is shown in **Figure 4**. **Table 1** summarizes various characteristics of the predominant soil types in the area. Soil types near the treatment lagoons and land application site include Chatburn silt loam (25), Downata silt loam (56) and Ririe silt loam (119). These soil types are primarily suitable for non-irrigated or irrigated cropland, hayland, and pasture.

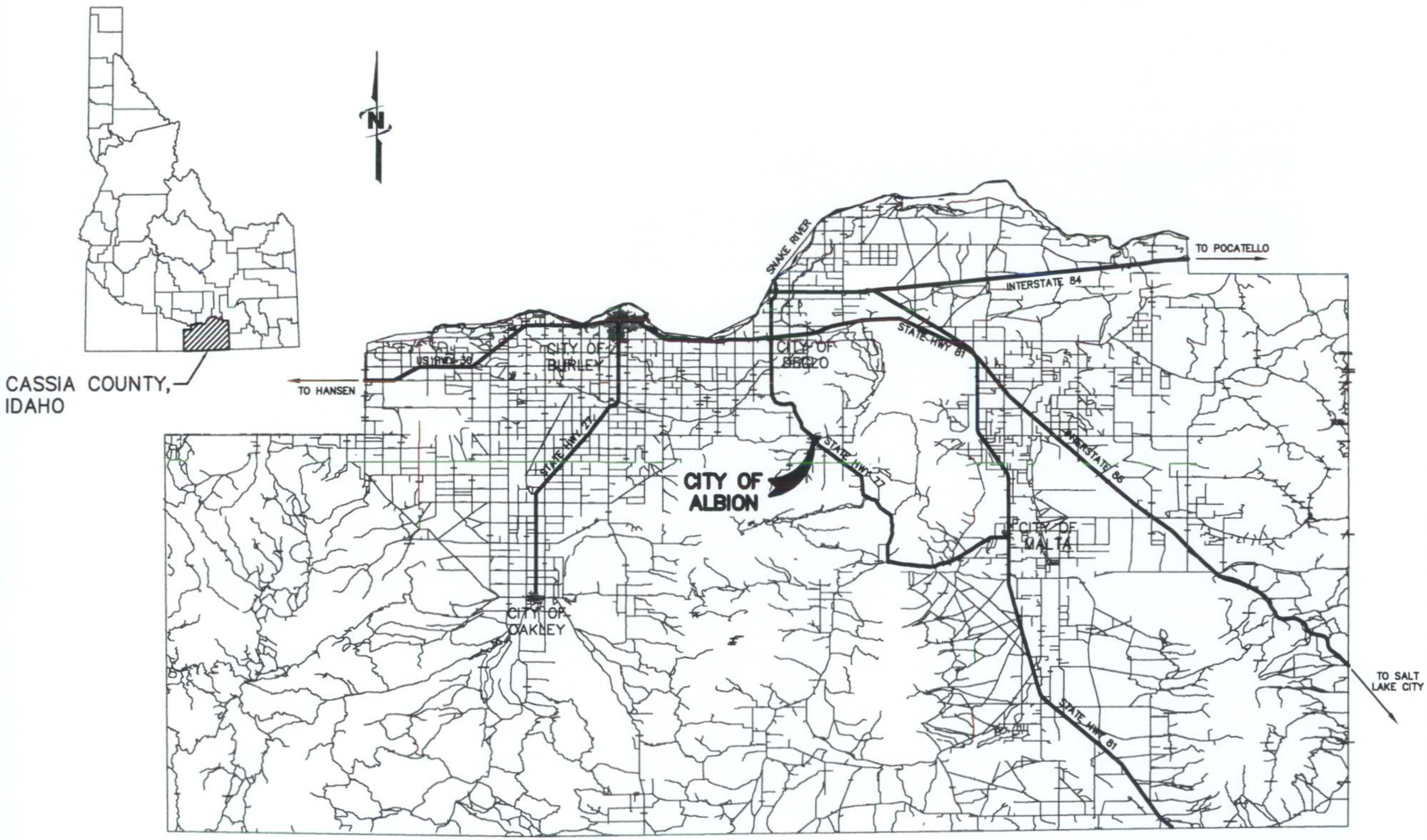


FIGURE 1
CITY OF ALBION
VICINITY MAP

J-U-B ENGINEERS, INC.

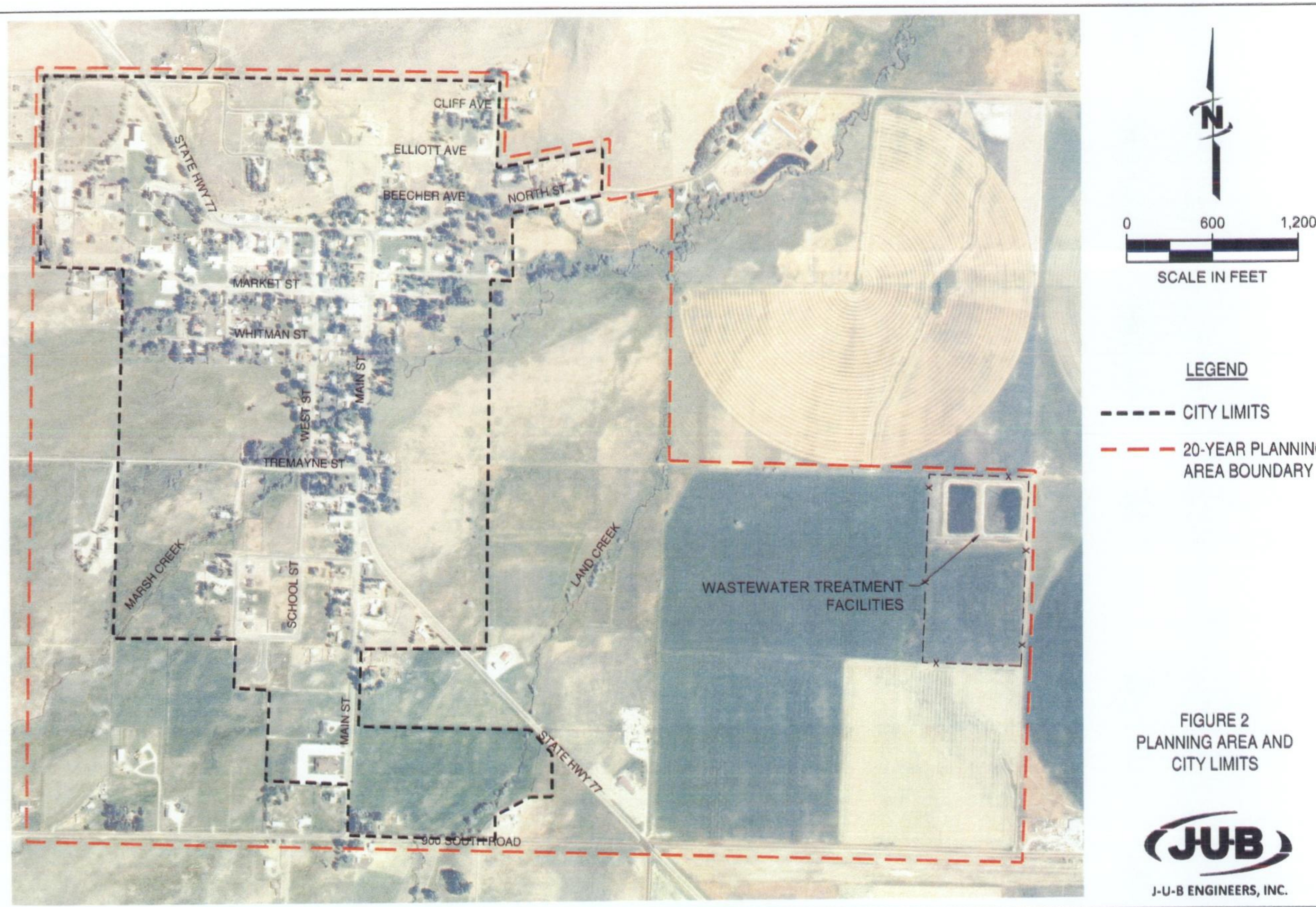


FIGURE 2
PLANNING AREA AND
CITY LIMITS

J-U-B ENGINEERS, INC.

Figure 3 – USGS Topographic Map

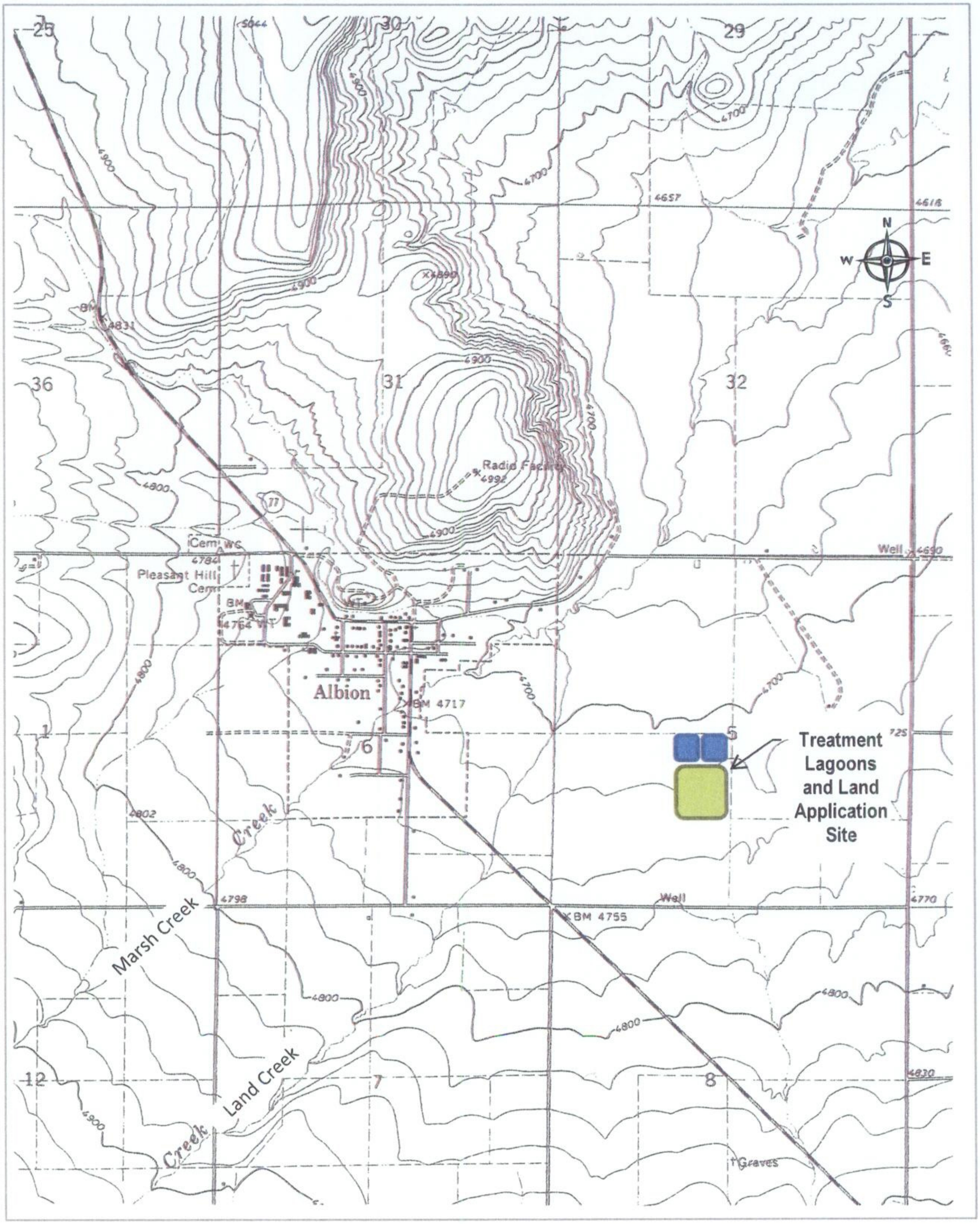


Figure 4 – NRCS Soil Survey Map

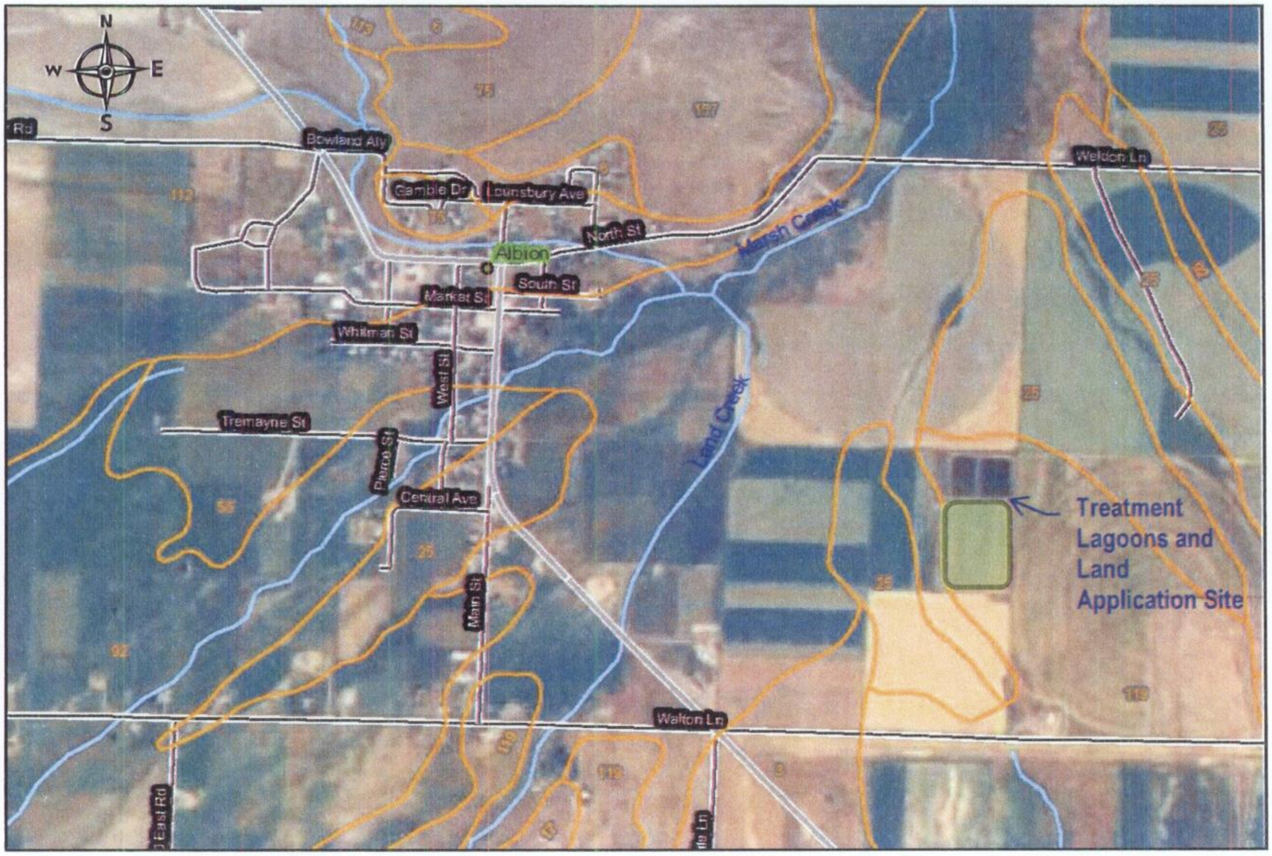


Table 1 – NRCS Soil Characteristics

Soil Map Unit	Description	Slope	Depth Class	Drainage Class	Permeability	Available Water Capacity	Potential Rooting Depth	Runoff
3	Acord Silt Loam	2-4%	Very Deep	Well Drained	Slow	7–8 in.	20-25 in.	Slow
6	Arbone Loam	4–12%	Very Deep	Well Drained	Moderate	9–11 in.	>60 in.	Medium
17	Bezzant Gravelly Loam	2–12%	Very Deep	Well Drained	Moderate	5–6 in.	>60 in.	Medium or Rapid
25	Chatburn Silt Loam	1–4%	Very Deep	Well Drained	Moderately Slow	8–10 in.	15–20 in.	Slow
56	Downata Silt Loam	0–2%	Very Deep	Very Poor	Moderately Slow	10–12 in.	10–45 in.	Very Slow
75	Hutchley Very Gravelly Silt Loam	10–35%	Shallow	Well Drained	Moderately Slow	1–2 in.	10–20 in.	Moderate to Very Rapid
92	Kovich Silt Loam	0- 3%	Very Deep	Poorly Drained	Moderate	7–8 in.	12–55 in.	Slow
112	Rexburg Silt Loam	1–3%	Very Deep	Well Drained	Moderate	11–13 in.	>60 in.	Slow
113	Rexburg Silt Loam	3-12%	Very Deep	Well Drained	Moderate	11–13 in.	>60 in.	Medium or Rapid
119	Ririe Silt Loam	1–3%	Very Deep	Well Drained	Moderate	11–13 in.	>60 in.	Slow

2.2.2 Surface and Groundwater Hydrology

The primary surface water sources within the planning area are Land Creek, Marsh Creek, and minor irrigation laterals (see **Figures 3 and 4**). The two creeks originate in the Albion Range and flow separately through the southern portion of the planning area before joining together east of the City. Surface water generally flows to the northeast through the area.

Most irrigation and drinking water in the planning area comes from wells. The City reports that there are no springs or drinking water wells located within one-quarter mile of the treatment lagoons and land application site. There is an irrigation well situated approximately one-quarter to one-half of a mile south (upgradient) of the land application site. Albion's three existing municipal drinking water wells are located within the City limits near residential areas.

The source of groundwater in the Albion area is the Albion Basin aquifer. An unconfined, shallow aquifer is contained within the upper alluvium deposits beneath the valley floor. Deeper confined artesian aquifers are also located within the basin. This basin is generally enclosed on the east by the Cotterell (Malta) Range, on the south and west by the Albion Range, and on the northwest by the East Hills. The primary source of groundwater is from precipitation that falls on the Albion Range to the south. Groundwater is also recharged to a lesser extent by infiltration from natural streams and irrigation practices in the area. The groundwater generally travels northward and discharges through springs in Howell Creek and Marsh Creek and as underflow beneath the valley.

Groundwater levels in the area generally fluctuate seasonally from near the surface to approximately 5 to 45 feet below the ground surface. This information is based on historical monitoring data from a USGS monitoring well (USGS 422405113343801 12S 25E 06DCC1) located adjacent to the City in Section 6, Township 12 South, Range 25 East, B.M., and on data from the City's 1975 treatment plant Operation and Maintenance Manual. Regional groundwater flow direction is generally north to northwest.

2.2.3 Fauna, Flora, and Natural Communities

Due to the mountainous topography of the area, there is wide range of plants and animals. Vegetative habitats common to the area include pinyon-juniper woodlands, aspen-riparian communities, sagebrush steppe, mountain mahogany woodlands, and high elevation meadows.

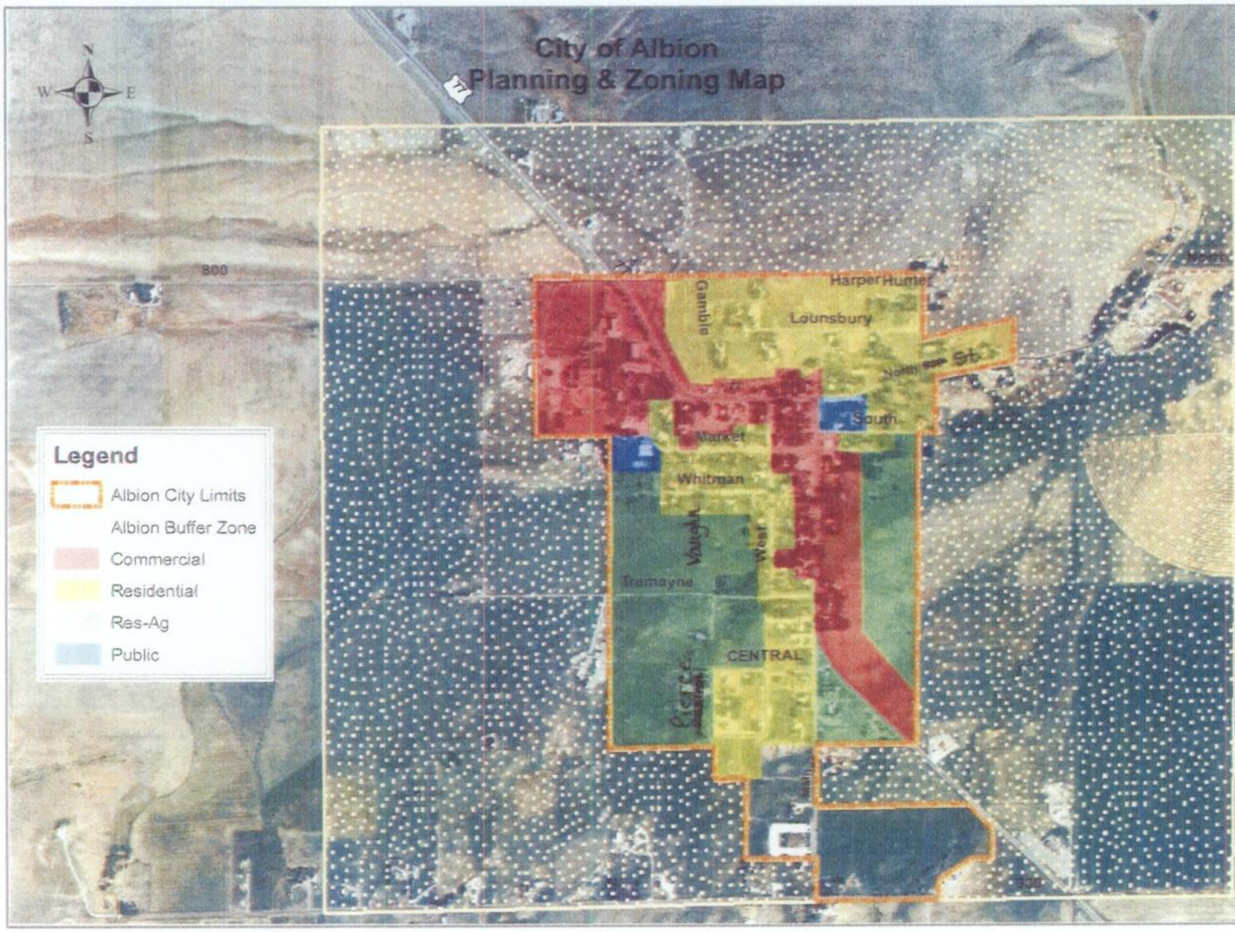
Migratory wildlife, many of which are avian species, seasonally pass through the area. Common upland game birds in the area include pheasants, partridge, quail and sage grouse. Raptors such as hawks, eagles and owls are also found in the area. Animals commonly found in the area include mountain lions, bobcat, coyote, squirrels, mule deer, rabbits, marmot, chipmunk, fox, skunks and coyote. The surrounding area is popular during hunting season for deer, elk, and other game. Fish common to the area include trout.

Species listed under the Endangered Species Act for Cassia County include the Utah Valvata Snail, Snake River Physa Snail, Christ's Indian Paintbrush, and Goose Creek Milkvetch.

2.2.4 Housing, Industrial, and Commercial Development

Figure 5 shows a current planning and zoning map of the City and Area of Impact, depicting the generalized land use designations. As shown in the figure, land use within the planning area is predominantly residential and residential-agricultural, with smaller areas of light commercial development. There is no "wet" commercial/industrial development or industrial zoning in the City.

Figure 5 – Planning and Zoning Map



Residential areas in Albion are located throughout the planning area. Commercially zoned areas are primarily located along Highway 77 through town, Market Street, and the old college campus which has been turned into a hotel and retreat center. Areas zoned for public use are the elementary school and city park.

Some of the businesses located within the area include restaurants, grocery stores, service stations, veterinarians, financial institutions, retreat centers, museums, construction companies, convenience stores, and hotels. A year-round resort and ski area is located approximately 12 miles south of Albion.

There are no inhabited private residential dwellings or public dwellings located within one-quarter mile of the treatment lagoons and land application site. There is an uninhabited airport hangar located approximately one-quarter of a mile southeast of the treatment facilities.

The area surrounding the City and planning area is predominantly used for agricultural purposes. The fertile soils allow for the production of a wide variety of crops, including wheat, barley, alfalfa, beans, potatoes, and sugar beets.

2.2.5 Cultural Resources

Native Americans from numerous tribes hunted in the Albion area but they did not establish any permanent settlements. In the early 1800s, trappers entered the valley. In the 1860s and 1870s prospectors arrived searching for gold and silver in the nearby mountains. The Oregon and California trails as well as the transcontinental railroad passed near or through the Albion Valley. Albion City was platted in 1880 and was the largest city in the region. In 1893, the Albion State Normal School was constructed. This college trained thousands of Idaho school teachers. In 1951, the school was closed and its programs were transferred to Idaho State University in Pocatello. The college campus fell into disrepair but it was recently purchased, renovated, and portions of it turned into a retreat center.

The Historic Preservation Office (HPO) of the Idaho State Historical Society was consulted regarding cultural resources in Albion. The following buildings are listed on the HPO's National Register of Historic Places in Idaho for the City: Albion Methodist Church, Albion Normal School Campus, and Swanger Hall.

2.2.6 Public Utilities and Services

The City is serviced by a full complement of public utilities and services. These services are intended for permanent and seasonal residents of Albion, and the immediate surrounding county residents. Some of the public utilities and services offered within the area include:

- Water and Sewer
- Fire and Police Protection
- Communication Systems
- Post Office
- Power/Electric Services
- Government Services
- Public Library
- Cable Television
- Solid Waste Disposal
- Public Schools
- Meeting and Lodging Facilities
- Transportation Services
- Recreational Facilities

2.2.7 Flood Plains and Wetlands

A Federal Emergency Management Agency (FEMA) flood zone map (Community Panel Number 160041 0011 A) was reviewed to examine flood plains in the planning area (see **Figure 6**). As shown in the figure, areas designated as 100-year flood zones (Zone A) are primarily located adjacent to Land Creek, Marsh Creek, and Howell Creek. The wastewater treatment facilities do not appear to be located within any 100 year flood plains. The FEMA flood zone map does not address 25 and 50 year flood plains.

The U.S. Fish and Wildlife Service's National Wetlands Inventory provides mapping of wetlands across the United States. The basic criteria that define wetland types are water depth and permanence, water chemistry, life form of vegetation and dominant plant species. As shown in **Figure 7**, the predominant types of wetlands in the planning area include palustrine unconsolidated bottom (PUB), palustrine aquatic bed (PAB), palustrine emergent (PEM), palustrine scrub-shrub (PSS), riverine streambed (RSB), and riverine unconsolidated bottom (RUB) wetlands.

2.2.8 Wild and Scenic Rivers

The Wild and Scenic Rivers Act, as promulgated by Congress on October 2, 1968, states that "...certain selected rivers of the Nation which, with their immediate environments, possess outstandingly remarkable scenic, recreational, geological, fish and wildlife, historical, cultural, or other similar values, shall be protected for the benefit and enjoyment of present and future generations."

Figure 6 – FEMA Flood Zone Map

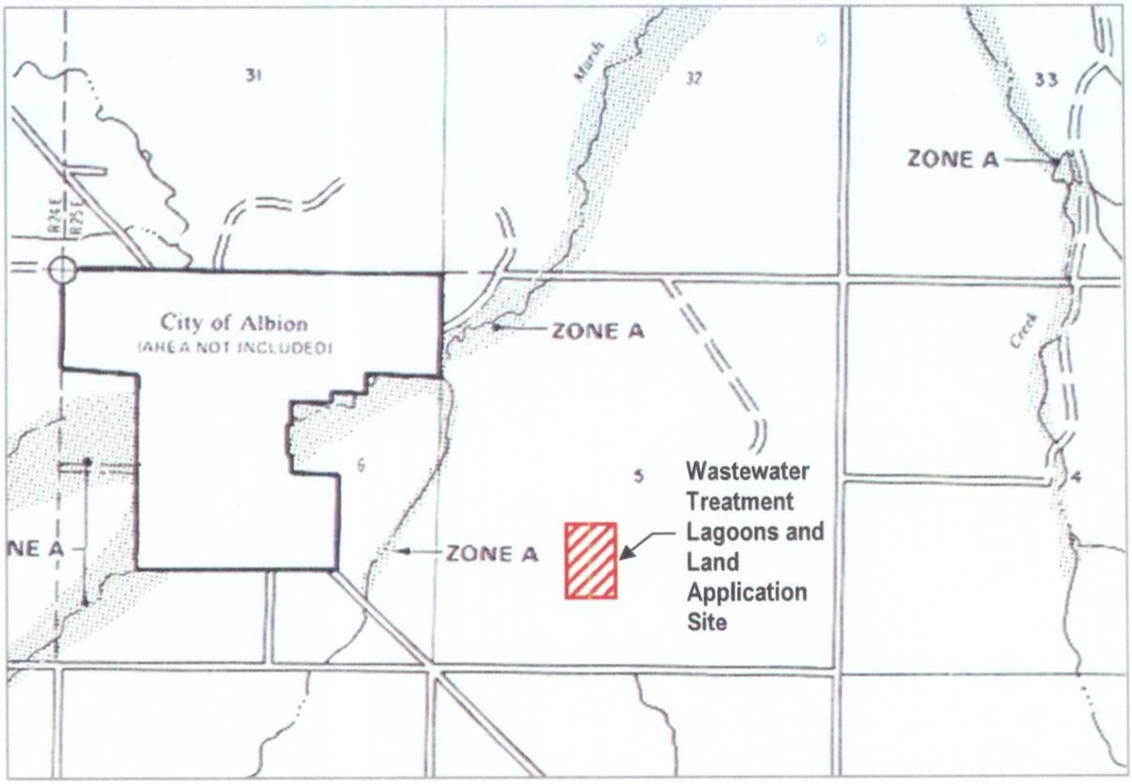
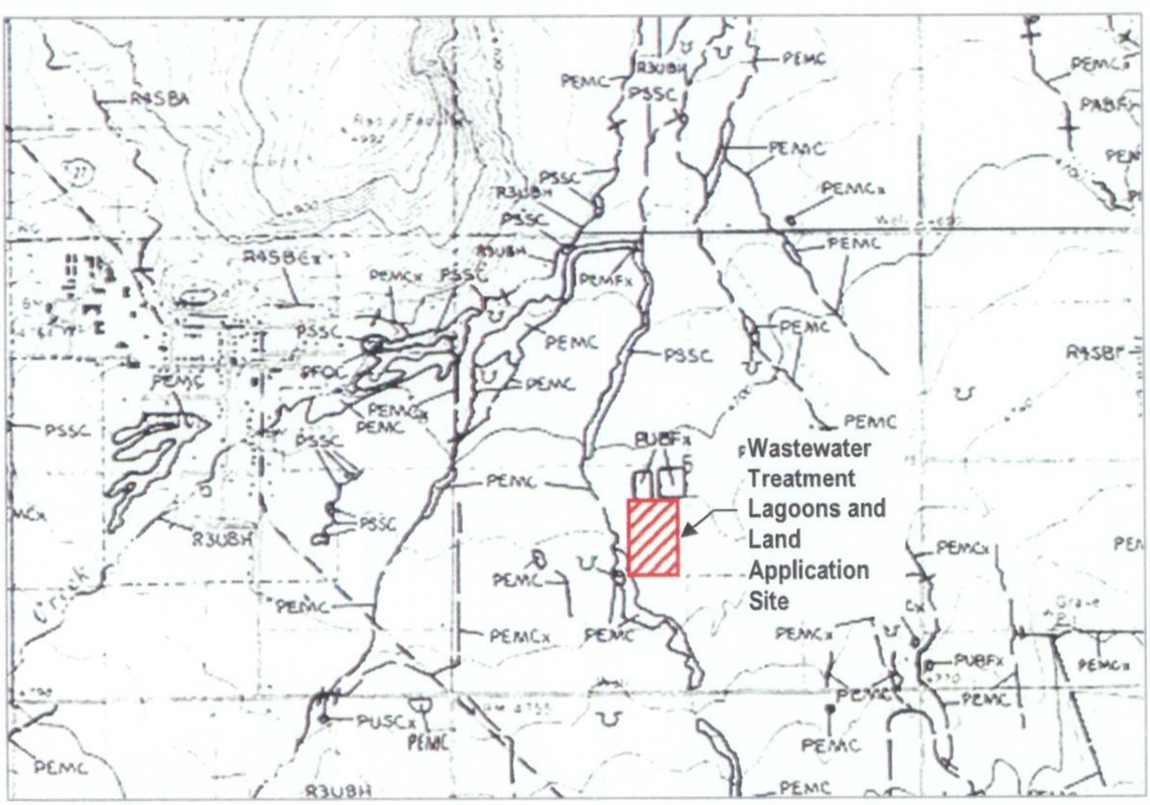


Figure 7 – US Fish and Wildlife Wetlands Map



All or portions of the following rivers in Idaho have been designated as Wild and Scenic Rivers:

- Battle Creek
- Big Jacks Creek
- Bruneau River
- Clearwater River (Middle Fork)
- Cottonwood Creek
- Deep Creek
- Dickshooter Creek
- Duncan Creek
- Jarbridge River
- Little Jacks Creek
- Owyhee River (North and South Fork)
- Rapid River
- Red Canyon
- Salmon River
- Sheep Creek
- Snake River (Hells Canyon)
- Selway River
- St. Joe River
- Wickahoney Creek

No surface water sources near the Albion area are classified as "Wild and Scenic".

2.2.9 Public Health Considerations

In general, there are minimal public health issues related to the wastewater system in the planning area. However, since the aquifer in the area is relatively small and shallow, there is some concern that lift station overflows, leaks from damaged sewer lines, lagoon seepage, and/or improper land application of effluent could potentially contaminate the drinking water supply for the City and other users in the valley.

2.2.10 Prime Agricultural Lands

As defined by the 1978 EPA Policy to Protect Environmentally Significant Agricultural Lands, prime farmland has the "...best combination of physical and chemical characteristics for producing food, feed, forage, fiber and oilseed crops, and is available for these uses". According to the NRCS, soils within the Albion area considered "prime or unique" farmland include Chatburn silt loam (25), Rexburg silt loam (112) and Ririe silt loam (119) (see **Figure 4**).

2.2.11 Proximity to a Sole Source Aquifer

The Sole Source Aquifer program was established under Section 1424(e) of the Safe Drinking Water Act of 1974. The program allows individuals and organizations to petition the EPA to designate aquifers as the "sole or principal" source of drinking water for an area. To meet the sole source criteria, an aquifer must supply at least 50 percent of the drinking water consumed in the area overlying the aquifer. The EPA guidelines also stipulate that these areas can have no alternative drinking water source(s) that could physically, legally and economically supply all those who depend upon the aquifer for drinking water. At this time, the Albion Basin aquifer is not considered to be a sole source aquifer. The nearest sole source aquifer to Albion is the Eastern Snake River Plain Aquifer.

2.2.12 Precipitation, Temperature, and Prevailing Winds

Albion has a semi-arid climate typical of southern Idaho; although, due to its higher elevation it tends to receive more precipitation than surrounding areas. **Table 2** summarizes historical temperature, precipitation and evaporation data for Albion.

Table 2 – Monthly Climatic Data

Month	Mean Temperature A (°F)	Mean Precipitation A (in)	Average Snowfall A (in)	Average Evaporation B (in)
January	27.7	0.92	7.1	0.2
February	32.0	0.73	4.3	0.6
March	39.1	0.90	3.4	1.6
April	46.4	1.18	1.5	3.2
May	53.7	1.54	0.3	5.6
June	61.1	1.12	0.0	6.0
July	69.2	0.72	0.0	6.8
August	68.2	0.68	0.0	6.4
September	58.9	0.77	0.0	4.0
October	48.7	0.89	0.4	2.4
November	37.2	0.89	3.1	2.0
December	29.0	0.84	5.2	1.2
Annual	**47.6**	**11.2**	**25.3**	**40.0**

A. Values in the table are an average from four independent weather monitoring stations near the planning area: Albion College of Education (1899-1953), Malta 2E (1963-2002), Malta Aviation (1984-2010), and Oakley (1893-2012). The data was obtained from the Western Regional Climatic Center (www.wrcc.dri.edu/ summary/climsmid.html).

B. Calculated based on "Monthly Shallow Pond Evaporation in Idaho", 1992, Lolnau, Kpordze and Craine, ASAE Paper PNW 92-111 (Region 3, snow.cals.uidaho.edu/publications/Pond_evap/evap_fws.gif).

Winter is characterized by alternating high and low pressure systems that bring associated inclement or clear conditions. January is historically the coldest month with an average temperature of approximately 27.7°F. Most of the annual precipitation in the area falls as rain and snow during the winter and spring. Summer weather is normally dry with warm to hot temperatures. July is historically the warmest month with an average temperature of 69.2°F. The warm summer temperatures combined with low relative humidity produce an annual evaporation rate of approximately 40.0 inches. The prevailing wind direction in the region is from the west to southwest with an average wind speed of approximately 5 to 10 mph. Tornadoes and funnel clouds are rare, as are destructive force winds.

According to IDEQ's Guidance for Reclamation and Reuse of Municipal and Industrial Wastewater, Albion appears to be located within Irrigation Climatic Area II. As a result, the number of frost free days for the growing season ranges from approximately 100 to 120 days.

2.2.13 Air Quality and Noise

EPA has developed standards for monitoring and protecting air quality. IDEQ is responsible for implementing, monitoring and enforcing the air quality standards within Idaho. An area that exceeds the air quality standards is considered to be a "non-attainment area" (NAA) for a particular component, or total air quality. There are several NAA's in Idaho, with the closest being the Fort Hall, Cache Valley, and Portneuf Valley NAAs. As such, the Albion planning area is currently not located within a NAA.

Residents in Albion generally feel that air quality is excellent and cite this amenity as one of the area's quality of life factors. Albion is well removed from any major urbanized areas and there are very few sources of pollution in the immediate vicinity. Local automobile emissions and agricultural activities are the primary contributors to air quality degradation. High levels of particulate matter may be experienced during certain weather events or during certain times of the agricultural season due to farming practices.

Noise in the Albion area is generally limited to normal traffic, commercial activities, and agricultural practices.

2.2.14 Energy Consumption and Production

A majority of the population in the planning area consumes energy in the form of electricity, natural gas, propane and/or fuel oil. A few residents may also use wood or pellet stoves for heating purposes. There is no known energy producing facilities within the planning area.

2.2.15 Economic and Social Profile

The area's economy is based primarily on the agricultural and service industries. Many residents commute to larger cities, such as Burley and Twin Falls, for work. Tourism and recreation are significant contributors to the area's economy. In addition to Pomerelle Ski Resort, Albion is the gateway to the City of Rocks National Reserve, an area known for its unique rock formations. City of Rocks is popular for hiking and rock climbing and is located approximately 30 miles from town. Other recreation activities available within the Albion area include hunting, camping and hiking.

Table 3 summarizes a social profile of the City of Albion.

2.2.16 Environmental Justice

It appears that no disadvantaged group will be adversely affected by a project to improve the existing wastewater facilities. In addition, it is not expected that any specific population segment will benefit from an improvement project. However, the community in general will collectively benefit from improving the wastewater facilities.

2.3 EXISTING WASTEWATER COLLECTION SYSTEM

2.3.1 Existing Collection System

The City's collection system was originally constructed in 1975 and consists primarily of 8 inch asbestos cement gravity sewer mains (see **Figure 8**). Some of these original mains have been repaired or replaced over time with PVC as they have deteriorated. Additionally, several new PVC sewer mains have been installed to service new customers. Since high groundwater conditions may be prevalent within the City, there are very few basements that need to be serviced by the collection system. As a result, most sewer mains are generally between 4 and 8 feet deep. There are approximately 45 manholes and 6 clean-outs in the existing system. **Table 4** provides a summary of the collection system pipe sizes, types and lengths.

Table 3 – Social Profile [A]

Parameter	Value
Sex	
Male	136
Female	131
Total Population	267
Age	
Under 10 Years	39
10 to 19 Years	36
20 to 29 Years	31
30 to 39 Years	24
40 to 49 Years	27
50 to 59 Years	48
60 to 69 Years	29
70 to 79 Years	26
80 and Older	7
Median Age	42.8
Race	
Caucasian	257
African-American	0
Asian	0
American Indian / Alaska Native	0
Native Hawaiian / Other Pacific Islander	0
Other	10
Housing	
Total Housing Units	138
Occupied Housing Units	113
Vacant Housing Units	25
Seasonal, Recreational or Occasional Use	5
Home Owner Vacancy Rate (%)	4.7
Rental Vacancy Rate (%)	14.3
Owner Occupied Housing Units	81
Renter Occupied Housing Units	32

A. Data from 2010 U.S. Census.

Table 4 – Existing Collection System Summary

	Pipe Type		Total Length	
Pipe Diameter	PVC (LF)	Asbestos Cement (LF)	(LF)	(Miles)
8" Gravity Lines	4,300	13,200	17,500	3.3
6" Pressure Mains	4,600	0	4,600	0.98

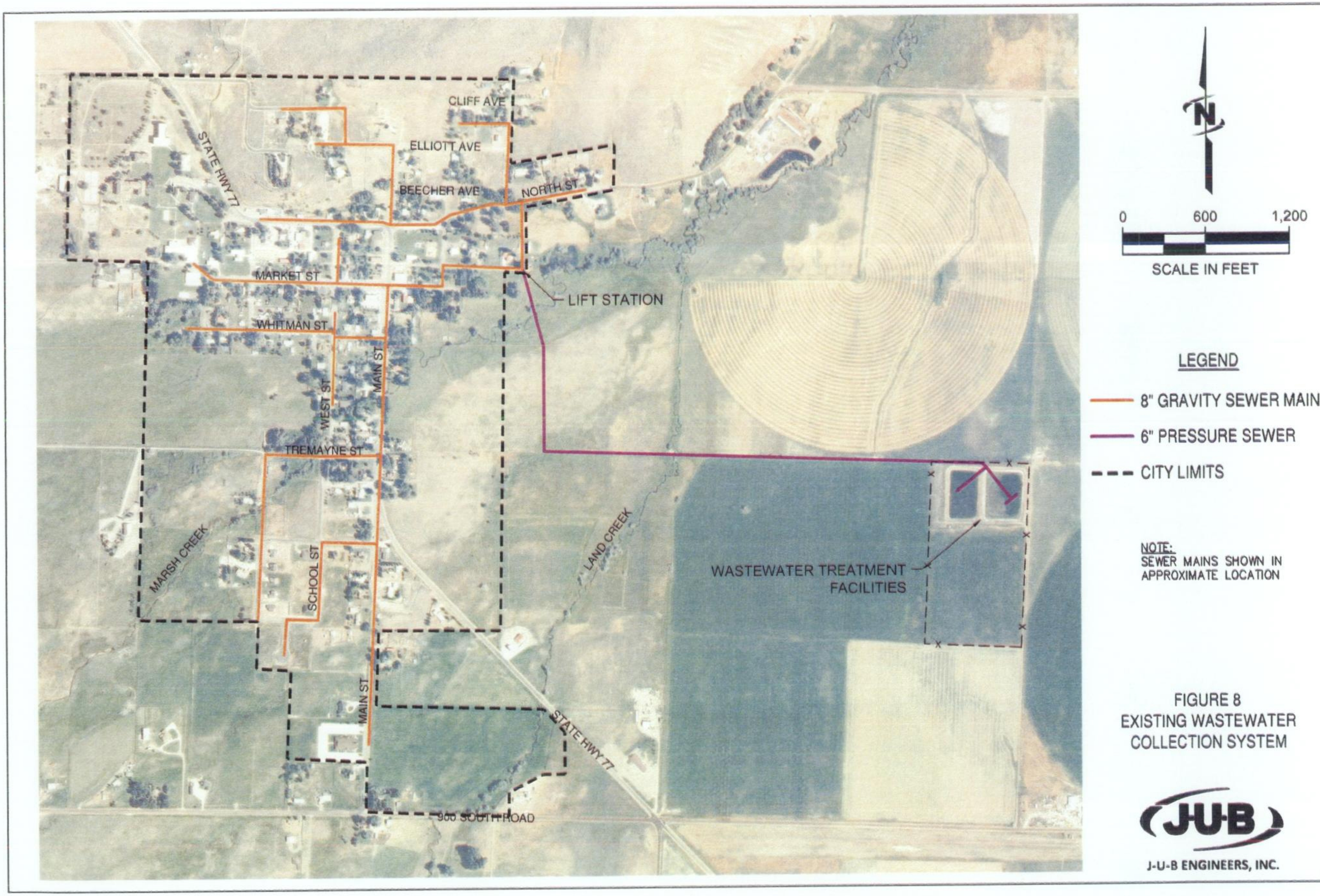


FIGURE 8
EXISTING WASTEWATER
COLLECTION SYSTEM

2.3.2 Existing Lift Station

All of the wastewater from the City is collected and transported to a duplex submersible lift station located at the intersection of South Street and Liberty Street. The lift station was constructed in 1975. Solids collection racks are located below each of the two gravity inlet lines to capture large solids that may damage the lift station pumps. These collection racks also serve to keep rags and other debris from entering the lagoon treatment system.

Figure 9 – Existing Lift Station

The lift station is equipped with two 2-horsepower (hp) submersible pumps that were installed in 2001. Each pump has a design capacity of 180 gallons per minute (gpm) at a total dynamic head (TDH) of 20 feet (see pump curve in **Appendix A**). However, a flow monitoring system installed during July 2011 indicated they were only pumping an average of 67 gpm. This is likely due to their flat pump curve and the system having a higher dynamic head than anticipated. The City has an identical third pump that is used as a spare.

Pump cycling is controlled by float switches in the wet-well. Under normal conditions, operation of the pumps is alternated at the end of each pump cycle. The pumps may also operate in parallel if the wastewater level in the wet-well exceeds the high water level. There is currently no flow meter at the lift station to monitor influent flows to the treatment lagoons. However, pump run-time meters were installed at the time of the original lift station construction. Information for the lift station is provided in **Table 5**.

Table 5 – Existing Lift Station Summary

Item	Lift Station
Pump Manufacturer	Hydromatic Pumps
Number of Pumps	2
Pump Type	Submersible
Pump Horsepower	2.5
RPM	1150
Design Flow and Head per Pump	180 gpm at 20 ft TDH
Actual Flow and Head per Pump	~ 67 gpm at 24 ft TDH
Wet-Well Size	6 ft Diameter and 12.5 ft Deep
Level Sensor	Float Switches
Power	3 Phase, 230 Volts, 60 Hz
Lead/Lag Pump	Alternating
Back-Up Power	Portable Generator
Alarms	High Water, Sensaphone Dialer

Wastewater from the lift station is pumped through a 6-inch PVC pressure main for approximately 4,600 feet to the treatment lagoons. The pressure main was installed at a depth of approximately 3 to 5 feet below the ground surface. Air-vacuum valves were installed at high points in the pressure main to vent any air trapped in the line and to provide a vacuum break. However, it is unknown if these valves are still functional.

2.4 EXISTING WASTEWATER TREATMENT FACILITIES

2.4.1 Overview

The City's wastewater treatment facilities were originally constructed in 1975. They are located approximately one mile east of the City within Section 5 of Township 12 South, Range 25 East, B.M. They system has undergone minor upgrades since its initial construction and currently generally consists of the following components:

- Two facultative lagoons in series.
- A sodium hypochlorite disinfection system.
- A chlorine contact basin.
- An irrigation pump station.
- A 13-acre slow-rate land application (reuse) site.

Figures 10 and 11 show site layouts of the treatment lagoons and land application site, respectively. **Figure 12** provides a general flow diagram and hydraulic profile. **Table 6** summarizes the design parameters for the wastewater treatment facilities as shown on the 1974 Design Drawings. The following sections provide a brief description of the treatment facility components.

2.4.2 Treatment Lagoons

Raw wastewater from the collection system is pumped to two facultative lagoons through a 6-inch pressure main. The pressure main splits into two 6-inch force mains just prior to discharging to the lagoons. One pressure main directs flow to Cell #1 (normal operation) and the other directs flow to Cell #2 (bypass operation). The direction of flow is controlled by buried gate valves on each main. The wastewater is discharged onto concrete splash pads in each lagoon to distribute the flow and prevent erosion of the lagoon liners.

Primary and secondary treatment is achieved by a two stage facultative lagoon system. The lagoons provide for the continuous treatment of the wastewater through several physical and biochemical reactions, resulting in the removal of organics, nutrients and suspended solids. The lagoons also serve as storage reservoirs during the non-growing season (i.e., November to March) when the Reuse Permit does not allow irrigation of the land application site.

The lagoons consist of medium-depth cells (approximately 4 to 6 feet of active depth) operated in series, with Cell #1 being the first in the series under normal conditions. Each of the lagoons is lined with a bentonite clay liner along the bottom and side embankments to minimize seepage. The sides are also lined with 6-inch minimum diameter rip-rap to help control erosion.

There is an 8-inch pipe and transfer structure connecting the two lagoons. This allows partially treated wastewater from one lagoon to gravity flow to the other. The transfer structure consists of a 48-inch diameter concrete manhole with a weir gate and valve to regulate effluent flows and control the water level in the lagoons. During seepage testing in the summer of 2011, the City discovered a leak around the transfer structure from one cell to the other. This was confirmed using a dye test. The City made repairs the lagoon embankment and transfer structure in the spring of 2012.

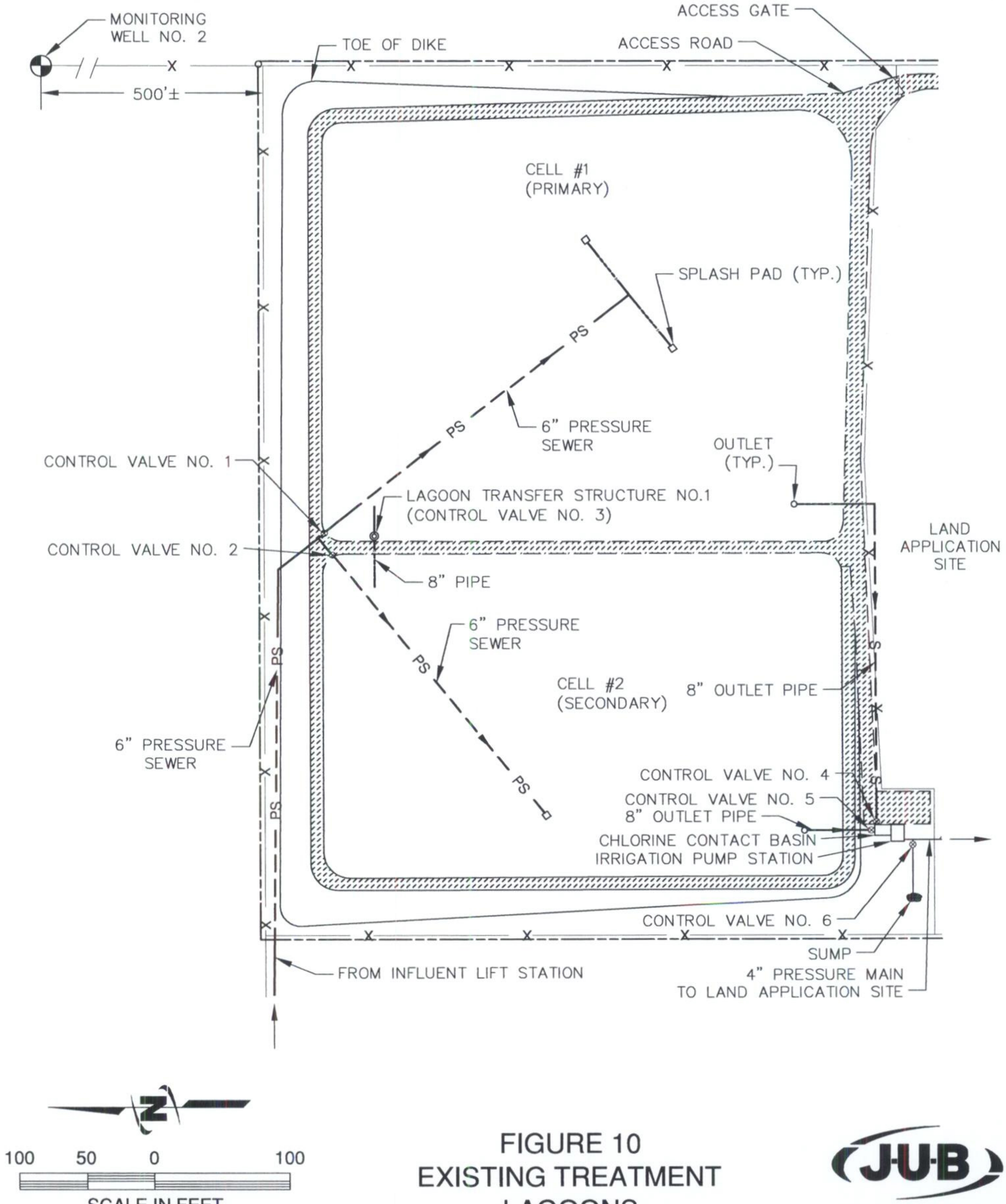
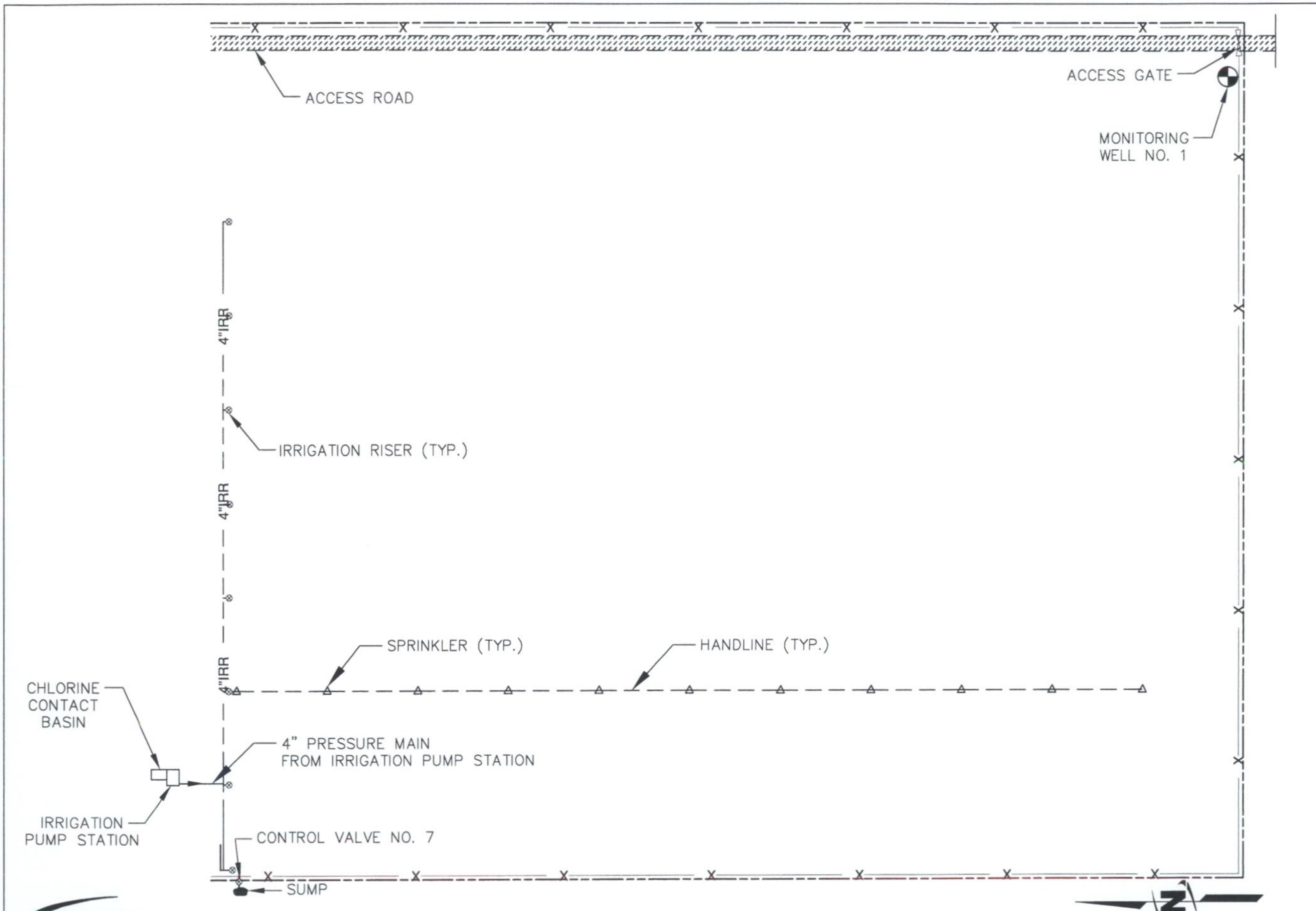


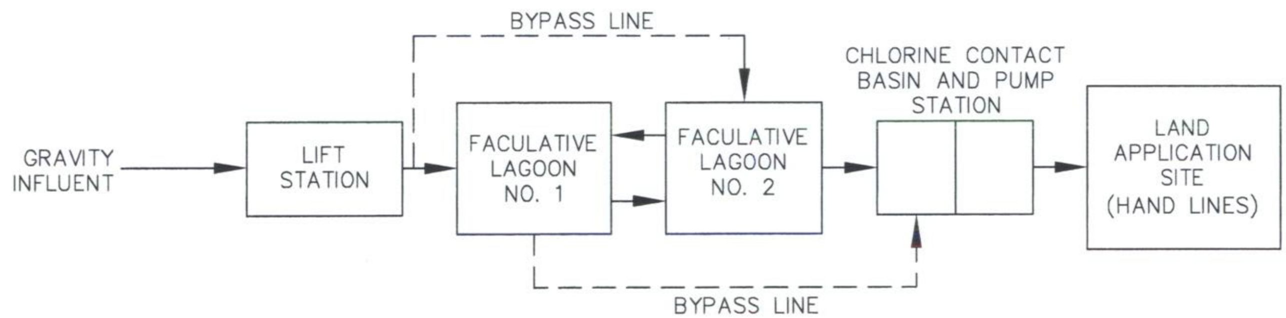
FIGURE 10
EXISTING TREATMENT LAGOONS

J-U-B ENGINEERS, INC.

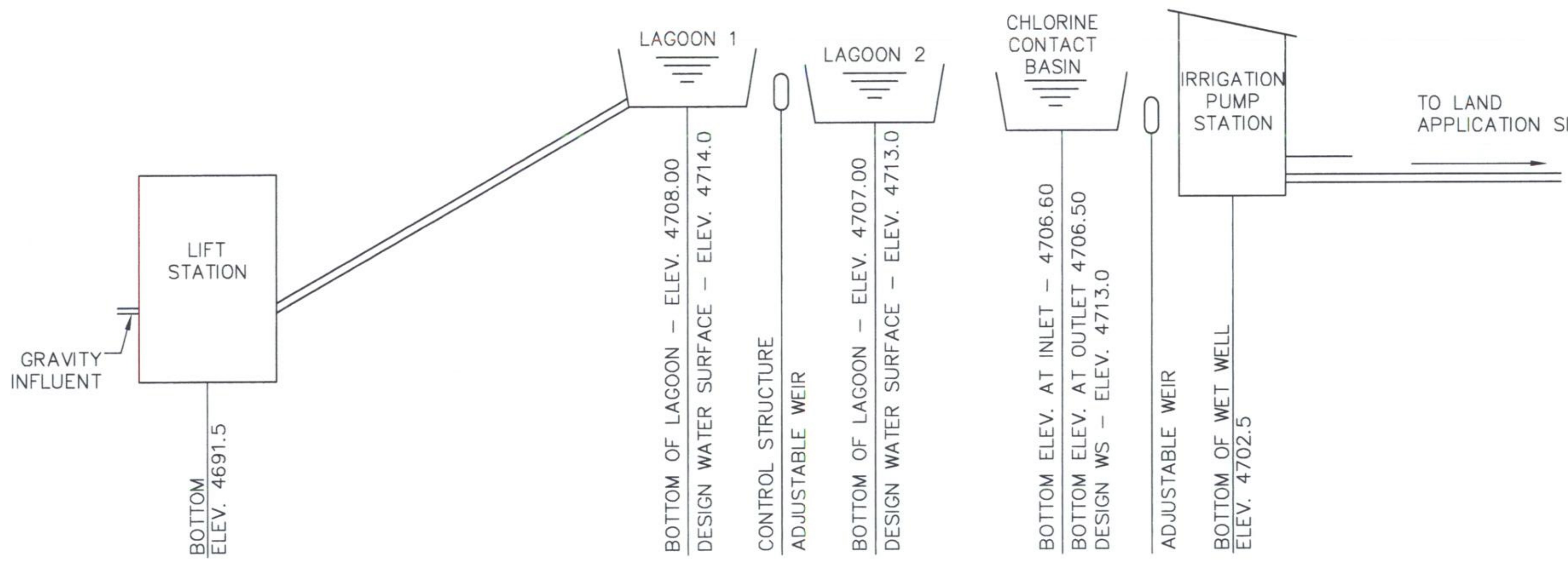


J-U-B ENGINEERS, INC.

FIGURE 11
EXISTING LAND APPLICATION SITE



FLOW DIAGRAM



HYDRAULIC PROFILE

J-U-B ENGINEERS, INC.

FIGURE 12
FLOW DIAGRAM AND HYDRAULIC PROFILE

Table 6 – Existing Wastewater Treatment Facilities Design Parameters [A]

Design Parameter	Units	Value
Population Equivalent	People	385
Loadings		
Hydraulic - Average Day	GPD	38,850
BOD_5	lbs/d	65
Primary Lagoon #1		
BOD_5 Loading Rate	lbs/acre-d	24
Maximum Surface Area	Acres	2.63
Maximum Volume	Million Gallons	5.1
Active Volume [B]	Million Gallons	3.2
Hydraulic Retention Time [C]	Days	131
Top of Dike Elevation	Feet	4,716
Maximum Water Surface Elevation	Feet	4,714
Average Bottom Elevation	Feet	4,708
Secondary Lagoon #2		
Maximum Surface Area	Acres	2.00
Maximum Volume	Million Gallons	3.9
Active Volume [B]	Million Gallons	2.4
Hydraulic Retention Time [C]	Days	100
Top of Dike Elevation	Feet	4,715
Maximum Water Surface Elevation	Feet	4,713
Average Bottom Elevation	Feet	4,707
Land Application Site		
Permitted Acreage	Acres	13
Irrigation Method	-	Hand Lines

A. Design parameters from 1974 Plans for the Sewerage Project, City of Albion (J-U-B ENGINEERS, Inc.).
B. Assumes that the water level in the lagoon does not drop below 2 feet in depth.
C. Calculated using the maximum volume and design flow rate.

2.4.3 Disinfection System

Treated effluent from the lagoons flows by gravity to a 5,000-gallon chlorine contact basin for disinfection. The outlet from each lagoon consists of an 8-inch gravity main with a gate valve. The City reports the gate valves are damaged and cannot be closed. As a result, the chlorine contact chamber cannot be isolated. The contact basin is located south of the lagoons and consists of a concrete vault with two redwood baffles. The contact basin was designed to provide mixing following chlorine injection and to allow for contact time at the design flows. **Figures 13 and 14** show layouts of the existing chlorine contact basin.

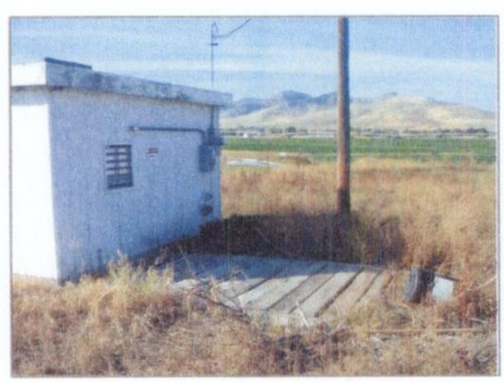
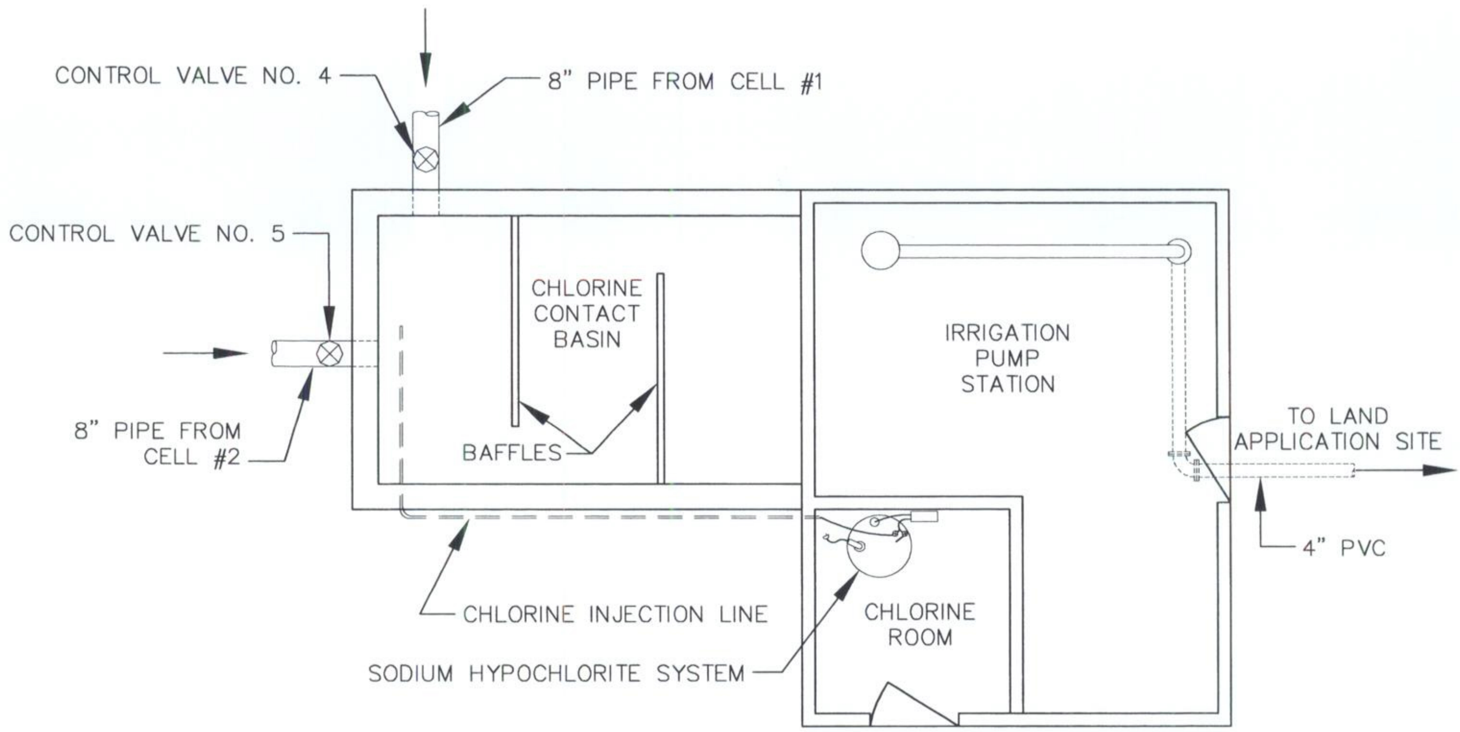
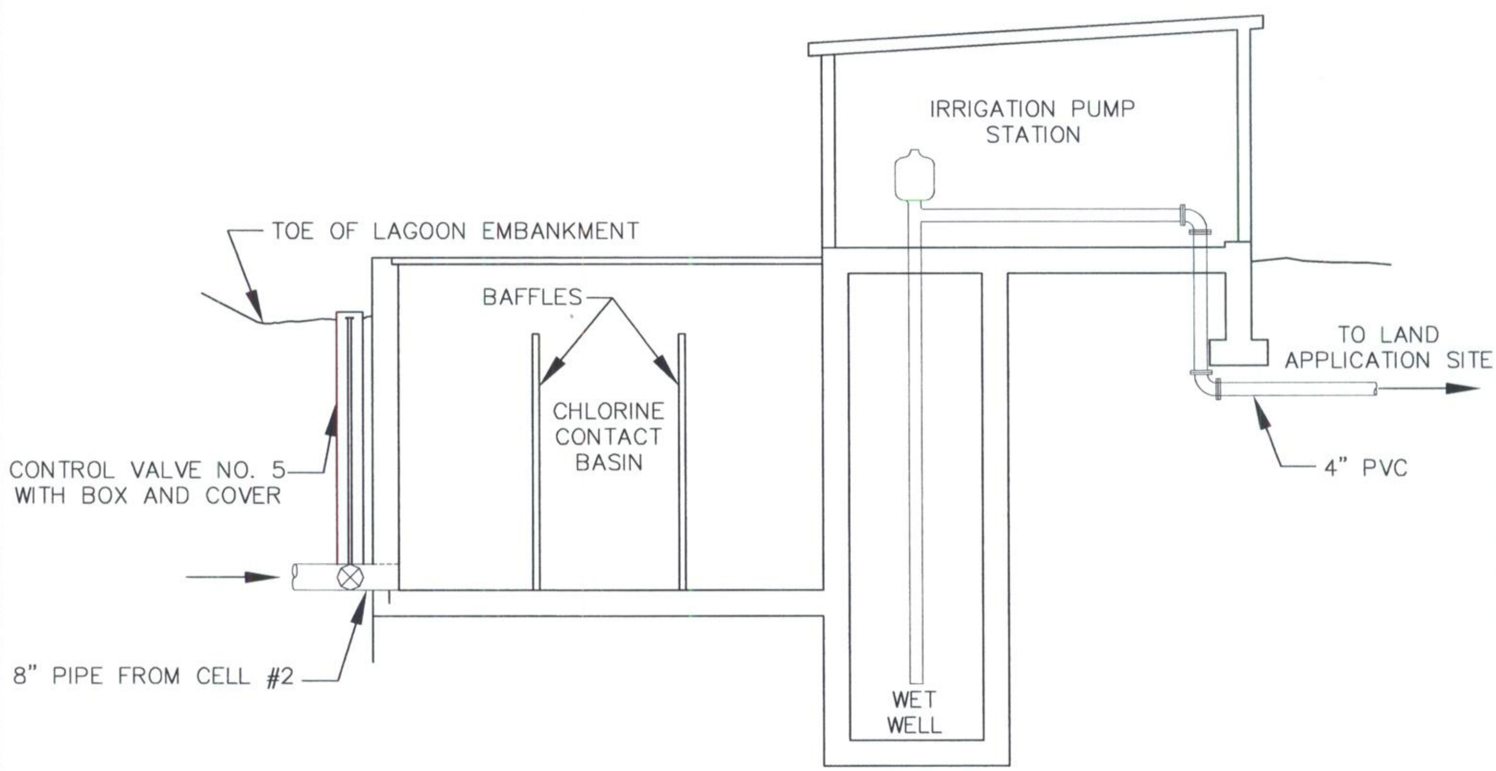


Figure 13 – Existing Chlorine Contact Basin





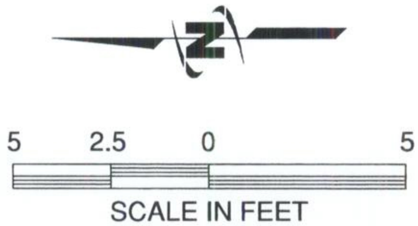


FIGURE 14
EXISTING DISINFECTION SYSTEM
AND IRRIGATION PUMP STATION

J-U-B ENGINEERS, INC.

A chlorine gas system was originally installed in 1975 to disinfect the effluent as it enters the contact basin. The chlorination equipment consisted of two 150-pound chlorine gas cylinders, a wall mounted v-notch chlorinator, cylinder scales and miscellaneous piping and appurtenances. However, the chlorine gas system fell into disrepair in the early 2000s. As a result, the City could not reliably and consistently meet the total coliform limits in the Reuse Permit.

In 2004, the chlorine gas system was decommissioned and a new liquid sodium hypochlorite disinfection system was installed. The hypochlorite system is located in a separate, ventilated room in the pump house (see **Figure 14**). Sodium hypochlorite (10 to 12.5 percent) is injected into the effluent at the head of the contact basin, which then provides for contact time. All effluent is disinfected prior to discharge at the land application site. **Table 7** summarizes the design parameters for the sodium hypochlorite disinfection system.

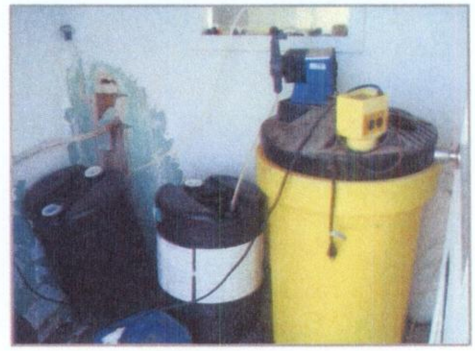


Figure 15 – Existing Hypochlorite System

Table 7 – Existing Sodium Hypochlorite Disinfection System

Parameter	Value
Disinfection Chemical	10 – 12.5% sodium hypochlorite
Metering Pump Size	1.0 gph at up to 110 psi
Metering Pump Control	Manual
Contact Tank Volume at Max. Water Surface Elevation	5,710 gallons
Contact Tank Volume at Min. Water Surface Elevation A	2,640 gallons

A. The City lowers the water level in the lagoons (to approximately 2 feet) in the fall of each year to provide for storage during the non-growing season.

2.4.4 Irrigation Pump Station

Following disinfection, effluent is pumped to the land application site using a 10-hp vertical turbine pump that was installed in 1975. The City reports that the pump was rebuilt in 2011. The pump and associated equipment are housed within the ventilated, wood-framed pump house (see **Figure 14**). The pump was designed to pump 140 gpm at 175 feet of TDH. Effluent is discharged through a 4-inch main to the land application site. Effluent flow is measured using a propeller meter on the 4-inch discharge main.

Figure 16 – Irrigation Pump

2.4.5 Wastewater Reuse Site

The City is currently permitted to irrigate the treated effluent on a 13-acre slow-rate land application under an IDEQ Wastewater Reuse Permit (LA-000077-03) issued by IDEQ (see **Appendix B**). The site is located immediately south of the existing lagoons. Alfalfa hay is typically grown on the land application site. The City currently contracts with a local farmer for irrigation and harvesting of the alfalfa crop.

Effluent is applied to the site using handlines during the irrigation season (e.g., April 1 through October 31 of each year). No effluent is irrigated during the non-growing season (e.g., November 1 through March 31 each year) and during harvesting of the crops. Wastewater is stored in the lagoons during the non-growing season. Typically, effluent is pumped continuously for a few weeks and then the pump is turned off for a period of time depending on crop demand. Operation of the pump is controlled manually by the farmer who has been contracted to farm the land application site. The farmer generally tries to adjust the irrigation schedule to match the water demand of the crop. A retention berm is located along the west boundary of the site to prevent potential runoff.

According to 1974 design drawings for the treatment facilities, there are two groundwater monitoring wells for the land application site. One is located near the southeast corner of the site and the other is near the northeast corner of the lagoons (see **Figures 10 and 11**). The exact location of the two monitoring wells is unknown. The farmer from whom the City leases the land, however, has indicated that he has observed the monitor wells in the past. It is unknown whether the wells have been destroyed or abandoned. An effort should be made to locate these wells for future monitoring purposes.

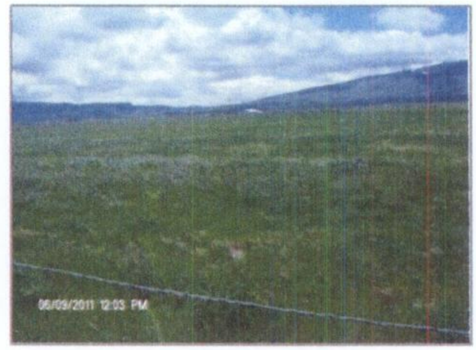


Figure 17 – Existing Land Application Site

2.5 REUSE PERMIT

The City is authorized to discharge their effluent to the slow-rate land application site under an IDEQ Reuse Permit No. LA-000077-03 (see **Appendix B**). The effective date of the permit extends for a period of five years from February 26, 2010 to February 26, 2015. During this time, IDEQ provides regulatory oversight of the permit and operation of the system. Key components of the permit include:

- Facility information (Schedule D)
- Compliance schedule for required activities (Schedule E)
- Permit limits and conditions (Schedule F)
- Monitoring requirements (Schedule G)
- Standard reporting requirements (Schedule H)
- Standard permit conditions (Schedules I and J)

2.6 EXISTING FLOWS AND WASTE LOADS

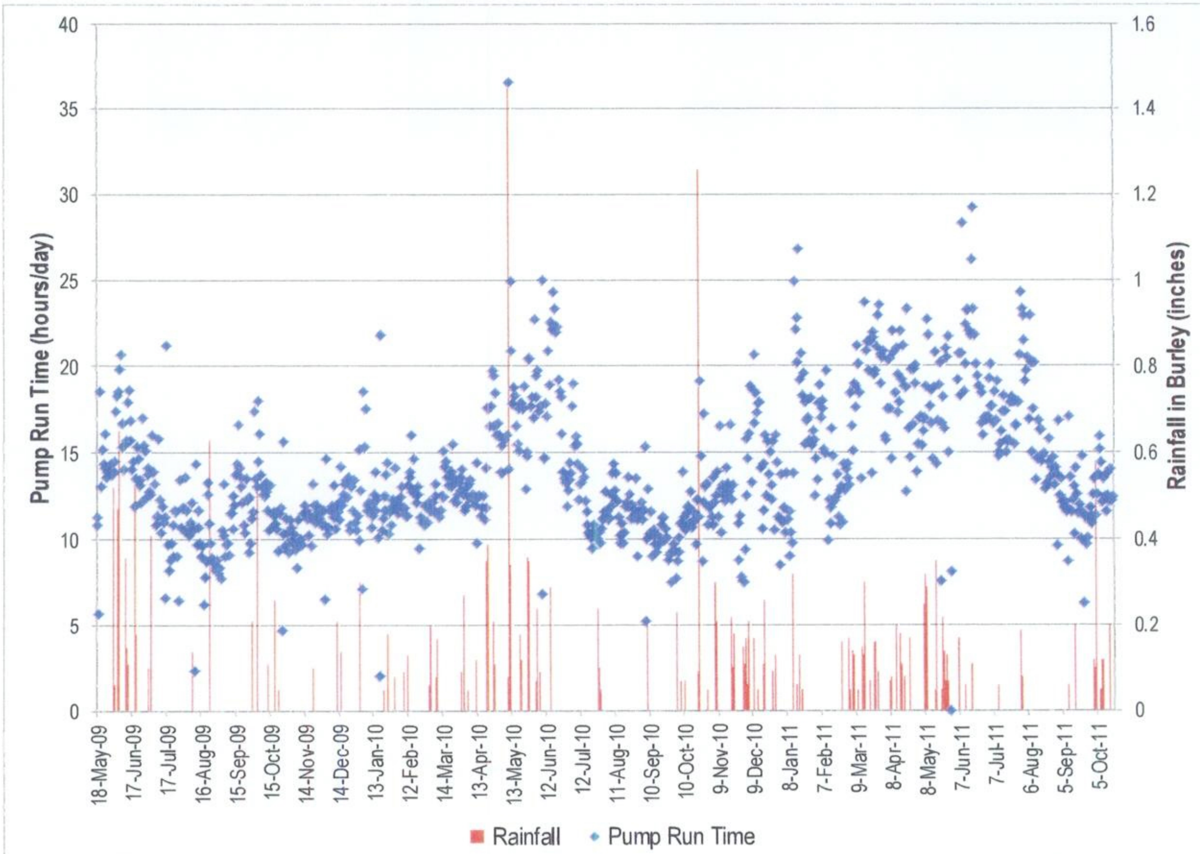
2.6.1 General

Influent lift station pump run time data and wastewater quality data were compiled and analyzed to evaluate existing flows and waste loads. This information was then used to project future flows and waste loads in Chapter 3. Flow and load calculations can be found in **Appendix C**.

2.6.2 Existing Influent Flows

All of the wastewater generated in the City is pumped from the lift station to the lagoons. However, the City does not have a method to measure and record this influent flow. The City does have several years of pump run-time data for the lift station pumps (see **Figure 18**). To utilize the pump run-time data to calculate an influent flow rate, the discharge rate of the lift station pumps must be known.

Figure 18 – Influent Lift Station Pump Run-Time Data (2009-2011)

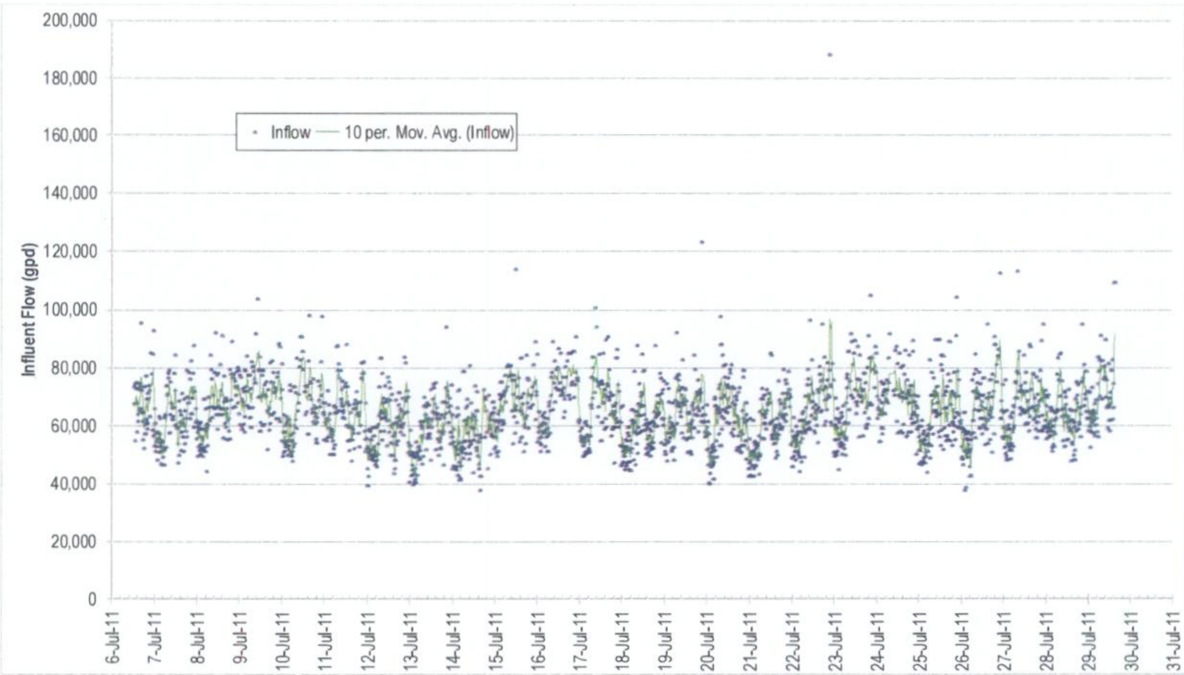


In an effort to better define the pump discharge rates, J-U-B installed a flow monitor in the lift station from July 6 to July 29, 2011. The flow monitor recorded the pump run-times and calculated an average discharge rate of 67 gallons per minute for each pump. **Figure 19** summarizes the lift station flow monitoring data.

Typically, the measured pump discharge rate could be multiplied by the pump run-time data to estimate an influent flow rate. However, further examination of the pump curve for the lift station pumps indicates that it is very flat (see **Appendix A**). As a result, small changes in head conditions will result in large changes in the pump discharge rates. Since the water level in the lagoons can fluctuate greatly throughout the year, especially during the growing season and when the lagoons are drawn down in the fall, the pump discharge rates will vary greatly. The 67 gpm discharge rate was measured in July when the lagoons were full, resulting in higher head conditions. However, based on the pump curve, it is likely the pumps discharge more than 90 gpm when the lagoon water levels are lowered and head conditions decrease.

The other uncertainty related to the pump-run time data is that the City reported there was a period of time when Pump #2 appeared to be turning on, but not actually pumping. Despite not pumping any wastewater, the damaged pump was logged as "running" in the run-time data. The City reported the pump was repaired in the summer of the 2011, but was unsure of the exact date. Upon review, the run-time data from November 26, 2010 to June 30, 2011 appeared to be abnormally high when compared to previous years. For many days during this time period, the combined run time was between 30-45 hours per day. It is likely that one pump was turning on but not actually pumping, forcing the working pump to continue pumping much of the day when the high level float was surpassed. As a result, the Pump #2 run times were removed from the dataset during this 7 month time period. This resulted in run-time data that was more in line with previous years, although the average run-time during this time period was still higher than other years. This was likely due to abnormally high precipitation during the spring of 2011.

Figure 19 – Influent Lift Station Flow Monitoring Data (July 2011)



Based on the preceding analysis, it is not prudent to directly convert the City's extensive pump run-time data to influent flow rates because the actual pump discharge rates vary considerably throughout the year and are unknown. As a result, it appears the existing pump run-time data can primarily be used to qualitatively show seasonal fluctuations and potential effects of infiltration and inflow.

Due to the uncertainties related to the pump run time data, influent flows were estimated using the lift station flow monitoring data from July 2011 and typical per-capita wastewater flow rates based on literature values. Typical design values for domestic wastewater production range from 80 to 120 gallons per capita per day (gpcd) (Metcalf and Eddy, 2003). Other recommended values from the literature include a minimum of 100 gpcd (10 State Standards, 2004) and a maximum of 120 gpcd (IDEQ). For planning purposes, a current residential service population of 267 people and a per-capita flow of 100 gpcd were used to estimate annual average day domestic influent flows. This per-capita value includes residential and commercial flows but not industrial flows or infiltration and inflow (I&I).

The total average flow rate measured in July 2011 was approximately 64,400 gallons per day (gpd). It is assumed the flows measured in July 2011 are representative of an annual average based on a review of the lift station pump run-time data. The measured flow of 64,400 gpd is more than twice the flow rate than would be expected for a City of Albion's size and is an indicator of I&I. Since Albion currently does not treat any industrial flows, it is assumed the difference between the measured flow rate and the typical domestic flow rate is due to I&I.

The existing average day influent flow is summarized in **Table 8**. The calculated I&I value correlates well with the minimum baseline flow condition observed in the middle of the night when there is minimal domestic flow contribution (see **Figure 19**).

Table 8 – Existing Influent Average Day Flow

Parameter	Annual Average Day Flow
Domestic	26,700 gal/day [A]
Infiltration and Inflow	37,700 gal/day [B]
Annual Average Day Influent Flow	64,400 gal/day [C]

A. Based on 100 gpcd (typical design value) and 267 residents (2010 US Census).
B. Difference between total flow rate measured in July 2011 and typical domestic flows
C. Average day flow monitoring results (July 6-29, 2011). Assumed to be representative of average annual flow rate.

In a city the size of Albion, influent flows reaching the treatment facilities will likely vary over the course of a typical day, month, or year due to variations in residential and commercial activities and I&I. As a result, peaking factors are often applied to the annual average day flow to estimate maximum month, minimum month, peak day, and peak hour flows. Due to the limited available flow data, influent flow peaking factors were estimated as follows:

- Domestic influent flow – Domestic peaking factors were estimated by averaging regional peaking factors observed in similar communities in southern Idaho and literature values (see **Table 9**).

- Infiltration – A review of the pump run-time data indicates that infiltration varies seasonally. Typically, peak infiltration events occur in June, while minimum infiltration is usually observed around September. Average peaking factors based on the run-time data from these time periods were used to estimate infiltration peaking factors (see **Table 10**).

- Inflow – As shown in **Figure 18**, inflow into the collection system appears to spike during large rain events. A typical value of 250 gallons/acre was used to estimate inflow into the system over the 120-acre service area. This inflow was added to the annual average infiltration rate to calculate the peak day I&I condition. The peak hour I&I condition was calculated by adding the 30,000 gallons per day inflow to the maximum month infiltration condition.

Table 9 – Typical Domestic Wastewater Flow Peaking Factors

Location	Maximum Month	Minimum Month	Peak Day	Peak Hour
City of Kimberly	1.18	-	1.89	3.31
City of Filer	1.47	0.85	2.11	2.65
City of Hagerman	1.14	0.81	-	-
Carey Water and Sewer District	1.30	0.59	-	-
Modified 10 States Standards (GLUMRB)	-	-	-	**3.21** [A]
City of Albion	**1.27**	**0.75**	**2.00**	**3.21**

A. Estimated using 10 State Standards Equation modified to local conditions $(14+P^{0.5})/(4+P^{0.5})$ where P equals population in thousands (0.267 in 2010)

Table 10 – Existing Infiltration Peaking Factors

Wet Weather Peaking Factor			Dry Weather Peaking Factor		
Month	Pump Run Time (hrs)	Max Month Peaking Factor	Month	Pump Run Time (hrs)	Min Month Peaking Factor
Jun-09	15.5	1.33	Aug-09	10.0	0.86
Jul-09	11.6		Jul-09	11.6	
Jun-10	18.6	1.48	Sep-10	10.4	0.83
Jul-10	12.5		Jul-10	12.5	
Jun-11	18.6	1.08	Sep-11	12.0	0.70
Jul-11	17.2		Jul-11	17.2	
2009-2011 Average Peaking Factor		**1.30**	**2009-2011 Average Peaking Factor**		**0.80**

The peaking factors in **Tables 9 and 10** were applied to the annual average day flows in **Table 8** to estimate maximum month, minimum month, peak day, and peak hour influent flows. **Table 11** summarizes the existing influent flow rates. The existing per-capita flow rate of 241 gallons per person per day is significantly higher than expected as a result of I&I into the system.

Table 11 – Existing Influent Flows

Parameter	Unit	Domestic	I&I	Total
Average Day Flow	gpd	26,700	37,700	64,400 [A]
Maximum Month [B]	gpd	33,900	49,000	82,900
Minimum Month [B]	gpd	20,000	30,000	50,000
Peak Day [B]	gpd	53,400	67,700 [C]	121,100
Peak Hour [B]	gpd	85,700	79,000 [D]	164,700
Average Day Per Capita	gpcd	100 [E]	141 [E]	241 [E]
Total Annual Volume	MGal	-	-	23.5

A. Based on the July 2011 flow monitoring data. .
B. Based on the peaking factors shown in Tables 9 and 10.
C. Average day I&I plus 250 gal/acre over 120 acres of inflow (typical design value).
D. Maximum month I&I plus 250 gal/acre over 120 acres of inflow (typical design value).
E. Based on average day flow rate and a population of 267 in 2010 (US Census Data).

2.6.3 Existing Influent Waste Loads

The City has not historically sampled the influent wastewater to the lagoons. Three samples of the influent were collected at the lift station in October 2011. **Table 12** summarizes the results of these three samples and compares them to typical domestic wastewater and I&I observed in southern Idaho.

It appears the influent concentrations are diluted relative to the typical concentrations. This is another indicator of I&I in the collection system since I&I generally has relatively low concentrations of the constituents typically found in domestic wastewater. It should be noted these influent samples were collected during the time of year that typically has the least amount of I&I. During the spring when I&I flow rates increase, it is anticipated that influent concentrations would be further diluted beyond that shown in **Table 12**.

Table 12 – Existing Influent Sampling Data

Date	BOD (mg/L)	TSS (mg/L)	TKN (mg/L)	Nitrate-Nitrogen (mg/L)	Total Phosphorus (mg/L)
10/13/2011	104	111	14.1	<0.3	3.71
10/19/2011	275	601	24.6	<0.3	5.07
10/26/2011	151	136	20.4	<0.3	6.44
Average	**177**	**283**	**19.7**	**<0.3**	**5.07**
Typical Domestic Wastewater Strength	***150 – 450 (300 avg)***	***150 – 450 (300 avg)***	***20 – 60 (50 avg)***	***<0.3***	***4 – 12 (8 avg)***
Typical I&I Wastewater Strength	***5***	***5***	***1***	***1.3 [A]***	***0.5***

A. Nitrate concentration as reported in the Albion potable water system.

Since the historical influent sampling data is limited (e.g., three data points taken within 2 weeks), average constituent concentrations typically observed in domestic wastewater and I&I were assumed for planning purposes.

Similar to flows, the influent waste loads will vary over the course of a typical day, month, or year. As a result, peaking factors are applied to the annual average day loads to estimate maximum month and peak day loads. Due to the limited sampling data, domestic influent load peaking factors were estimated based on peaking factors observed in similar communities in southern Idaho and literature values. **Table 13** summarizes the peaking factors used in the domestic influent waste load analysis. Peaking factors for I&I loads were assumed to be the same as for flows.

Table 13 – Typical Domestic Wastewater Load Peaking Factors

Parameter	City of Kimberly	City of Filer	City of Hagerman	Metcalf & Eddy [A]	City of Albion
BOD					
Max Month	1.30	1.40	1.69	1.25	**1.41**
Peak Day	2.27	3.20	-	2.50	**2.66**
TSS					
Max Month	1.51	1.50	1.50	1.30	**1.45**
Peak Day	2.30	4.20	-	2.70	**3.07**
TKN					
Max Month	1.30	1.30	1.51	1.30	**1.35**
Peak Day	2.17	2.20	-	2.17	**2.18**
Nitrate-N					
Max Month	1.30	1.30	1.95	1.30	**1.46**
Peak Day	2.17	2.20	-	2.17	**2.18**
Total Phosphorus					
Max Month	1.25	1.30	1.47	1.25	**1.32**
Peak Day	1.75	1.80	-	1.75	**1.77**

A. From Metcalf & Eddy, 2003, Figure 3-8, p. 195.

Table 14 summarizes the existing influent waste loads.

Table 14 – Existing Influent Waste Loads

Parameter		Units	Domestic	I&I	Total
BOD_5	Average Day	mg/L	300	5	***127***
		ppd	67	2	***69***
		ppcd [A]	0.25	0.01	***0.26***
	Maximum Month	ppd	95	2	***97***
	Peak Day	ppd	178	3	***181***
TSS	Average Day	mg/L	300	5	***127***
		ppd	67	2	***69***
		ppcd [A]	0.25	0.01	***0.26***
	Maximum Month	ppd	97	2	***99***
	Peak Day	ppd	205	3	***208***
TKN	Average Day	mg/L	50	1	***21***
		ppd	11.1	0.3	***11.4***
		ppcd [A]	0.042	0.001	***0.043***
	Maximum Month	ppd	15.0	0.4	***15.4***
	Peak Day	ppd	24.2	0.6	***24.8***
Nitrate-N	Average Day	mg/L	0.5	1.3	***1.0***
		ppd	0.1	0.4	***0.5***
		ppcd [A]	0.0004	0.0015	***0.0019***
	Maximum Month	ppd	0.1	0.5	***0.6***
	Peak Day	ppd	0.2	0.8	***1.0***
Total Phosphorus	Average Day	mg/L	8.0	0.5	***3.6***
		ppd	1.8	0.2	***2.0***
		ppcd [A]	0.0067	0.0007	***0.0074***
	Maximum Month	ppd	2.4	0.2	***2.6***
	Peak Day	ppd	3.2	0.3	***3.5***

A. Based on a population of 267 in 2010 (U.S. Census data).

Chapter 3

Future Conditions

3.0 FUTURE CONDITIONS

3.1 FUTURE LAND USE AND DEVELOPMENT

A review of the City's Planning and Zoning map (see **Figure 5**) indicates the planning area is predominantly zoned for residential and residential-agricultural land use. In addition, a sizeable portion of the City is zoned for commercial use. The commercial areas are generally located along Highway 77 and encompass restaurants, hotels, special event facilities, banks, and other businesses. No areas are zoned for industrial use in the planning area.

Discussions with the City indicate that some residential population growth is anticipated for the 20-year planning period. The Mountain Meadows subdivision has been constructed on the south side of town over the past several years. Many of the sites have been developed; however, vacant lots remain for another 6 to 10 single-family homes in this subdivision. In addition, a developer recently approached the City proposing the concept of a new 30 to 40 single-family home subdivision east of the LDS Church. Between these two subdivisions, it is anticipated the City could see an increase of 40 to 50 single-family homes in the next 10 to 20 years. The City anticipates there may be other residential development within the planning area during the planning period.

Another area within the City that may experience growth in the next 20 years is the Albion Campus Retreat Center. This facility is utilizing the old Albion State Normal College as a family reunion and events center. The owner reports that all 15 rooms in the facility are typically booked every day for the entire summer season. During the winter, rooms fill sporadically; mostly on weekends to accommodate people skiing at nearby Pomerelle ski resort. In the next 10 to 20 years, the owner intends to renovate the gymnasium to use as a wedding venue. They also hope to add up to 40 recreational vehicle (RV) sites on the property. The City has indicated they would likely not allow the RV sites to connect into the sewer collection system.

It is likely there will be some commercial development that will accompany the residential growth. Development of commercial businesses will most likely occur along Highway 77. It is anticipated most commercial developments will be "dry" with insignificant wastewater production. New commercial operations with significant water requirements could affect the City's ability to accommodate the additional wastewater using the existing collection and treatment facilities. The City does not plan on any industrial developments within the City limits in the foreseeable future.

3.2 20-YEAR POPULATION PROJECTIONS

3.2.1 Historical Population Growth

As shown in **Table 15**, data from the U.S. Census Bureau indicates that the City's population has fluctuated over the past 50 years. Overall, the City experienced an average annual decrease in population of approximately 0.87 percent per year from 1990 to 2010.

Table 15 – Historical Population Growth

Year	Population A	Average Annual Percent Change
1960	415	
1970	229	-5.78%
1980	286	2.21%
1990	305	0.64%
2000	262	-1.52%
2010	267	0.19%

A. Data from U.S. Census Bureau.

3.2.2 Population Forecast

Population projections were developed for the 20-year planning period to provide the basis for forecasting wastewater flows and waste loads and for evaluating the need for future wastewater system facilities. Based on discussions with the City, three growth scenarios were initially considered:

- Growth Scenario 1 – Assume an annual average 4 percent growth rate over the planning period.
- Growth Scenario 2 – Assume build-out of the existing Mountain Meadows Subdivision (6 to 10 additional homes) plus the development of a new 30 to 40 home subdivision over the next 20 years. This would result in an increase of approximately 50 homes over the 20-year planning period.
- Growth Scenario 3 – Assume an annual average 2 percent growth rate over the planning period.

The three population growth scenarios are summarized in **Figure 20**.

Based on subsequent discussions with the City regarding land use and development patterns in the area, the City selected Growth Scenario 3 as the most reasonable estimate of population growth for the planning period. As a result, an annual average population growth rate of 2 percent was used for planning purposes. **Figure 21** summarizes the estimated population growth for the 20-year planning period.

Based on the 2010 U.S. Census, the total number of permanently occupied houses within the City limits was approximately 108. Given the reported population of 267 in 2010, this equates to approximately 2.47 people per household. This is slightly less than the statewide average of 2.64 people per household, but this is to be expected since Albion tends to have a slightly older demographic than the state as a whole. The planning area generally has a stable year-round population and experiences little, if any seasonal population fluctuations. One exception is the hotel and event centers in town which tend to have more customers during the summer months.

Figure 20 – Population Growth Scenarios

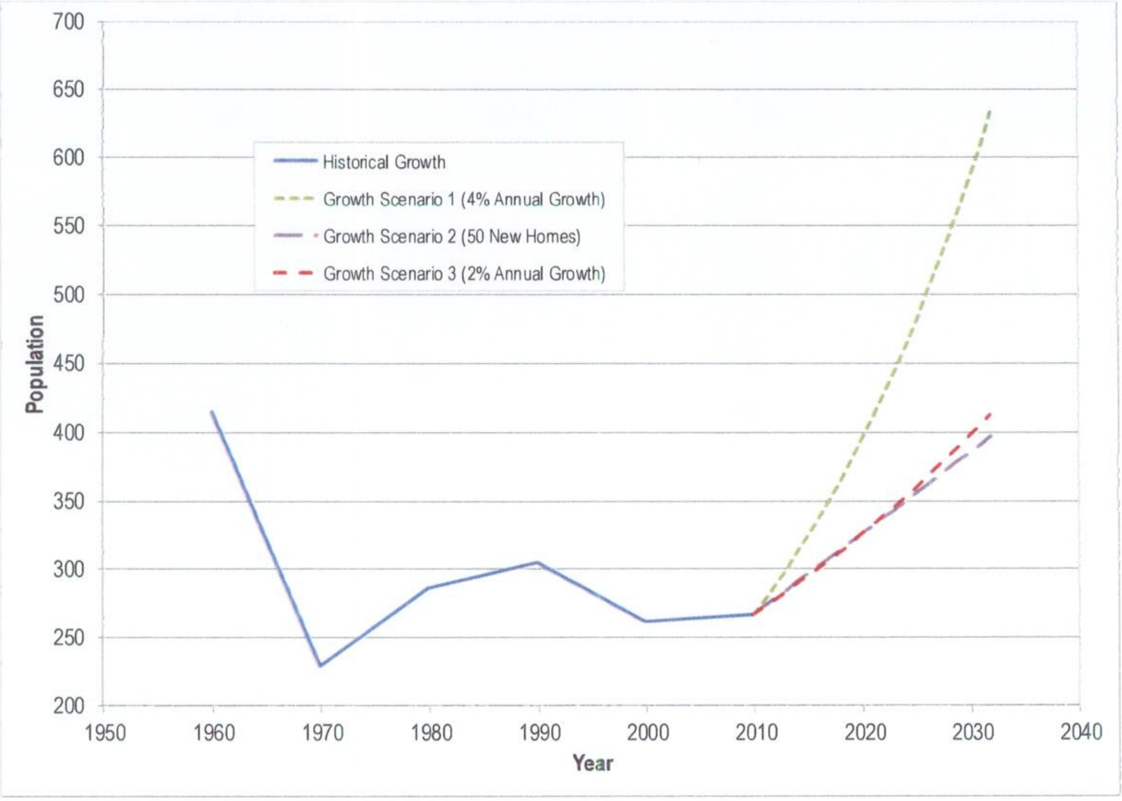
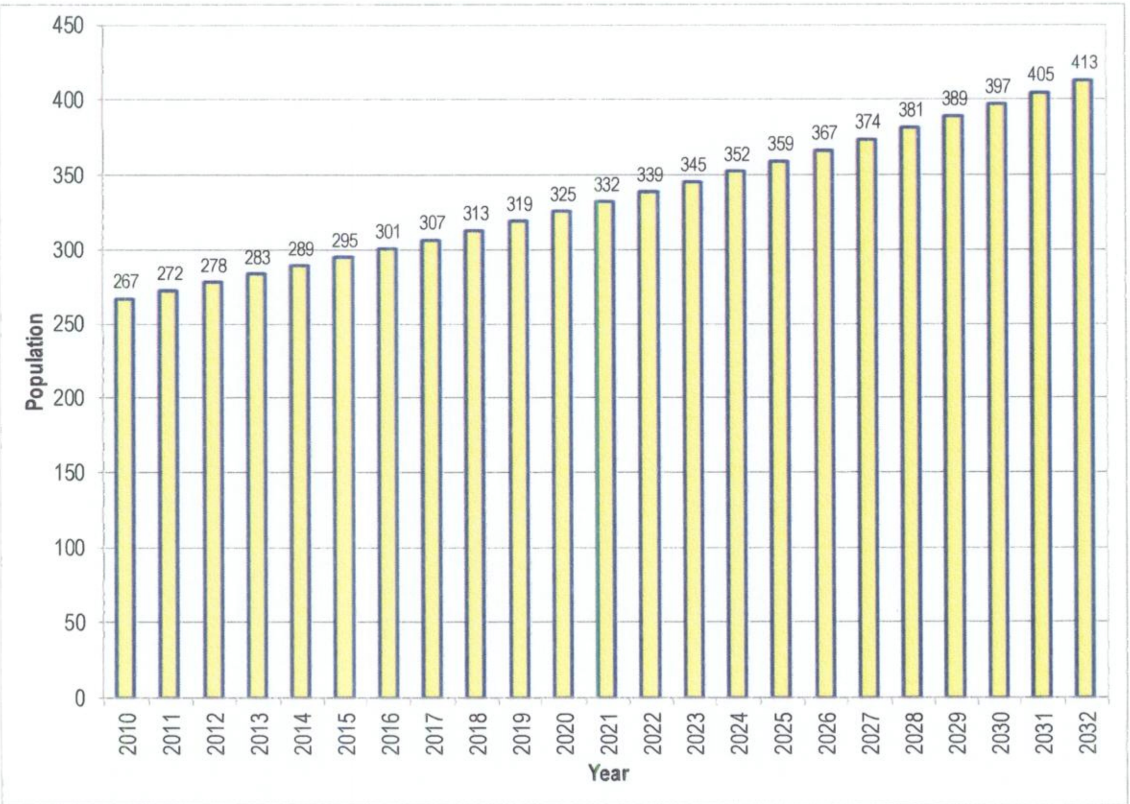


Figure 21 – Population Growth Projections for Planning Period



3.3 FUTURE FLOWS AND WASTE LOADS

3.3.1 Projected Influent Flows

Influent flows to the City's wastewater treatment facilities were projected over the 20-year planning period based on the following assumptions (see **Appendix C** for detailed calculations):

- Average day domestic flows were forecast by multiplying the projected population by 100 gpcd.
- Infiltration and inflow values were assumed to remain the same throughout the planning period. This is likely a conservative assumption because collection system repairs that are anticipated in the future should decrease I&I into the system. Although there will likely be new sewer lines added in the community, it is anticipated that these lines will contribute minimal I&I because new PVC lines are typically relatively watertight.
- No new "wet" commercial or industrial dischargers with significant wastewater discharges are anticipated in the planning period.
- The domestic wastewater peaking factors developed in Chapter 2 (see **Tables 9 and 10**) will be applied to the projected annual average day flow to estimate future maximum month, minimum month, peak day and peak hour flows.

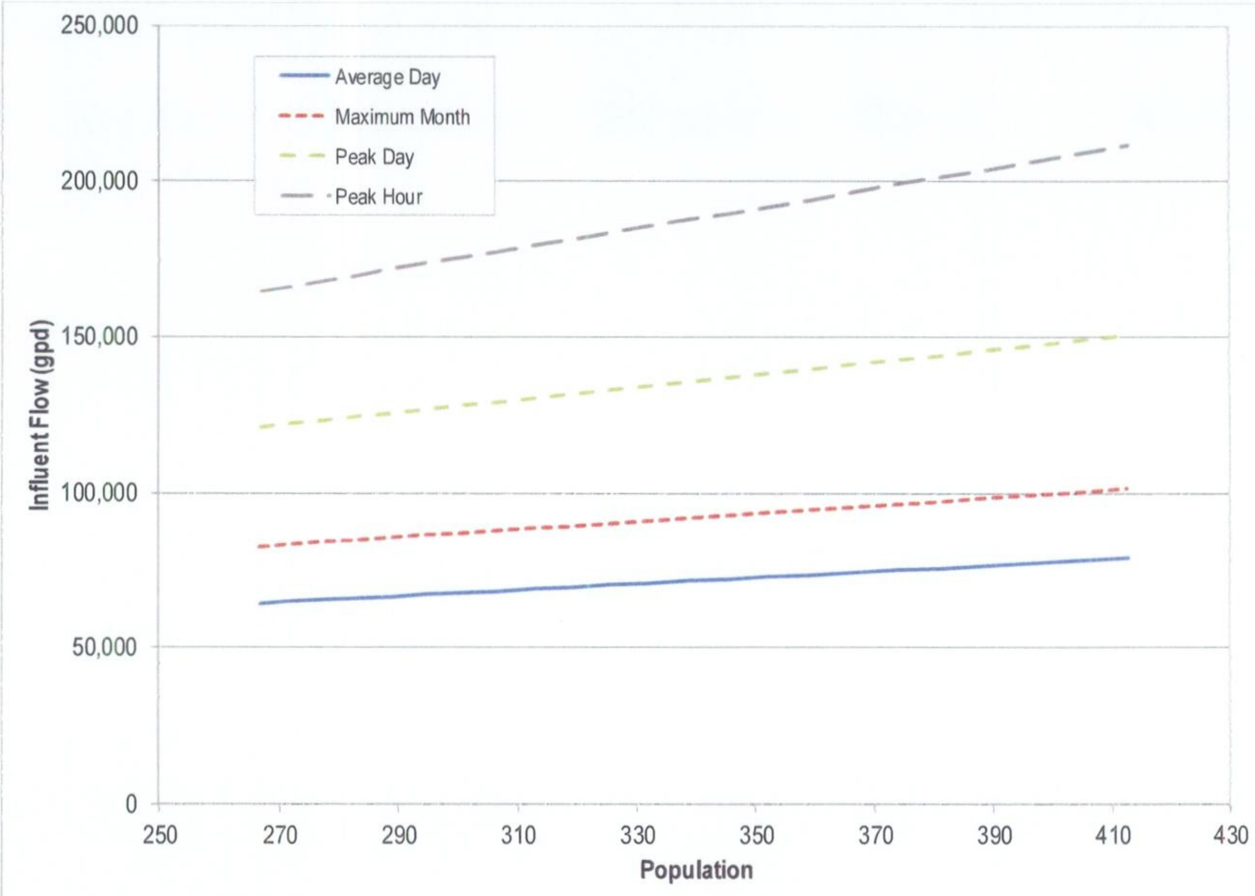
Table 16 summarizes the 20-year projected influent flows. **Figure 22** shows the projected influent flow versus population growth.

Table 16 – Projected Future Influent Flows

Parameter	Unit	Domestic	I&I	Total
Average Day Flow	gpd	41,300	37,700	79,000
Maximum Month A	gpd	52,500	49,000	101,500
Minimum Month A	gpd	31,000	30,000	61,000
Peak Day A	gpd	82,600	67,700 B	150,300
Peak Hour A	gpd	132,600	79,000 C	211,600
Average Day Per Capita	gpcd	100 D	91 D	191 D
Total Annual Volume	MGal	-	-	28.8

A. Based on the peaking factors shown in Tables 9 and 10.
B. Average day I&I plus 250 gal/acre over 120 acres of inflow (typical design value).
C. Maximum month I&I plus 250 gal/acre over 120 acres of inflow (typical design value).
D. Based on average day flow rate and a population of 413 in 2032.

Figure 22 – Projected Influent Flows



3.3.2 Projected Influent Waste Loads

Influent waste loads to the City's wastewater treatment facilities were projected over the 20-year planning period based on the following assumptions (see **Appendix C** for detailed calculations):

- Flow-weighted annual average day waste loads were calculated using the existing domestic and I&I concentrations in **Table 12** and the projected domestic and I&I average day flows in **Table 16**.

- No new "wet" commercial or industrial dischargers with significant wastewater discharges are anticipated in the planning period.

- The domestic wastewater peaking factors developed in Chapter 2 (see **Table 13**) were applied to the projected annual average day domestic loads to estimate future domestic maximum month, minimum month, peak day and peak hour flows.

- Infiltration and inflow loads were assumed to remain the same throughout the planning period.

Table 17 summarizes the 20-year projected influent waste loads.

Table 17 – Projected Future Influent Waste Loads

Parameter		Units	Domestic	I&I	Total
BOD_5	Average Day	mg/L	300	5	***159***
		ppd	103	2	***105***
		ppcd [A]	0.25	0.01	***0.26***
	Maximum Month	ppd	145	2	***147***
	Peak Day	ppd	274	3	***277***
TSS	Average Day	mg/L	300	5	***159***
		ppd	103	2	***105***
		ppcd [A]	0.25	0.01	***0.26***
	Maximum Month	ppd	150	2	***152***
	Peak Day	ppd	316	3	***319***
TKN	Average Day	mg/L	50	1	***27***
		ppd	17.2	0.3	***17.5***
		ppcd [A]	0.042	0.001	***0.043***
	Maximum Month	ppd	23.3	0.4	***23.7***
	Peak Day	ppd	37.5	0.6	***38.1***
Nitrate-N	Average Day	mg/L	0.5	1.3	***0.9***
		ppd	0.2	0.4	***0.6***
		ppcd [A]	0.0005	0.0010	***0.0015***
	Maximum Month	ppd	0.3	0.5	***0.8***
	Peak Day	ppd	0.4	0.8	***1.2***
Total Phosphorus	Average Day	mg/L	8.0	0.5	***4.4***
		ppd	2.8	0.2	***3.0***
		ppcd [A]	0.0068	0.0005	***0.0073***
	Maximum Month	ppd	3.7	0.2	***3.9***
	Peak Day	ppd	4.9	0.3	***5.2***

A. Based on a population of 413 in 2032.

Chapter 4

Evaluation of Existing Facilities

4.0 EVALUATION OF EXISTING FACILITIES

4.1 WASTEWATER COLLECTION SYSTEM

4.1.1 Condition of Existing Gravity Sewer Mains

The sewage collection system was installed in 1975 and is comprised primarily of aging asbestos cement pipe. Pipe of this age is likely to exhibit some deterioration, potentially included cracks and breaks in the pipe walls and joints; erosion and/or corrosion of the interior pipe surface, solids build-up, roots, and sags/bellies. A video inspection of approximately 1,600 lineal feet of the gravity sewer mains in 2010 showed some signs of deterioration; however, in general the pipe walls appeared to be in relatively good structural condition.

As presented in Chapter 2, the primary concern with the gravity collection system in I&I. Groundwater depth within the service area ranges from near the surface to approximately 5 to 10 feet below the surface. The gravity collection system lines are generally between 5 and 8 feet below the surface, so they are often exposed to groundwater. The City reports the bedding material for the pipes is typically washed, smooth gravel. It is likely the groundwater travels through the bedding and tracks along the pipe until it can infiltrate into the sewer system via leaky joints, services, or manhole connections. This is particularly true for the asbestos cement pipe. The newer PVC lines typically have tighter joints that limit infiltration.

The 2010 video inspections showed some leaky and/or cracked joints and service lateral saddles that had groundwater infiltrating into the sewers. There also appeared to be water entering the collection system through some of the gravity clean-outs and manholes.

The City has attempted to identify infiltration areas by looking in manholes for abnormally high flows. As a result of these visual observations, the City has implemented spot repairs in several locations in the collection system over the past two years:

- Repaired clean-out leak and leaky joint on Whitman Street (west of West Street).
- Repaired leaky joint on Main Street near Central Street.
- Removed grease plugs and jet cleaned the North Street gravity line.
- Repaired two leaks near the manhole at North Street and East Street
- Repaired an offset joint upstream from the lift station.

The City reports these spot repairs may have reduced infiltration to an extent, but that groundwater is still entering in other locations and infiltration remains a problem. There is also a potential for cross-contamination of the groundwater if exfiltration of the wastewater occurs through joints, service lateral connections, clean-outs, or manholes.

In addition to infiltration, it is apparent that influent flows increase during periods of heavy rains or rapid snowmelt. To address this issue, the City has raised several manholes to grade or above grade and installed solid lids to try and decrease inflow during storm events.

Due to I&I issues, the City is currently treating more than twice the amount of wastewater than would be expected for a town of Albion's size (e.g., 241 gpcd when 80 to 120 gpcd is more typical). The system processes 1,400 gallons/day/mile/inch diameter of gravity sewer pipe. EPA Guidance (11022 EFF 12/70) recommends flows of 500 gal/d/mile/inch diameter or less.

The 2010 video inspection also showed heavy grease and solids buildup in the North Street trunk line due to two restaurants in the vicinity. The City has been forced to jet clean the grease out of this section of pipe on numerous occasions over the past few years. Neither restaurant on North Street currently has a grease trap; however, both of them have a screen installed on the sewer line from the kitchen that they try to maintain. The City has been working with the restaurant owners to decrease the quantity of grease entering the sewer system through grease traps and have informed the owners that this is required by City Ordinance. One of the restaurants is currently installing an improved grease trap, while the other restaurant recently went out of business.

The City reports that most of the existing concrete manholes are in relatively good condition, considering that most of them are also over 35 years old. The manholes are reported to have minor to moderate erosion/corrosion of the surface and minor cracks. The limited video inspection confirmed the reported condition of the manholes.

The City would like the entire gravity collection system to be cleaned and video inspected. They currently don't have a regular schedule for cleaning and videoing the lines.

One other concern expressed by the City regarding the gravity collection system is the trunk line which enters the lift station wet-well from the north. This trunk enters the wet-well below the water surface. Since the pumps are undersized to accommodate the influent flows, wastewater occasionally surcharges up this line when water levels rise in the wet-well and backs up into a nearby residence.

4.1.2 Hydraulic Capacity of Gravity Sewer Mains

A simple hydraulic capacity analysis of the collection system was performed to evaluate whether the existing gravity sewer lines need to be replaced with larger diameter pipes. For the analysis, the maximum hydraulic capacities of the existing 8-inch gravity sewer lines were estimated and compared to the existing and projected peak hour flows. The hydraulic capacity analysis assumed a minimum pipe slope of 0.4 percent to maintain a minimum velocity of 2 feet per second (fps), a maximum Manning's roughness coefficient for asbestos cement pipe of 0.015, and a water depth of 75 percent of the pipe diameter. These assumptions are conservative for the following reasons:

- According to the 1974 design drawings, slopes of the gravity mains vary from 0.47 to 8.0 percent, and the majority of the system was installed at a slope of approximately 1.0 percent. These slopes would result in higher hydraulic capacities than the assumed 0.4 percent used in the analysis.

- A properly installed gravity main can have a water depth of 100 percent of the pipe diameter, which would result in a higher hydraulic capacity than the 75 percent assumed for the analysis.

- No single gravity trunk line transports 100 percent of the wastewater from the community to the lift station. One trunk line collects wastewater from the north side of town, while a second trunk line collects wastewater from the west and south sides of town. Both trunk lines independently discharge directly into the lift station.

The analysis results in a hydraulic capacity of approximately 391,000 gpd for the 8-inch gravity sewer mains. As presented in Chapters 2 and 3, the existing and projected 20-year peak hour flows are approximately 164,700 and 211,600 gpd, respectively. Additionally, the projected 40-year peak hour flow (assuming a 2 percent annual growth rate) is approximately 275,800 gpd. As such, it appears the existing 8-inch gravity sewer lines have sufficient hydraulic capacity to handle the existing and projected peak hour flows and should not need to be upsized during the planning period. It should be noted that this analysis assumes all of the gravity lines were constructed at least at the minimum recommended slope of 0.4 percent and that there are minimal obstructions and/or grade problems (e.g., root intrusion, debris, sags or bellies, flat slopes, etc.) in the lines that could affect the analysis.

4.1.3 Lift Station Condition and Capacity

The original lift station pumps were installed in 1975 and designed to pump approximately 125 gpm at 33 feet of total dynamic head (TDH). These pumps were replaced in 2001 with submersible pumps that were designed to discharge 180 gpm at 20 feet of TDH. However, the pumps were only discharging 65 to 70 gpm during the July 2011 lift station monitoring event. Further examination of the pump curve (see **Appendix A**) and system head curves indicate that:

- The actual TDH is higher than the design value of 20 feet and varies with lagoon water surface elevation. A higher TDH results in a lower pumping rate.

- The pump curve is relatively flat. As a result, small changes in the head conditions will result in large changes in the pumping rate. This is of a concern since the water surface elevation in the lagoons varies as water is drawn down for irrigation and winter storage.

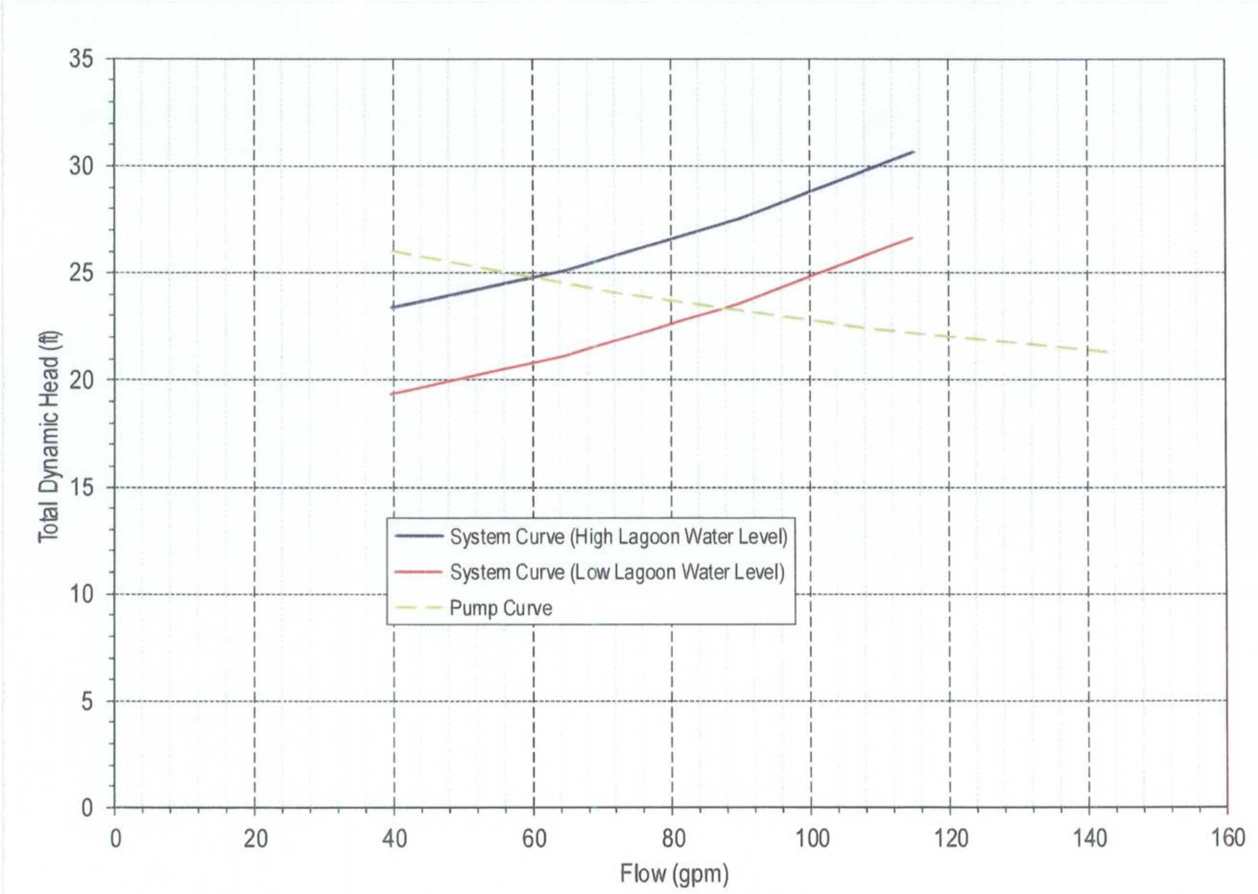
Figure 23 shows the pump curve relative the system head curve for the high and low water surface elevations in the lagoons. As illustrated in the figure, it appears the pump discharge rate varies from approximately 60 to 88 gpm, depending on the water surface level in the lagoons. In the spring when the lagoons are full due to winter storage, the pumps likely pump closer to 60 gpm. In the fall when the lagoons are drained for winter storage, the pumps likely pump closer to 88 gpm.

Since the pumps are not operating near their design point of 180 gpm, they are operating at a lower efficiency (e.., 25 to 35 percent) than the design efficiency (e.g., 52 percent). As a result, power costs for the lift station are likely higher than expected.

Idaho Administrative Rules require a minimum of two pumps that each has the capacity to handle the peak hour flows (IDAPA 58.01.16.440.02.c.i). As presented in Chapters 2 and 3, the existing and projected 20-year peak hour flows are approximately 164,700 gpd (114 gpm) and 211,600 gpd (147 gpm), respectively. Since the existing pumps are only capable of discharging 60 to 88 gpm, it appears they do not have sufficient capacity to pump the existing or projected peak hour flows in accordance with IDEQ regulations. This has been evidenced on several occasions in the past when the lift station has overflowed, or nearly overflowed, spilling raw wastewater onto the surrounding ground. This presents a public and environmental health and safety concern if raw wastewater flows to surface water, infiltrates the groundwater, or is exposed to the public. If I&I is reduced in the future, the existing pumps may have sufficient capacity to handle the peak hour flows.

In addition to capacity issues, the lift station is more than 35 years old and is approaching its useful design life. Most of the exposed metal and concrete is heavily corroded and in need of replacement.

Figure 23 – Lift Station System Head Curve



All of the influent entering the wet-well from the westerly gravity line passes through a bar rack to remove debris. The operator has to manually clean the bar rack and dispose of the debris and rags every 1 to 2 days to avoid blockages and surcharging of the gravity lines. The screenings are disposed of in a 55-gallon drum next to the lift station. The drum is open to the atmosphere and is a source of obnoxious odors. The gravity line entering the wet-well from the north is below the water surface. Although this line is also equipped with a bar rack, it is not useful since the line is submerged. This allows debris, rags and other floatable items to bypass the collection rack. As a result, the City reports they also have problems with rags and debris clogging the pumps. The City also reports the bar racks are corroded and in need of replacement.

Idaho Administrative Rules requires that the lift station have a suitable method for measuring flow rates (IDAPA 58.01.16.440.02.h). The existing lift station does not have a flow meter, nor is there an influent flow measurement device at the treatment lagoons.

Lift stations are also required to provide emergency pumping capabilities (IDAPA 58.01.16.07.b). This regulatory condition may be met by providing a back-up power supply, portable pumping equipment, connection to two independent utility substations, or adequate emergency storage capacity (e.g., twice the estimated emergency response time multiplied by the peak hour flow). The City does have a portable generator that can be used to power the lift station during a power outage. The City is also considering purchasing approximately one-half acre adjacent to the existing lift station to install a permanent back-up generator and to construct a new lift station.

4.1.4 Hydraulic Capacity of Pressure Sewer Mains

A 6-inch PVC pressure main is used to transport the City's raw wastewater approximately 4,600 feet from the lift station to the treatment lagoons. The City reports the pressure main has not been cleaned or inspected since its original construction in 1975. As such, its structural condition is unknown.

Idaho Administrative Rules (IDAPA 58.01.16.440.10.a) require a minimum cleansing velocity of 2 fps in pressure mains at the design pumping rates to prevent the deposition of debris and solids. For a 6-inch pressure main, this requires a minimum pumping rate of 180 gpm. However, the existing pumps only appear to be pumping between 60 to 88 gpm. Velocities in the pressure main at these flows range from approximately 0.68 to 1.00 fps. As a result, it is likely that some debris and solids have settled in the pressure main over time. This may result in reduced hydraulic capacity in the pressure main, increase the potential for a plugged line, and increase future maintenance requirements.

It is recommended that the City clean the pressure main to remove any debris and solids that may have settled in the line. Cleaning can be achieved using a pipe "pig" or by installing pressure cleanouts on the line for hydraulic jetting. A wye connection may need to be installed downstream of the lift station to facilitate cleaning of the force main.

In addition to considering low flow conditions, a maximum flow velocity of approximately 5 fps is typically designed in pressure mains to maintain reasonable friction head losses. A 6-inch sewer pressure main is capable of transporting approximately 400 to 450 gpm (576,000 to 648,000 gpd) at a flow velocity of approximately 5 fps. As such, it appears the 6-inch pressure main has sufficient hydraulic capacity to accommodate the existing and projected peak hour flows. If new, larger pumps are installed at the lift station, they should discharge at a velocity sufficient to flush accumulated debris out of the force main

Air-vacuum valves were originally installed on the 6-inch pressure main; however, the City has not routinely exercised or maintained these valves in the past and their condition and functionality is unknown. If the valves are not functioning properly, trapped air in the line could restrict flow. A vacuum condition could also result in a collapsed pipe. It is recommended that the City locate the air-vacuum valves to exercise and maintain them and to verify they are operating properly. If necessary, the valves should be repaired and/or replaced.

4.2 WASTEWATER TREATMENT LAGOONS

4.2.1 Headworks

There is currently no headworks (e.g., screening, flow measurement, etc.) at the lagoons. Screening of the influent prior to the lagoons should be considered to minimize the amount of debris and solids entering and filling the lagoons. Some screening of the influent occurs at the lift station through the bar racks on the gravity trunk lines. However, these bar racks are in poor condition and the northern one is often submerged, rendering is useless. Screening is recommended at the lagoons through a coarse bar rack.

Idaho Administrative Rules requires flow measurement of the influent flow (IDAPA 58.01.16.450.06.e.i). Flow monitoring also provides a means for evaluating the lagoon performance and assessing I&I into the collection system. The City currently does not have an influent flow measurement device. Influent flow measurement may occur either at the lift station (e.g., magnetic flow meter) or at the lagoons (e.g., splitter box with a Parshall flume and ultrasonic transducer).

4.2.2 Treatment Performance and Effluent Quality

The City samples the effluent discharged to the land application site during the growing season in accordance with their Reuse Permit. Since the discharge is intermittent, there is a relatively limited amount of effluent quality data available for analysis. **Table 18** summarizes the effluent quality from 2009 through 2011.

Table 18 – Historical Effluent Quality (2009 – 2011)

Parameter	Units	Average	Range	Number of Samples
COD	mg/L	108	46 – 199	9
TKN	mg/L	10.5	3.3 – 21.4	9
Nitrate-N + Nitrite-N	mg/L	1.1	0.3 – 4.3	9
Total-Nitrogen [A]	mg/L	11.6	3.6 – 22.0	9
TDS	mg/L	477	270 – 560	9
TVDS	mg/L	161	50 – 350	9
Total-Phosphorus	mg/L	2.9	1.2 – 5.6	9
pH	s.u.	8.4	7.4 – 9.7	9
Total-Coliform	mpn/100 mL	393	1 – 2,407	16
E. Coli	cfu/100 mL	1.3	1 – 2	6

In general, the effluent concentrations appear to be typical for a lagoon system in southern Idaho. This is an indication that the lagoons are performing adequately in terms of constituent removal. It is recommended that the City continue to monitor effluent concentrations in accordance with their Reuse Permit.

4.2.3 Biosolids Accumulation

A portion of the settleable solids in the influent will be removed from suspension within a few hours of entering the lagoons. Additional biological solids will be formed and precipitated within the lagoons as treatment occurs. Algae growth and subsequent die-off may also result in the formation of settable solids in the lagoons. As the solids settle to the bottom of the lagoons, they form a biosolids, or "sludge", layer. A portion of these biosolids undergo anaerobic degradation and are released back into the wastewater as various gases, solids and soluble organics. Biosolids accumulation can be variable due to variations in degradation rates and the formation of algae. Excessive biosolids accumulation may result in potential treatment problems, including:

- A reduction in the operating volume and hydraulic retention time (HRT).
- Increased oxygen demand from the feedback of soluble organics during degradation.
- Elevated effluent TSS and BOD_5 from the feedback of solids and excessive algal growth triggered by the release of nutrients from sludge stabilization.
- High effluent ammonia caused by the release of ammonia during sludge stabilization.

The City has no records of biosolids removal from the lagoons since their 1975 construction. **Table 19** summarizes the estimated biosolids depth and volume, as well as the effective operating water volume, in the lagoons.

Table 19 – Lagoon Biosolids Accumulation

Lagoon	Sludge Depth A (inches)	Sludge Volume (MGal)	Maximum Lagoon Water Volume (MGal)	Operating Lagoon Water Volume (MGal)	% of Maximum Volume Occupied by Sludge
Cell #1	8.0	0.46	4.62	4.16	10.0
Cell #2	8.0	0.33	3.45	3.12	9.6

A. Sludge depth estimated by operator after draining lagoon cells for winter storage. The City reports IRWA came out during the fall of 2011 to measure sludge depth at various locations in the cells using a Sludge Judge but the lagoons were already frozen over and measurements could not be taken.

It appears a relatively small portion of the lagoon volume is likely occupied by accumulated biosolids. It is recommended that the City verify this information by measuring the actual sludge levels. The City should continue to monitor sludge levels in the cells in the future, and remove and dispose of it, as necessary.

4.2.4 Hydraulic Retention Time (HRT)

The treatment performance of the lagoons partially depends on providing an adequate HRT, which is a function of the influent flow rate and cell operating volume. During cold weather conditions, microbial activity is reduced by approximately one-half for every 10ºC decrease in the temperature. As a result, a longer HRT is generally required to maintain a removal efficiency equivalent to that observed during warm weather conditions.

Hydraulic retention times for the lagoons were calculated at the existing and projected average day and maximum month flows. It was assumed the sludge depth in the lagoons remains approximately the same as current levels over the planning period. **Table 20** summarizes the lagoon HRTs under current and future conditions.

Table 20 – Lagoon Hydraulic Retention Times

Lagoon	Existing HRT (days)		Future HRT (days)	
	Average Day	Max Month	Average Day	Max Month
Cell #1	64.6	50.2	52.7	41.0
Cell #2	48.4	37.6	39.5	30.7
Total	113	87.8	92.2	71.7

The HRT required for a facultative lagoon system typically ranges from approximately 20 to 180 days, and may vary considerably depending on site specific conditions (i.e., climate, consortium of microorganisms, effluent limits, etc.). The Recommended Standards for Wastewater Facilities (10 State Standards) recommends an HRT of 90 to 120 days for flow-through facultative systems. The EPA suggests an HRT of approximately 40 to 60 days for systems with an average winter air temperature of 0º to 15ºC (Process Design Manual for Wastewater Treatment Facilities for Sewered Small Communities). In general, it appears that the lagoons provide an adequate HRT under existing and future flow conditions, particularly if I&I is reduced in the future.

The long HRT required for facultative lagoons typically promotes the growth of algae, which are relied upon for aeration through photosynthesis. Algal blooms can often lead to increased levels of BOD_5, TSS, and pH in the effluent. The City indicates that algae blooms have resulted in elevated effluent TSS levels in the past, resulting in non-compliance with the Reuse Permit limits.

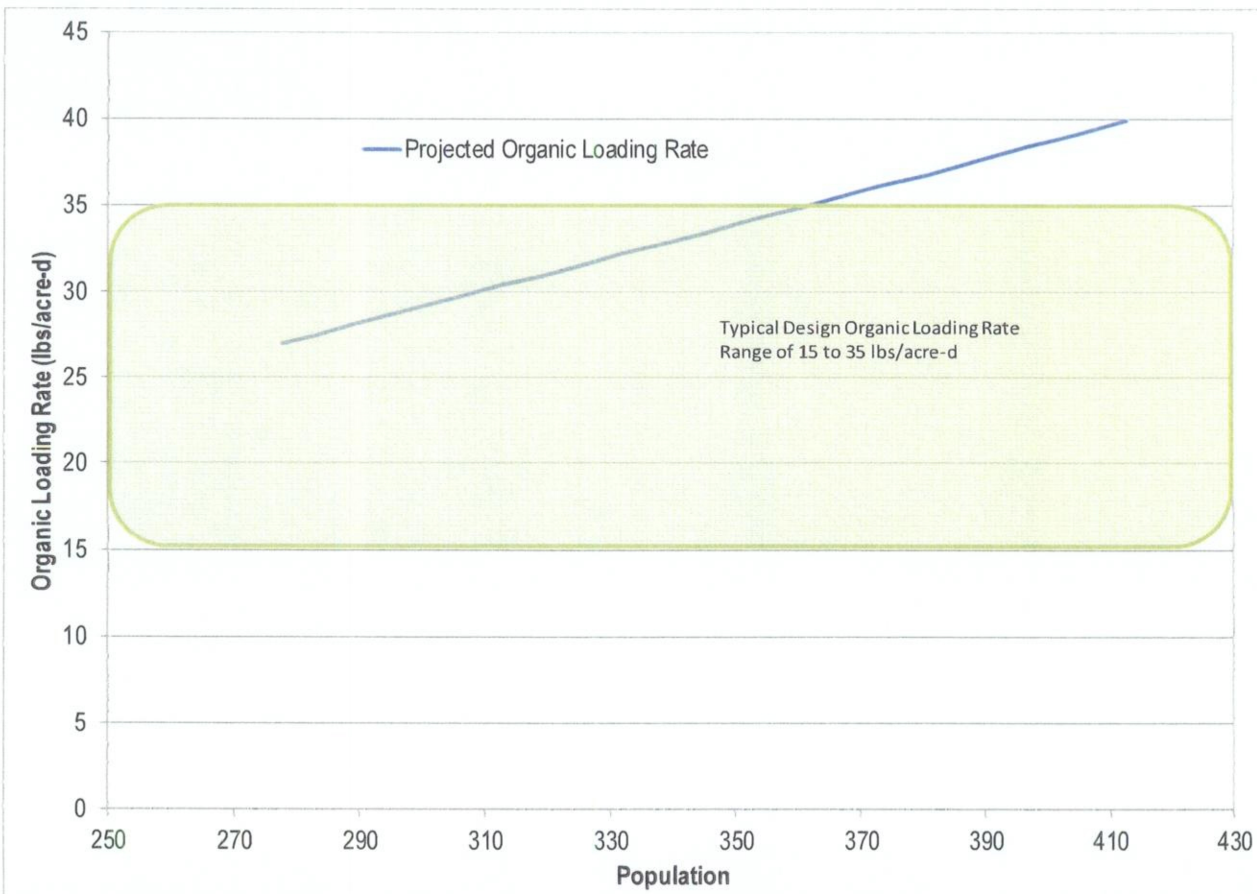
Short-circuiting can also be a problem in facultative lagoon systems if there is insufficient mixing of the wastewater. This is typically most prevalent during cold weather conditions when waters of varying temperatures don't mix well. Short-circuiting may lead to a reduction in the HRT and decreased treatment performance of the lagoons.

4.2.5 Organic Loading

The organic (BOD_5) loading rate to the first cell is another parameter commonly used to evaluate the capacity of a facultative lagoon system. Overloading of organics to the first cell may result in reduced treatment performance and the generation of obnoxious odors as the atmospheric re-aeration capacity of the system is exceeded.

Typical ranges of design organic loading rates to the primary lagoon reported in the literature range from approximately 15 to 60 lbs BOD_5/acre-d. The higher end of this range is typical for lagoons in warmer climates, while the lower end is typical for lagoons in colder climates. The Recommended Standards for Wastewater Facilities (10 State Standards) suggests that organic loading rates to the first cell be limited to approximately 15 to 35 lbs BOD_5/acre-d. **Figure 24** summarizes the current and projected organic loading rates to Cell #1 relative to the 10 State Standards recommendation.

Figure 24 – Organic (BOD_5) Surface Loading to Cell #1



As shown in the figure, the existing organic loading rate is within the typical design range. As population growth occurs, projected organic loading rates appear to exceed the upper recommended value of 35 lbs BOD_5/acre-d. It is recommended the City continue to monitor the treatment performance of the lagoons and for obnoxious odors. If reduced treatment performance and/or odors become a problem in the future, the City may need to consider installing mechanical surface aerators in the lagoons.

4.2.6 Seepage

A portion of the wastewater entering the lagoons is lost to seepage. To minimize potential impacts on the subsurface soil and groundwater, IDEQ has required a maximum seepage rate of 0.25 inches per day for the existing lagoons (refer to the Reuse Permit in **Appendix B**).

The City conducted seepage tests on both lagoons from August 23 to September 25, 2011. Based on these tests, the seepage rate for Cell #2 was lower than the allowable seepage rate; however, the seepage rate for Cell #1 was higher than the allowable seepage rate. Using dye testing, the City discovered a leak around the transfer structure between the two lagoons. This leak was likely the cause of failure in the seepage test for Cell #1. In the spring of 2012, the City replaced the transfer structure and repaired the surrounding lagoon embankment. Following the repairs to the transfer structure, a second seepage test was performed on Cell #1 between August 17 and September 1, 2012. This second test demonstrated that the seepage rate for Cell #1 was lower than the allowable rate. The results of the passing seepage tests are summarized in **Table 21** for both lagoons. Both cells should not have to be tested again until 2021 unless they are damaged, signs of leaking are apparent, or regulations change.

Table 21 – Lagoon Seepage Rates

Lagoon	Lagoon Seepage Rate (inch/day)	Permitted Seepage Rate (inch/day)
Cell #1	0.216 [A]	0.25
Cell #2	0.221 [B]	0.25

A. Seepage testing conducted between August 17 and September 1, 2012.
B. Seepage testing conducted between August 23 and September 7, 2011.

4.2.7 Transfer Structures and Piping

As discussed in Chapter 2, the 6-inch pressure main delivers the influent from the lift station splits into two 6-inch pressure mains just prior to discharging to the lagoons. One pressure main directs flow to Cell #1 and the other directs flow to Cell #2. The direction of flow is controlled by buried gate valves on each main. These valves are buried approximately 8- to 10-feet below the ground surface within the lagoon embankments. Because of their bury depth, the City reports it is difficult to open and close the valves and to know which position they are in. The City would like to reconstruct the pressure mains and isolation valves so they are closer to the surface of the lagoon embankments.

There is an existing 8-inch transfer pipe and structure between the two lagoons. This piping and structure allows partially treated wastewater from one lagoon to gravity flow to the other. During lagoon seepage testing in the summer of 2011, the City discovered a leak around the existing 48-inch diameter transfer structure from one lagoon to the other using a dye test. In the spring of 2012, the City replaced the old transfer structure and repaired the leak in the lagoon embankment. The new transfer structure consists of a 5-foot by 5-foot concrete structure with a weir gate to regulate flow between the lagoons and to control water levels. The lagoon embankment was also reconstructed and bentonite was placed around the transfer structure and piping to minimize leakage. The 8-inch transfer piping appears to have adequate hydraulic capacity to transmit the projected maximum month flows between lagoons.

Effluent from each lagoon flows through an 8-inch gravity sewer line to a chlorine contact basin for disinfection (see **Figure 10**). There is an 8-inch gate valve on each of the outlet lines for isolation of the lagoons from the contact basin. The City reports these gate valves are damaged and cannot be closed. The existing 8-inch gravity outlet lines appear to have adequate hydraulic capacity to transmit the projected maximum month flows and to supply the irrigation pumps.

4.2.8 Disinfection System

Table 22 summarizes the estimated sodium hypochlorite dosing rate for the minimum and maximum discharge rates of the irrigation pump, assuming a maximum chlorine dose of 8 mg/L. The table also summarizes the estimated chlorine contact time at the minimum and maximum pump rates and lagoon water surface elevations.

Table 22 – Existing Sodium Hypochlorite Disinfection System

Parameter	Values at Minimum Irrigation Pump Discharge Rate	Values at Maximum Irrigation Pump Discharge Rate
Irrigation Pump Discharge Rate	129 gpm	164 gpm
Sodium Hypochlorite Injection Rate [A]	0.62 gph	0.79 gph
Chlorine Contact Time at Max. Water Surface Elevation [B]	44 min	35 min
Chlorine Contact Time at Min. Water Surface Elevation [C]	20 min	16 min

A. Assuming a chlorine concentration of 8.0 mg/L and a liquid sodium hypochlorite concentration of 10 percent.
B. Approximate chlorine contact volume of 5,710 gallons.
C. Approximate chlorine contact volume of 2,640 gallons.

It appears the existing sodium hypochlorite metering pump is adequate since it is capable of pumping a maximum flow of 1.0 gph. Additionally, Ten State Standards recommend 15 minutes of contact time at the maximum rate of pumping. It appears the existing contact basin is adequately sized to comply with this recommendation, provided the irrigation pump is not replaced with a larger unit in the future.

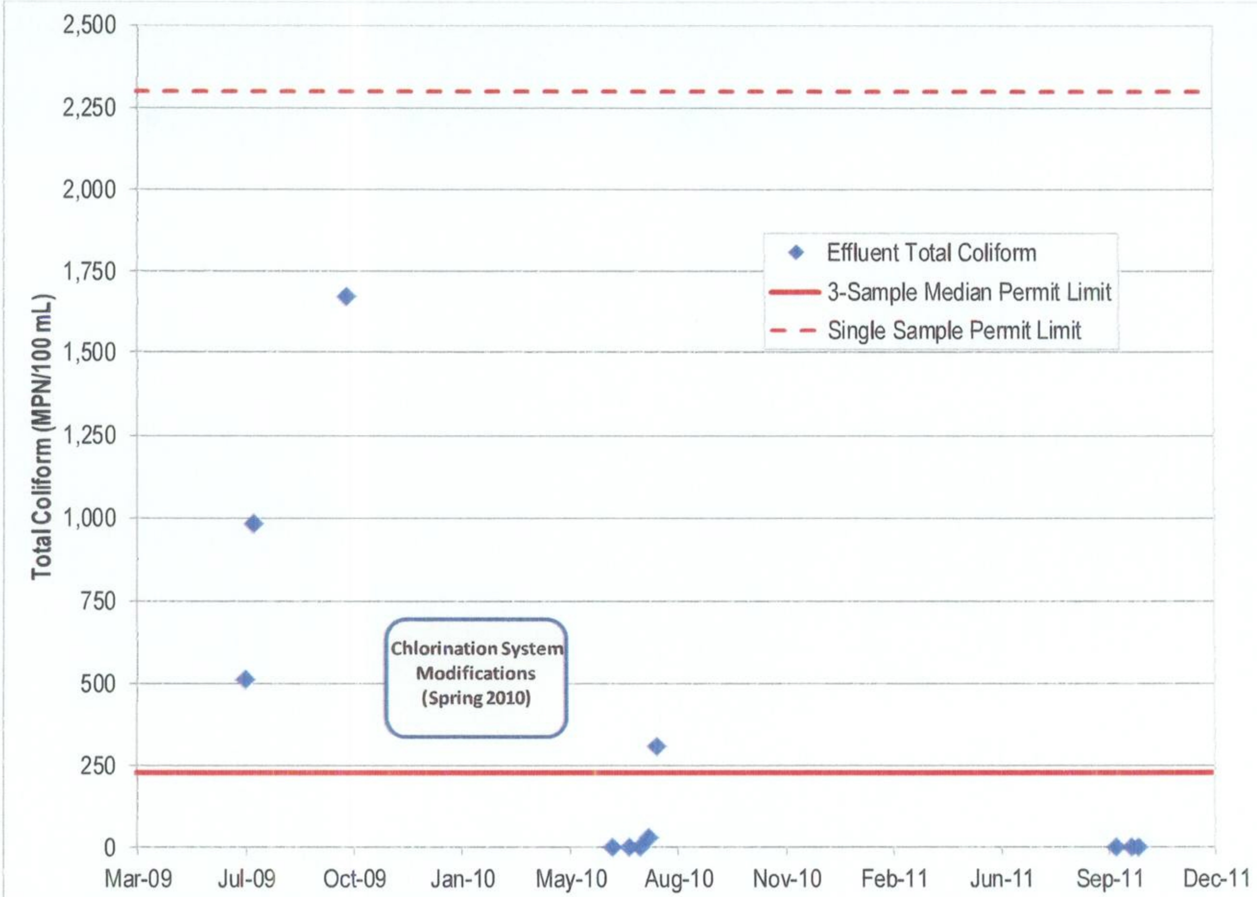
During the 2009 growing season, the City began to exceed their Reuse Permit limits for total coliform. To remedy the problem, they moved the stainless steel sodium hypochlorite injection quill further upstream of the chlorine contact basin prior to the 2010 growing season to improve mixing and increase contact time. As shown in **Figure 25**, this change has improved the disinfection system performance and allowed the City to generally meet the Reuse Permit limits for total coliform.

4.3 WASTEWATER LAND APPLICATION (REUSE) SYSTEM

4.3.1 Irrigation System

The City reports the effluent irrigation pump (rebuilt in 2011) and effluent flow meter are in relatively good condition. Based on readings from the effluent flow meter, the average discharge rate for the pump in 2011 was 129 gpm (185,950 gpd) and the maximum discharge rate was 164 gpm (235,900 gpd). Both of these values are relatively close to the design flow rate of 140 gpm, which is an indication that the pump is operating near its design point and that the flow meter is likely working correctly. Based on a water balance conducted for the land application site (see Section 4.3.2), it appears the irrigation pump has sufficient capacity for the 20-year planning period.

Figure 25 – Historical Effluent Total Coliform



Effluent is discharged through a 4-inch pressure main to the land application site for irrigation through handlines. The City reports the irrigation piping and handlines are in relatively good condition A 4-inch pressure main has adequate hydraulic capacity to deliver the effluent flow (e.g., 129 to 164 gpm) within typically accepted velocities and head loss (e.g., less than 5 fps).

4.3.2 Lagoon Storage and Irrigated Area Requirements

The City currently discharges its effluent to a 13-acre reuse site during the growing season (April through October). They are required to store the effluent in the lagoons during the non-growing season (November through March).

The amount of lagoon storage and irrigated acreage required for slow-rate land application is governed by the type of crop and hydraulic, nutrient (i.e., nitrogen and phosphorus) and/or organic loading rates. The growing season hydraulic application rate should be commensurate to the seasonal crop irrigation water requirement (IWR). Nutrients should be applied to the reuse site at a rate that is comparable to the uptake rate of the crop to minimize potential impacts on the soil and groundwater. The City's current reuse permit allows a nutrient loading rate of up to 150 percent of the crop uptake.

To evaluate the lagoon storage volume and irrigated acreage requirements, an iterative water and nutrient balance was conducted around the lagoons and reuse site under current and future loading conditions. The water balance accounted for the crop IWR, allowable nutrient and organic loadings, and gains or losses in the system, including precipitation, evaporation and seepage. Assumptions used in the water balance include:

- Current and future average day flows and loads were used.
- Alfalfa would be grown on the reuse site.
- All 13-acres are available for irrigation.
- Only the effective storage volume in the lagoons is available for non-growing season storage.
- Seepage losses from the lagoons were based on recent seepage test results (see **Table 21**).
- No effluent will be discharged to the land application site during the non-growing season, but would be stored in the lagoons.

Table 23 summarizes the water and nutrient balance results at current and future loadings for the lagoons and land application site (see **Appendix C** for detailed calculations).

Table 23 – Reuse System Water and Nutrient Balance Results

Parameter	Units	Existing Loading Conditions	Projected Loading Conditions
Hydraulic Loading			
Growing Season Annual Average	mgal	9.8	15.1
Allowable Loading	mgal	17.5	17.5
Phosphorus Loading			
Growing Season Annual Average	lbs/ac	18	28
Allowable Loading	lbs/ac	32	32
Nitrogen Loading			
Growing Season Annual Average	lbs/ac	72	112
Allowable Loading	lbs/ac	335	335
Total Irrigated Acreage Required	acres	7.4	11.3
Existing Irrigated Acreage	acres	13.0	13.0
Additional Irrigated Acreage Required	acres	0.0	0.0
Total Lagoon Volume Required	mgal	5.4	7.8
Existing Effective Lagoon Storage Volume	mgal	7.3	7.3
Additional Lagoon Storage Volume Required	mgal	0.0	0.5

As shown in the table, the existing and projected hydraulic and nutrient loadings are within the allowable permitted values. Additionally, the existing irrigated land application acreage is sufficient to accommodate the projected flows and loads over the 20-year planning period. It appears additional non-growing season lagoon storage volume may be required within approximately 15 to 20 years unless I&I to the system is reduced by approximately 10 percent.

4.3.3 Buffer Zones

The City's existing Reuse Permit requires the following buffer zones for the land application site:

- 500 feet to inhabited dwellings
- 300 feet to areas accessible by the public

- 100 feet to permanent and intermittent surface water
- 50 feet to irrigation ditches and canals
- 500 feet to private water supply wells
- 1,000 feet to public water supply wells

The nearest inhabited dwelling is more than 2,000 feet south of the land application site. The closest building to the land application site is an aircraft hangar/farm equipment storage facility located approximately 1,100 feet to the south. The closest municipal drinking water well is located approximately 4,400 feet west of the lagoons and there are no known private water supply wells within 500 feet of the site. The City reports the land application site does not border any sources of surface water (other than the lagoons) or irrigation canals. The land application site does border the dirt access road to the treatment facility with a buffer distance of only a few feet. However, two different gates restrict public access to this road. As such, it appears the land application site meets all of the buffer zone distances required in the permit. In addition, the land application site is enclosed with a fence and warning/entry prohibited signs are posted to keep the public out.

Chapter 5

Development and Screening of Improvement Alternatives

5.0 DEVELOPMENT AND SCREENING OF IMPROVEMENT ALTERNATIVES

5.1 WASTEWATER COLLECTION SYSTEM

5.1.1 "Do-Nothing" Option

Under this alternative, no action would be taken to replace and/or rehabilitate the existing gravity or pressure sewer collection mains. The pipes would be left in place and continue to operate under the existing and projected flow conditions. This option is likely acceptable for pipes, manholes, valves, and other appurtenances that are in good condition and are not broken or deteriorated. However, it does not address the issues outlined in Chapter 4 for excessive I&I, contamination of groundwater through sewer exfiltration, leaky and/or cracked joints and service lateral connections, potentially deteriorated mains, potentially faulty air-vacuum valves on the pressure sewer, reduced capacity due to I&I in the gravity mains and solids deposition in the pressure mains. This alternative also does not correct for pipes with potential grade, depth, or alignment problems.

5.1.2 Gravity Sewer Mains

As previously discussed, excessive I&I is the primary concern with the gravity collection system. A reduction in I&I will extend the useful life of the treatment lagoons and reuse site, increase the available hydraulic capacity in the collection system, and reduce operation and maintenance requirements. However, as noted in Chapter 4, the City has only cleaned and video inspected a small portion of the gravity sewer mains. As a result, there is generally insufficient information available to identify specific improvements to reduce I&I, correct deteriorated piping, and/or remedy grade, depth or alignment issues. As such, it is recommended the City clean and video inspect the entire gravity collection system. This will allow them to:

- Identify the condition of the gravity sewer mains and areas of high I&I.
- Prioritize the mains that need replaced and/or rehabilitated so that the most problematic areas (i.e., "low hanging fruit") can be addressed as funding becomes available.
- Identify the appropriate method for replacing and/or rehabilitating the mains (e.g., open trench, cured-in-place-pipe [CIPP], pipe bursting, etc.).
- Provide a systematic approach to replacing and/or rehabilitating the gravity sewer mains.

An opinion of the probable cost in 2012 dollars to clean and video inspect the entire gravity collection system is shown in **Table 24** (see **Appendix D** for detailed costs). Costs are included for engineering support to review and prioritize the lines for repair and to identify the recommended method for replacement and/or rehabilitation. The analysis should also compare (1) reducing I&I through collection system improvements to (2) increasing the capacity of the lift station and wastewater treatment facilities to handle I&I. At some point, the costs for collection system repairs may not justify the reduction in I&I; funding may be better spent on increasing the capacity of the lift station and treatment facilities to accommodate some I&I.

Table 24 – Opinion of Probable Capital Costs to Clean and Video Inspect Gravity Sewer Mains

Item	Capital Costs
Clean and Video Inspect Gravity Collection System (17,500 LF)	$14,000
Sub-Total Construction Costs	**$14,000**
Contractor Mob/Demob, Bonding, Insurance, Admin (10%)	$1,400
Contingencies (25%)	$3,500
Total Construction Costs	**$18,900**
Engineering & Construction Management (20%)	$3,800
Total Project Capital Costs	***$22,700***

Since specific gravity collection system improvements cannot be identified with the available information, it was assumed that approximately 25 percent of the existing gravity sewer mains would need to be replaced via open trench. This is simply an assumption that may allow the City to address some of the "low-hanging" improvements to reduce I&I. Once the video inspection and analysis is complete, a more detailed list of improvements and an opinion of probable capital costs for implementing specific gravity collection system improvements can be prepared. An opinion of the probable cost in 2012 dollars to replace approximately 25 percent of the existing gravity collection system mains is shown in **Table 25** (see **Appendix D** for detailed costs).

Table 25 – Opinion of Probable Capital Costs to Replace Approximately 25 Percent of Gravity Sewer Mains

Item	Capital Costs
Open Trench Replacement of Approximately 25% of Gravity Sewer Mains (4,400 LF)	$388,700
Sub-Total Construction Costs	**$388,700**
Contractor Mob/Demob, Bonding, Insurance, Admin (10%)	$38,900
Dewatering (10%)	$38,900
Contingencies (25%)	$97,200
Total Construction Costs	**$563,700**
Engineering & Construction Management (20%)	$112,700
Administration & Funding (5%)	$28,200
Inflation (4% for 2 Years)	$45,100
Total Project Capital Costs	***$749,700***

As discussed in Chapter 4, the City would also like to improve the 8-inch trunk line which enters the lift station wet-well from the north. This trunk line enters the wet-well below the water surface and occasionally surcharges when water levels rise in the wet-well. This situation may be remedied by simply changing the pump on/off set-points for the wet-well such that surcharge conditions do not exist. This would also require replacing the existing lift station pumps with larger capacity units, as discussed in Section 5.2.2. However, based on discussions with City staff, it may also be necessary to replace one section of this trunk line to the north (e.g., open trench) and to reconstruct it at a higher elevation (e.g., shallower bury depth). It needs to be verified that existing service connections could still enter the trunk line if it is raised to a higher elevation.

An opinion of the probable capital costs in 2012 dollars to reconstruct one section of this gravity trunk line is shown in **Table 26** (see **Appendix D** for detailed costs).

Table 26 – Opinion of Probable Capital Costs to Replace Gravity Sewer Trunk Line North of Lift Station

Item	Capital Costs
Open Trench Replace 480 LF of 8-Inch Gravity Sewer	$44,900
Sub-Total Construction Costs	**$44,900**
Contractor Mob/Demob, Bonding, Insurance, Admin (10%)	$4,500
Dewatering (10%)	$4,500
Contingencies (25%)	$11,200
Total Construction Costs	**$65,100**
Engineering & Construction Management (20%)	$13,000
Administration & Funding (5%)	$3,300
Inflation (4% for 2 Years)	$5,200
Total Project Capital Costs	***$86,600***

Annual operation and maintenance (O&M) costs associated with the gravity sewer lines are not expected to change considerably. However, it is recommended that the City consider adding annual budget line items for capital improvements ($10,000 to $20,000) and routine cleaning and video inspection ($2,000 to $3,000) of the sewer lines. Additionally, as part of their program to reduce I&I, the City should continue to raise manholes and use solid lids in locations where stormwater inflow is a concern.

5.1.3 Pressure Sewer Mains

As discussed in Chapter 4, it is recommended that the City clean the existing 6-inch pressure main and locate, inspect, exercise, and replace, if necessary, the air-vacuum valves. Cleaning can be achieved using a pipe "pig" and/or by installing pressure cleanouts on the line for hydraulic jetting. A wye connection may need to be installed downstream of the lift station to facilitate cleaning of the force main. For planning purposes, it is assumed that all three of the air-vacuum valves will need to be replaced. An opinion of the probable capital costs in 2012 dollars to clean the pressure sewer main and replace the air-vacuum valves is shown in **Table 27** (see **Appendix D** for detailed costs).

Table 27 – Opinion of Probable Capital Costs to Clean Pressure Sewer Main and Replace Air-Vacuum Valves

Item	Capital Costs
Clean Pressure Sewer Main (4,600 LF)	$12,200
Replace Air-Vacuum Valves/Vaults (3 EA)	$33,000
Bypass Pumping	$2,500
Sub-Total Construction Costs	**$52,700**
Contractor Mob/Demob, Bonding, Insurance, Admin (10%)	$5,300
Dewatering (10%)	$5,300
Contingencies (25%)	$13,200
Total Construction Costs	**$76,500**
Engineering & Construction Management (20%)	$15,300
Administration & Funding (5%)	$3,800
Inflation (4% for 2 Years)	$6,100
Total Project Capital Costs	***$101,700***

5.2 LIFT STATION

5.2.1 "Do-Nothing" Option

This option consists of the City taking no action to improve the existing lift station. However, this does not appear to be a viable option for the following reasons:

- The existing pumps do not have sufficient pumping capacity to handle the existing and projected peak hour flows in accordance with the Idaho Administrative Rules (IDAPA 58.01.16.440.02.c.i). This has resulted in sewer overflows of the lift station in the past, presenting a public and environmental health and safety concern.
- The existing pumps are not operating near their design point and are operating at a lower efficiency, resulting in higher power costs to the City.
- The lift station is more than 35 years old and is approaching its useful design life. Most of the exposed metal and concrete is heavily corroded and in need of replacement.
- The existing lift station does not have a flow meter in accordance with the Idaho Administrative Rules (IDAPA 58.01.16.440.02.h), nor is there an influent flow measurement device at the treatment lagoons.
- The City would like to install a permanent back-up power supply for the lift station in accordance with the Idaho Administrative Rules (IDAPA 58.01.16.07.b).
- The existing bar racks for screening of influent debris and solids are in poor condition and in need of replacement. Additionally, the gravity line entering the wet-well from the north is below the water surface, allowing debris, rags and other floatable items to bypass the bar rack. As a result, the City reports they also have problems with rags and debris clogging the pumps.

Due to these concerns, the "Do-Nothing" option is not considered a long-term, reliable solution for the lift station. As a result, it was not considered any further in this report.

5.2.2 Replace Existing Lift Station

This alternative contemplates replacing the existing lift station due to concerns regarding its condition, capacity, and long-term reliability. This option includes the following components:

- Abandon, remove, and dispose of the existing lift station and its components.
- Construct a new wet-well and/or dry-well.
- At a minimum, install at least two pumps that each has the capacity to discharge the projected peak hour influent flow (IDAPA 58.01.16.440.02.c.i). In Albion's case, it is recommended that each pump be capable of discharging 180 gpm, which is in excess of the projected peak hour flow (114 gpm). This pumping rate is recommended to maintain a minimum cleansing velocity of 2 fps in the 6-inch pressure main. At 180 gpm, the estimated total dynamic head is approximately 42 feet at the maximum water level in the lagoons. This equates to an approximate 5-horsepower pump. The pump curve should be selected to account for the varying head conditions due to the lagoon water level fluctuations.
- Install new mechanical piping and fittings.
- Install new plug and check valves within a separate vault.

- Install a magnetic flow meter within a separate vault.
- Install a permanent generator to provide a back-up power supply to the lift station (IDAPA 58.01.16.07.b). It is recommended the City purchase the one-half acre parcel of ground south of the existing lift station to locate the generator and possibly the new lift station. The City also desires to construct a 1,000-gallon diesel fuel storage tank and a metal building in which to house the generator.
- Install pump electrical and control panels in the metal building.
- Install a jib crane for pump removal.

There are numerous configurations commonly used for sewer lift stations. Based on discussions with the City, they would like to consider the following configurations for the new lift station:

- Duplex submersible
- Triplex submersible
- Duplex wet/dry-well

Table 28 provides a comparison of these lift station configurations (see **Appendix E** for example drawings of each configuration).

As shown in the **Table 28**, there are several types of pumps commonly used in this application, including "non-clog" pumps, chopper/grinder pumps, and vortex recessed impeller pumps. They have varying efficiencies and costs, but are all generally suitable for wastewater pumping applications. Chopper/grinder pumps shred rags and debris and tend to clog less frequently than other pumps. Each of these pump types can be arranged in a submersible or wet/dry-well configuration.

An opinion of the probable capital costs in 2012 dollars for each of these three lift station configurations is shown in **Table 29** (see **Appendix D** for detailed costs).

Operation and maintenance costs associated with a new sewer lift station are assumed to remain approximately the same with the recommended improvements. Although power costs may increase slightly due to the larger pumps, the higher efficiency of the new pumps will result in lower power costs. Additionally, a more reliable lift station with sufficient capacity will also reduce maintenance costs.

Table 28 – Comparison of Lift Station Configurations

Parameter	Duplex Submersible	Triplex Submersible	Duplex Wet/Dry Well
Configuration	• Influent wastewater discharges to wet-well • Pumps, motors, rails, piping, and level controls located in the wet-well and submerged • Manifold piping, gate and check valves located in a separate vault • Flow meter located in a separate vault • Electrical/control panels located above grade near the vaults • Jib crane or overhead crane can be used for pump removal	• Influent wastewater discharges to wet-well • Pumps, motors, rails, piping and level controls located in the wet-well and submerged • Manifold piping, gate and check valves located in a separate vault • Flow meter located in a separate vault • Electrical/control panels located above grade near the vaults • Jib crane or overhead crane can be used for pump removal	• Influent wastewater discharges to wet-well • Suction piping and level controls located in wet-well • Pumps, motors, rails, piping, and valves located in a separate dry-well (no wastewater enters the dry-well) • Flow meter located either in dry-well or in a separate vault • Electrical/control panels located either in dry-well or above grade near the vaults • Jib crane or overhead crane can be used for pump removal
Pump Type and Operation	• Non-clog submersible • Chopper/grinder submersible • Vortex, recessed impeller submersible • 1 duty, 1 standby • Lead/lag operation	• Non-clog submersible • Chopper/grinder submersible • Vortex, recessed impeller submersible • 1 duty, 2 standby • Lead/lag operation	• Non-clog centrifugal • Chopper/grinder • Vortex, recessed impeller • 1 duty, 1 standby • Lead/lag operation
Pump Size	• 180 gpm each at 42 ft TDH • Approximately 5 horsepower	• 180 gpm each at 42 ft TDH • Approximately 5 horsepower	• 180 gpm each at 42 ft TDH • Approximately 5 horsepower
Vault Sizes and Types	• Wet-well – 6 to 8 ft diameter • Valve vault – 6 to 8 ft diameter • Flow meter vault – 4 ft diameter • Vaults can be site-built reinforced concrete or prefabricated reinforced concrete or fiberglass	• Wet-well – 10 to 12 ft diameter • Valve vault – 8 to 10 ft diameter • Flow meter vault – 4 ft diameter • Vaults can be site-built reinforced concrete or prefabricated reinforced concrete or fiberglass	• Wet-well – 6 to 8 ft diameter • Dry-well – 8 to 12 ft diameter • Flow meter vault – 4 ft diameter • Vaults can be site-built reinforced concrete or prefabricated reinforced concrete or fiberglass
Footprint	• Smaller	• Larger	• Larger
Efficiency	• Lowest	• Lowest	• Highest
Advantages	• Most common configuration for this size of lift station • Similar to existing lift station • Flooding of lift station less of a concern • Less ancillary piping and equipment	• High level of redundancy • Flooding of lift station less of a concern	• Pumps and motors easier to access, inspect, and maintain • Common configuration
Disadvantages	• Pumps and motors less easily accessed, inspected, and maintained	• Pumps and motors less easily accessed, inspected and maintained • Uncommon configuration for this size of lift station • Redundancy of third pump increases costs • More ancillary piping and equipment	• Flooding of dry-well is a concern • More ancillary piping and equipment • Confined space requirements for dry-well

Table 29 – Opinion of Probable Capital Costs to Replace Existing Lift Station [A]

Item	Duplex Submersible Lift Station Capital Costs	Triplex Submersible Lift Station Capital Costs	Duplex Wet/Dry Well Lift Station Capital Costs
Site Work and Remove Existing Lift Station	$9,300	$11,500	$12,800
Wastewater Structures/Vaults	$31,500	$45,500	$52,500
Mechanical Piping/Equipment	$51,900	$71,200	$54,400
Electrical/Controls	$15,000	$20,000	$17,500
Back-Up Generator	$20,000	$20,000	$20,000
1,000 Gallon Diesel Fuel Storage Tank	$17,900	$17,900	$17,900
Building for Generator and Panels	$42,000	$42,000	$42,000
Sub-Total Construction Costs	**$187,600**	**$228,100**	**$217,100**
Contractor Mob/Demob, Bonding, Insur., Admin (10%)	$18,800	$22,800	$21,700
Dewatering (10%)	$18,800	$22,800	$21,700
Contingencies (25%)	$46,900	$57,000	$54,300
Total Construction Costs	**$272,100**	**$330,700**	**$314,800**
Engineering & Construction Management (20%)	$54,400	$66,100	$63,000
Administration & Funding (5%)	$13,600	$16,500	$15,700
Inflation (4% for 2 Years)	$21,800	$26,500	$25,200
Total Project Capital Costs	***$361,900***	***$439,800***	***$418,700***

A. Costs are not included for land acquisition.

5.3 WASTEWATER TREATMENT FACILITIES

5.3.1 "Do Nothing" Option

In general, it appears the existing wastewater treatment lagoons and land application site are adequate for the 20-year planning period. There are some relatively minor improvements that could be constructed at the City's discretion to optimize performance of the system (refer to the following section). Additionally, continued reduction in I&I to the collection system will provide additional hydraulic capacity for population growth and extend the useful life of the treatment facilities.

5.3.2 Optimize Existing Treatment Facilities

Under this option, the City would upgrade the existing treatment lagoons and land application site and continue to use them for wastewater treatment and effluent disposal. As previously noted, the existing treatment facilities appear to be in relatively good condition and have sufficient capacity for the planning period. However, there are a few improvements the City would like to consider under this alternative to optimize the performance of the treatment facilities. The recommended upgrades have been split into Phase 1 improvements that should be considered in the "near-term" and Phase 2 improvements that may be required in the "long-term":

Phase 1 "Near-Term" Improvements

- As discussed in Chapter 4, the gate valves on the 6-inch pressure main delivering the influent to the lagoons are buried approximately 8- to 10-feet below the ground surface within the lagoon embankments. Because of their bury depth, the City reports it is difficult to operate the valves and to know which position they are in. The City would like to reconstruct the pressure mains and isolation valves so they are closer to the surface of the lagoon embankments.

- In conjunction with reconstructing the influent force mains, it is recommended that the City consider a headworks structure to provide coarse screening of the influent. Screening will minimize the amount of debris and solids entering and filling the lagoons, extending the time before biosolids removal is required. The screening structure would likely consist of an open concrete box with two channels: a primary channel with a manually cleaned coarse bar rack and a bypass channel. Influent would normally pass through the coarse bar rack prior to gravity flowing to the lagoons. Additional gravity piping from the screening structure to the lagoon inlets will also need to be constructed. Slide gates would control the direction of flow to the lagoons. If the bar rack happens to clog, an overflow weir would direct influent to the bypass channel. Influent samples could also be collected at the screening structure rather than at the lift station.

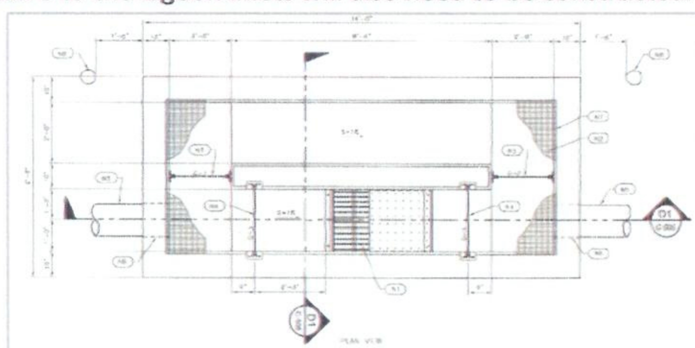


Figure 26 – Example Coarse Screening Structure

- The City reports the 8-inch gate valves on the lagoon outlet lines are damaged and cannot be closed. It is recommended these valves be replaced.

Phase 2 "Long-Term" Improvements

- As shown in **Figure 24**, it appears the projected organic loading rate to the lagoons may exceed the upper recommended design value during the planning period. It is recommended the City continue to monitor the treatment performance of the lagoons and for obnoxious odors. If reduced treatment performance and/or odors become a problem in the future, the City may need to consider installing two 5-horsepower mechanical surface aerators in Cell #1. This analysis is based on limited influent flow and sampling data. A more accurate determination of when to install aerators can be made if influent flow monitoring and sampling is implemented.

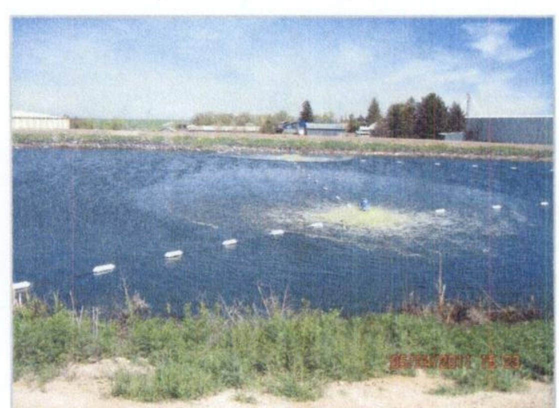


Figure 27 – Example Lagoon Aerators

An opinion of the probable capital costs in 2012 dollars for the Phase 1 "Near-Term" improvements is shown in **Table 30** (see **Appendix D** for detailed costs). Costs for the Phase 2 "Long-Term" improvements were not developed as part of this Facilities Plan since it is uncertain when or if they will need to be constructed.

Table 30 – Opinion of Probable Capital Costs to Optimize Existing Treatment Facilities

Item	Capital Costs
Reconstruct 6" Inlet Force Mains and Valves	$16,400
Coarse Screening Structure and 8" Gravity Inlets to Lagoons	$84,500
Replace 8" Outlet Valves	$5,300
Sub-Total Construction Costs	**$106,200**
Contractor Mob/Demob, Bonding, Insurance, Admin (10%)	$10,600
Contingencies (25%)	$26,600
Total Construction Costs	**$143,400**
Engineering & Construction Management (20%)	$28,700
Administration & Funding (5%)	$7,200
Inflation (4% for 2 Years)	$11,500
Total Project Capital Costs	***$190,800***

Operation and maintenance costs associated with the treatment facilities are anticipated to remain the same as existing values for the Phase 1 "Near-Term" improvements. If surface aerators are installed in the lagoons at some point in the future, additional power costs will be incurred.

5.3.3 Mechanical Treatment Facility

Under this option, wastewater from the City would be treated with a mechanical treatment plant, such as conventional activated sludge, oxidation ditch, or membrane bioreactor. A mechanical plant will provide a higher degree of wastewater treatment than a lagoon system. Treated effluent from the plan could continue to be discharged to the land application site or through an alternative disposal method, such as surface water discharge or other reuse systems (e.g., rapid infiltration basins). Continued discharge to the land application site or other reuse systems would require winter storage in the existing or new lagoons. Discharge to the land application site or other reuse system would require a Reuse Permit from IDEQ, while discharge to surface water would require a National Pollutant Discharge Elimination System (NPDES) Permit from EPA.

The capital costs to construct a mechanical treatment plant and to potentially reconfigure the existing lagoons for winter storage would be significantly higher than optimizing the existing lagoons and land application site. The O&M requirements and costs would also be significantly greater and more complex for a mechanical plant than for a lagoon treatment system. Additionally, the permitting requirements would likely be more challenging for a mechanical treatment plant, particularly for surface water discharge under a NPDES Permit. As such, it appears that a mechanical treatment plant is not a reasonable and economical approach for meeting the wastewater needs of the community.

5.3.4 Regional Treatment Facility

Wastewater from the City of Albion could potentially be combined with wastewater from one or more surrounding communities at a centralized location for regional treatment. Communities located near Albion that may potentially be involved in regional treatment include the City of Declo (8 miles to the north) and City of Malta (17 miles to the southeast). Each of these communities has existing wastewater treatment facilities.

Wastewater from one or more of the communities could be pumped to one of the existing treatment plants or to a new regional treatment plant. This would require pump stations and transmission mains from the participating communities to the regional treatment plant, resulting in extensive capital costs and increased O&M requirements.

The existing treatment facilities within these communities are generally adequate to handle their own loads, but would likely require extensive upgrades to accommodate flows and loads from other communities. It is more economical for the City of Albion to continue upgrading and operating their own treatment facilities rather than to construct transmission infrastructure and upgrades at a neighboring treatment plant or to construct a new regional treatment plant.

There may also be administrative difficulties associated with a regional plant, such as retention of ownership in the effluent, equitable sharing of costs, and the desired quality of the effluent. Due to concerns regarding participation by neighboring communities and cost-effectiveness, this alternative was not considered any further in this report.

Chapter 6

Implementation of Wastewater System Improvements

6.0 IMPLEMENTATION OF WASTEWATER SYSTEM IMPROVEMENTS

6.1 RECOMMENDED WASTEWATER FACILITY IMPROVEMENTS

This Facilities Plan identifies several areas of concern related to the City's wastewater collection and treatment infrastructure, including:

- Infiltration and inflow into the collection system is more than twice as much as would be expected for a city the size of Albion.
- The pumps within the existing sewer lift station are undersized for pumping the influent flow to the treatment lagoons, particularly during high I&I flow periods. As a result, the wet-well has overflowed on several occasions in the past. Additionally, there is no back-up power or flow monitoring and the lift station is over 35 years old, resulting in many components that are corroded and in need of replacement. The lift station pumps also have occasional problems with ragging and clogging.
- The existing pressure main from the lift station to the lagoons has not been cleaned since its construction and the condition of the air-vacuum valves are unknown.
- The wastewater treatment lagoons and land application system are generally in adequate condition and have sufficient capacity. However, there are several minor improvements needed to optimize their performance (e.g., inlet and outlet valve replacement, influent screening, etc.).

Based on the analysis contained in this report, the City would like to consider a phased approach for implementation of the wastewater system improvements. Highest priority was given to those upgrades necessary to meet regulatory requirements and to protect the health, safety, and welfare of the public and environment. The selected improvements will also provide the greatest benefit to the City in a cost-effective manner based on the needs of the community. A phased approach will likely minimize the initial capital costs and distribute the costs reasonably over time. Phasing of the improvements will also allow the City to implement them on an "as-needed" basis to meet future growth or as driven by regulatory requirements. Following is the phasing plan for the wastewater system improvements identified in this report:

Phase 1 Improvements (Highest Priority)

1. Replace the existing lift station with a duplex submersible lift station, including the associated piping and fittings, a valve vault, a flow meter and vault, electrical panels and controls, back-up generator, diesel fuel storage tank, building, and jib crane for removal. It is also recommended that the City purchase the land adjacent to the existing lift station.
2. Reconstruct the existing 8-inch gravity trunk line that enters the lift station wet-well from the north.

Phase 2 Improvements

1. Clean and video inspect the entire gravity sewer collection system. Once the video inspection and analysis is complete, an opinion of probable capital costs for implementing specific gravity collection system improvements can be prepared, as needed.

2. Replace approximately 25 percent of the existing gravity sewer mains via open trenching.
3. Clean the existing 6-inch pressure main and locate and replace the air-vacuum valves.

Phase 3 Improvements

1. Construct the improvements to the existing treatment lagoons to optimize their performance, including:
    a. Reconstruct the 6-inch force mains and valves at the lagoons.
    b. Construct a coarse screening structure and new 8-inch gravity mains to the lagoon inlets.
    c. Replace the 8-inch valves on the lagoon outlet lines.

6.2 COST ESTIMATES

An opinion of the overall probable capital costs in 2012 dollars for the recommended improvements is summarized in **Table 31**. As previously discussed, it is anticipated that no additional annual O&M costs will result from these improvements.

Table 31 – Opinion of Probable Capital Costs for the Recommended Improvements

Item	Capital Costs
Phase 1 Improvements (Highest Priority)	
Replace Lift Station with Duplex Submersible Lift Station	$361,900
Reconstruct 8" Gravity Trunk Line North of Lift Station	$86,600
Sub-Total Phase 1 Improvements	**$448,500**
Phase 2 Improvements	
Clean and Video Inspect Gravity Collection System	$22,700
Replace Approximately 25% of Gravity Sewer Mains	$749,700
Clean 6" Pressure Main and Replace Air-Vacuum Valves	$101,700
Sub-Total Phase 2 Improvements	**$874,100**
Phase 3 Improvements	
Construct Lagoon Optimization Improvements	$190,800
Sub-Total Phase 3 Improvements	**$190,800**
Total Project Costs	***$1,513,400***

6.3 MONTHLY USER CHARGE RATE ANALYSIS

Single-family residential connections (e.g., one equivalent residential unit, or ERU) are currently charged a monthly user rate of $30.00 per month. Other commercial entities within the City are billed different monthly rates based on their estimated sewer discharge.

The costs for any improvements should be distributed equitably between the residential and commercial users based on their respective loadings to the wastewater facilities. The cost distribution may be accomplished using an ERU analysis. One ERU typically represents the equivalent average amount of wastewater expected to be generated from a single-family residence. The ERU's or commercial entities were estimated based on a ratio of their monthly user rate to the single-family residential base rate. A summary of the current monthly user rates and estimated of the number of ERU's is summarized in **Table 32**, based on billing information provided by the City.

Table 32 – Monthly Sewer Rates and ERU's

Entity	Monthly User Rate	ERU [A]	Quantity	Total ERUs
Single-Family Residence	$30.00	1.0	113	113
Business	$50.00	1.7	5	8.3
Church	$50.00	1.7	2	3.3
Ellinar Enterprises	$50.00	1.7	1	1.7
LDS Church	$50.00	1.7	1	1.7
Marsh Creek Inn	$50.00	1.7	1	1.7
Post Office	$50.00	1.7	1	1.7
School	$50.00	1.7	1	1.7
Small Business	$50.00	1.7	3	5.0
			TOTAL ERUs	***138***

A. Number of commercial ERUs estimated by dividing the monthly user rate for that entity by the base single-family rate of $15 per month.

Changes to the monthly user rates were estimated for the Phase 1 Improvements. For comparison purposes, two financing scenarios were considered for the proposed improvements. The two scenarios were based on the source and amount of funding procured for the project:

1. Scenario 1 – No grant funding would be secured and the project would be funded entirely through low-interest loans.

2. Scenario 2 – Approximately $200,000 of the project will be funded through City reserve funds and the remaining portion would be funded through low interest loans.

There may be other project financing combinations that should be explored by the City. These two scenarios are simply used to illustrate possible changes to the monthly user rates for the Phase 1 Improvements. **Table 33** summarizes the results of the user charge rate analysis for the two financing alternatives. Additional revisions to the monthly user rates may also be necessary for the Phase 2 and 3 Improvements as they are implemented in the future.

Table 33 – Monthly User Rate Analysis for the Phase 1 Improvements

Parameter	Financing Scenario 1	Financing Scenario 2
Capital Costs		
Total Capital Costs	$448,500	$448,500
Grant/City Cash Amount	$0	$200,000
Loan Amount	$448,500	$248,500
Annual Costs		
Existing Annual Sewer Bond Repayment	$8,050	$8,050
New Annual Loan Repayment	$31,557	$17,485
New Annual O&M Costs	$0	$0
Loan Reserve	$3,160	$1,750
Monthly User Rate		
Total Annual Costs	$42,767	$27,285
ERUs	138	138
Additional Monthly User Rate [C]	**$25.83**	**$16.48**
Existing Monthly User Rate [C]	$30.00	$30.00
Total New Monthly User Rate [C]	***$55.83***	***$46.48***

A. Based on a 20 year loan at 3.5%.
B. Based on a reserve of 10% of the annual loan repayment over 10 years.
C. Monthly cost per ERU.

6.4 PROJECT FINANCING

There are several potential sources of funding available to the City to assist in financing the improvements, including:

- IDEQ State Revolving Fund (SRF) loan.
- U.S. Department of Agriculture Rural Development Agency (RD) loans and grants.
- Department of Commerce Idaho Community Development Block Grant Program (ICDBG).
- U.S. Department of Commerce Economic Development Administration (EDA) grants.
- EPA State and Tribal Assistance Grants (STAG).
- U.S. Army Corps of Engineers Section 595 Grants.
- Congressional appropriations.
- New user capacity fees or impact fees.
- City reserve funds.

The Idaho Department Environmental Quality has funds available through their SRF loan program. This program provides below market rate interest loans to Idaho communities to build new, or repair, existing wastewater facilities. The loan term is typically 20 years; however, some applicants may qualify as disadvantaged and be eligible for reduced loan terms. The funding is derived from an appropriation from the EPA (80%) and a 20% match from the Water Pollution Control Account.

USDA Rural Development makes loans and grants to public bodies and non-profit organizations in rural areas to construct or improve community facilities that are modest in size, cost and design. Water and Waste Disposal (WWD) Loans and Grants may be used to construct, repair, improve, expand or otherwise modify rural wastewater facilities; pay necessary fees and costs associated with the project; or finance facilities in conjunction with funds from other agencies or those provided by the applicant. The maximum loan term is 40 years and grant funds may be available for facilities serving the most financially needy communities.

The ICDBG program assists Idaho Cities and Counties under 50,000 population with the development of needed public infrastructure and housing in an effort to support local economic diversification and growth. The program is administered by the Idaho Department of Commerce and Labor Division of Community Development, with funds received annually from the U.S. Department of Housing and Urban Development. ICDBG funds are used to construct projects that benefit low and moderate income persons, help prevent or eliminate slum and blight conditions, or solve catastrophic health and safety threats in local areas.

The U.S. Department of Commerce EDA provides funding for the construction of public infrastructure under the authority of the Public Works and Economic Development Act of 1965. Eligible projects include water and wastewater improvement and projects that support economic development within the community. Cities, counties and special cities are eligible to apply. Projects must meet economic development eligibility criteria as established by Congress - specifically, per capita income, employment and other demographic characteristics, with an emphasis on resolving unemployment and barriers to economic growth and stability. EDA funds are provided as grants from 50 to 80 percent of the project. Applicants must provide the local share from acceptable sources.

The EPA provides STAG grant funds through their Office of Enforcement and Compliance Assurance (OECA) to carry out compliance assurance activities related to regional focus areas, potentially including water and wastewater systems. Eligible grant recipients include States, tribes, territories, local governments and multi-jurisdictional organizations. The OECA typically announces the availability of grant funds for a specific focus area through a Federal Register Notice. Preference is generally given to those applicants that provide some match towards the grant.

The U.S. Army Corps of Engineers offers grants through their Section 595 Partnership program for various wastewater related projects. The grant amount and eligibility requirements vary.

The City may submit an application to the U.S. Congress for grant funding to assist with their wastewater system improvements. The applications are due in February of each year for consideration in the next fiscal year. The grant funds are typically routed through one of the existing funding programs listed above (i.e., EPA STAG grant).

The Idaho State Legislature and Courts have specified that communities can attach a price to new growth and development through the implementation of impact and/or new user capacity fees. Current laws allows government entities to charge a developer for a "proportionate share" of the cost of public facilities, including wastewater systems, impacted by residential, commercial, and industrial development. The calculation of the proportionate share must be based on a sturdy planning foundation. The funds collected from these fees are generally held in separate accounts and used for specific infrastructure improvements.

Many of the funding agencies consider a monthly user rate of approximately $35 to $45 per month as the minimum level at which they will consider a grant funding package. At rates lower than this level, the City may not be as likely to receive grant funding for the proposed project. As such, the City may need to consider raising the current rate prior to applying for grants from the funding agencies. Additionally, the availability of many of the grant funding programs is contingent upon passing a revenue bond.

The City should begin planning for financing of the proposed improvements, including both loans and grants, to minimize the costs to the community.

6.5 ENVIRONMENTAL CONSIDERATIONS

The proposed improvements, as outlined in this Facilities Plan, will take place in existing street right-of-ways for the collection system and lift station and at the existing wastewater treatment plant site. It is beyond the scope of this study to determine the full impacts to the environment, as the study looks only at general locations for improvements.

The proposed improvements should have minimal environmental impacts from construction activities. Heavy equipment and machinery will be used during construction, resulting in increased noise levels. However, construction activity should be limited to normal working hours to reduce the noise impacts on residential areas. In addition, construction noise should be temporary and can be minimized by the use of well-maintained equipment and mufflers.

Air quality may be impacted during construction due to dust and exhaust emissions from construction equipment, which may produce some minor air pollution. Debris created by construction should not be burned, but transported to a disposal area to avoid further air pollution. The impacts of construction dust can be mitigated by ceasing activity during exceptionally windy conditions and using watering equipment.

Open trenches, electrical utilities and heavy equipment may present health and safety hazards during construction. These hazards may be mitigated by educating project personnel about the applicable health and safety regulations and by establishing safe operating procedures. Traffic control may also result in a safety hazard, as traffic patterns are altered for construction purposes.

It is anticipated that impacts on agricultural lands, cultural resources, wetlands, plants or wildlife from the improvements will be minimal. If properly designed, operated and maintained, the proposed improvements should have minimal impacts on the soil, groundwater, and surface water.

There is a possibility that some of the improvements will be constructed in areas where trees and vegetation have been planted and the area has been landscaped. In all areas where the construction or installation of proposed improvements takes place, an extensive effort will be required to reconstruct, replant and landscape the area to its former condition.

6.6 IMPLEMENTATION ISSUES

Implementation of the proposed projects is a function of regulatory approval, public acceptance, funding and constructability. It is anticipated that the City will be able to obtain the necessary regulatory approval and permits for construction and operation of the proposed wastewater system improvements. The City's existing wastewater Reuse Permit may need to be modified by IDEQ for the proposed improvements. A building permit may be required from the City for construction of the building at the lift station. Additionally, a conditional or special use permit may be required from the County for the proposed wastewater treatment facility improvements.

The recommended improvements should provide a reliable, long-term wastewater collection and treatment system capable of meeting the existing and future needs of the City. Several similar systems have been constructed in other communities in Idaho and have operated satisfactorily to date with no significant problems. However, it is essential that operating personnel perform the proper routine operation and maintenance of the improvements to maintain the reliability of the system.

Groundwater during construction may pose a challenge during construction. A geotechnical analysis of the subsurface conditions may be required. Dewatering of open pits and trenches will likely be necessary during construction.

Based on similar construction projects in southern Idaho, no insurmountable construction problems are expected for the recommended improvements.

6.7 PULIC PARTICIPATION

A public hearing was held XXXX YY, 2012 at the Albion City Hall to discuss the alternatives and recommendations considered in this Facilities Plan. J-U-B ENGINEERS, Inc. presented a brief description of the Facility Plan and outlined the alternatives under consideration. Comments and questions from the public were addressed and incorporated, as necessary, into the final Facilities Plan. A copy of the sign-in sheet and comments from the public hearing is included in **Appendix F**. In general, the public expressed support of the proposed wastewater system improvements.

Appendix A

Existing Lift Station Pump Curve

PumpTech Co., Inc.
Larry Cazier
City of Albion: Main Lift Station

H2Optimize ver: 6.041
07/12/01

PUMP DATA SHEET

HYDROMATIC

Selection file: (untitled)

Catalog: HYDRO60.MPC v 2

Curve: S4N1150

Design Point: Flow: 180 US gpm
Head: 20 ft

Fluid: Water Temperature: 60 °F
SG: 1
Viscosity: 1.122 cP
Vapor pressure: 0.2568 psi_a
Atm pressure: 11.33 psi_a

Pump: **NCLOG-4 - 1200** **Size: S4N/S4NX**
Speed: 1150 rpm Dia: 7.875 in

Limits: Temperature: 140 °F Sphere size: 3 in
Pressure: 125 psi_g Power: --- bhp

NPSHa: --- ft

Specific Speed: Ns: --- Nss: ---

Piping: System: ---
Suction: --- in
Discharge: --- in

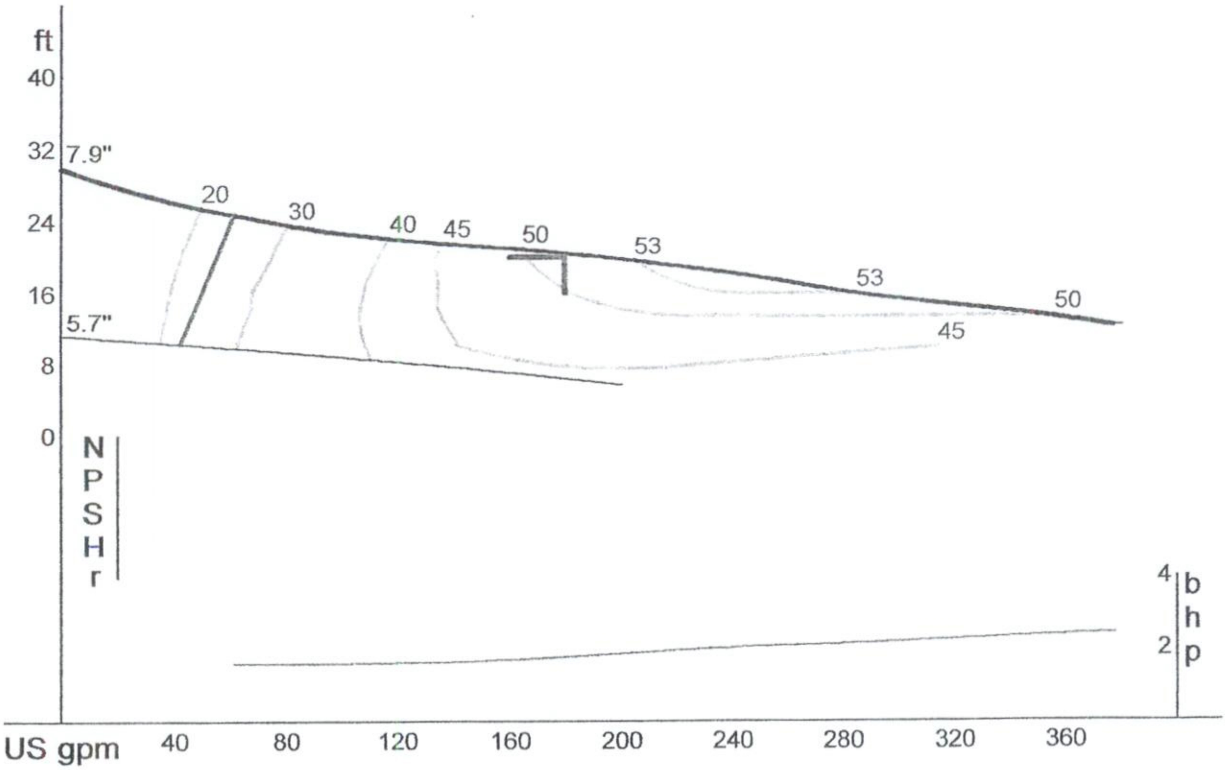
Dimensions: Suction: --- in Discharge: 4 in

Motor: 3 hp Speed: 1200 Frame: 213T
NEMA Standard TEFC Enclosure
sized for Max Power on Design Curve

---- Data Point ----
Flow: 180 US gpm
Head: 20.4 ft
Eff: 51%
Power: 1.8 bhp
NPSHr: - ft

-- Design Curve --
Shutoff Head: 29.9 ft
Shutoff dP: 12.9 psi
Min Flow: 61.3 US gpm
BEP: 54% eff
@ 245 US gpm
NOL Pwr: 2.44 bhp
@ 377 US gpm

-- Max Curve --
Max Pwr: 2.48 bhp
@ 381 US gpm



--- PERFORMANCE EVALUATION ---

Flow US gpm	Speed rpm	Head ft	Pump %eff	Power bhp	NPSHr ft	Motor %eff	Motor kW	Hrs/yr	Cost /kWh
216	1150	19.1	53	1.96	---				
180	1150	20.4	51	1.8	---				
144	1150	21.3	46	1.67	---				
108	1150	22.4	38	1.62	---				
72	1150	24.1	27	1.6	---				

Appendix B

Wastewater Reuse Permit

A. Permit Certificate

MUNICIPAL
WASTEWATER-LAND APPLICATION PERMIT
LA-000077-03

The <u>**City of Albion**</u> LOCATED AT <u>**P.O. Box 147, Albion, ID 83311**</u> AND IN <u>**Cassia County, Township 12 South, Range 25 East, Section 5**</u> IS HEREBY AUTHORIZED TO CONSTRUCT, INSTALL, AND OPERATE A WASTEWATER REUSE SYSTEM IN ACCORDANCE WITH THE WASTEWATER REUSE RULES (IDAPA 58.01.17) AND WASTEWATER RULES (IDAPA 58.01.16), THE GROUND WATER QUALITY RULE (IDAPA 58.01.11), AND ACCOMPANYING PERMIT, APPENDICES, AND REFERENCE DOCUMENTS. THIS PERMIT IS EFFECTIVE FROM THE DATE OF SIGNATURE AND EXPIRES ON <u>**February 26, 2015**</u>

Bill Allred 2-26-10.

Bill Allred, Regional Administrator Date:
Twin Falls Regional Office
Idaho Department of Environmental Quality

DEPARTMENT OF ENVIRONMENTAL QUALITY
1363 Fillmore St.
Twin Falls, ID 83301
208-736-2190

POSTING ON SITE RECOMMENDED

B. Permit Contents, Appendices, and Reference Documents

The Sections, Appendices, and Reference Documents listed on this page are all elements of Wastewater Reuse Permit LA-000077-03 and are enforceable as such. This permit does not relieve City of Albion, hereafter referred to as the permittee, from responsibility for compliance with other applicable federal, state or local laws, rules, standards or ordinances.

C. Abbreviations, Definitions

Ac-in	Acre-inch. The volume of water or wastewater to cover 1 acre of land to a depth of 1 inch. Equal to 27,154 gallons.
BMP or BMPs	Best Management Practices
COD	Chemical Oxygen Demand
DEQ or the Department	Idaho Department of Environmental Quality
Director	Director of the Idaho Department of Environmental Quality, or the Directors Designee, i.e. Regional Administrator
ET	Evapotranspiration – Loss of water from the soil and vegetation by evaporation and by plant uptake (transpiration)
GS	Growing Season – Typically April 01 through October 31 (214 days)
GW	Ground Water
GWQR	IDAPA 58.01.11 "Ground Water Quality Rule"
Guidance	Guidance for Land Application of Municipal and Industrial Wastewater http://www.deq.idaho.gov/water/permits_forms/permitting/guidance.cfm
HLRgs	Growing Season Hydraulic Loading Rate. Includes any combination of wastewater and supplemental irrigation water applied to land application hydraulic management units during the growing season. The HLRgs limit is specified in Section F. Permit Limits and Conditions.
HLRngs	Non-Growing Season Hydraulic Loading Rate. Includes any combination of wastewater and supplemental irrigation water applied to each hydraulic management unit during the non-growing season. The HLRngs limit is specified in Section F. Permit Limits and Conditions.
HMU	Hydraulic Management Unit (Serial Number designation is MU)
IWR	Irrigation Water Requirement – Any combination of wastewater and supplemental irrigation water applied at rates commensurate to the moisture requirements of the crop: $IWR = IR / E_i = (CU - P_e) / E_i$ Where: IR = net irrigation requirement $= CU - P_e$ CU = consumptive use (crop evapotranspiration) for a given crop in a given climatic area P_e = effective precipitation. E_i = irrigation system efficiency.
IDAPA	Idaho Administrative Procedures Act.
LG	Lagoon
lb/ac-day	Pounds (of constituent) per acre per day
MG	Million Gallons (1 MG = 36.827 acre-inches)
MGA	Million Gallons Annually (per WLAP Reporting Year)
NGS	Non-Growing Season – Typically November 01 through March 31 (151 days)
NVDS	Non-Volatile Dissolved Solids (= Total Dissolved Solids less Volatile Dissolved Solids)
O&M manual	Operation and Maintenance Manual, also referred to as the Plan of Operation
SAR	Sodium Adsorption Ratio

C. Abbreviations, Definitions

SI	Supplemental Irrigation water applied to the land application treatment site.
Soil AWC	Soil Available Water Holding Capacity - the water storage capability of a soil to a depth at which plant roots will utilize (typically 60 inches or root limiting layer)
SMU	Soil Monitoring Unit (Serial Number designation is SU)
SW	Surface Water
TDS	Total Dissolved Solids or Total Filterable Residue
TDIS	Total Dissolved Inorganic Solids – The summation of chemical concentration results in mg/L for the following common ions: calcium, magnesium, potassium, sodium, chloride, sulfate, and 0.6 times alkalinity (alkalinity expressed as calcium carbonate). Nitrate, Silica and fluoride shall be included if present in significant quantities (i.e. > 5 mg/L each).
TMDL	Total Maximum Daily Load – The sum of the individual waste-load allocations (WLA's) for point sources, Load Allocations (LA's) for non-point sources, and natural background. Such load shall be established at a level necessary to implement the applicable water quality standards with seasonal variations and a margin of safety that takes into account any lack of knowledge concerning the relationship between effluent limitations and water quality. IDAPA 58.01.02 *Water Quality Standards and Wastewater Treatment Requirements*
Typical Crop Uptake	Typical Crop Uptake is defined as the median constituent crop uptake from the three (3) most recent years the crop has been grown. Typical Crop Uptake is determined for each hydraulic management unit. For new crops having less than three years of on-site crop uptake data, regional crop yield data and typical nutrient content values, or other values approved by DEQ may be used.
USGS	United States Geological Survey
WWRU	Wastewater Reuse
WW	Wastewater

D. Facility Information

Legal Name of Permittee	City of Albion
Type of Wastewater	Class D Municipal Wastewater
Method of Treatment	Slow Rate Land Treatment
Type of Facility	Public
Facility Location	1 mile east of Albion, Idaho
Legal Location	Township 12S Range 25E Section 5
County	Cassia
USGS Quad	Albion
Soils on Site	Ririe Silt Loam, Downata Silt Loam
Depth to Ground Water	10 - 30 feet
Beneficial Uses of Ground Water	Agricultural, Domestic
Nearest Surface Water	Land Creek
Beneficial Uses of Surface Water	Agriculture, Aquatic Life
Responsible Official **Mailing Address** **Phone / Fax**	Don Bowden, Mayor P.O. Box 147 Albion, ID 83311 208-673-5352 / 208-673-6745

E. Compliance Schedule for Required Activities

The Activities in the following table shall be completed on or before the Completion Date unless modified by the Department in writing.

Compliance Activity Number Completion Date	Compliance Activity Description
CA-077-01 Twelve (12) Months after Permit Issuance	An updated Plan of Operation (Operation and Maintenance Manual or O&M Manual) for the wastewater land application facility, incorporating the requirements of this permit, shall be submitted to DEQ for review and approval. The Plan of Operation shall be designed for use as an operator guide for actual day-to-day operations to meet permit requirements and shall include sampling and monitoring requirements to assess the adequacy of wastewater treatment facility operation. The Plan of Operation shall contain at a minimum all of the information in the latest revision of the Plan of Operation Checklist found in the guidance.
CA-077-02 Twelve (12) Months after Permit Issuance	A Quality Assurance Project Plan (QAPP) for monitoring required in this permit, shall be submitted to DEQ for review and approval. The plan shall cover field activities; monitoring locations; laboratory analytical methods and other activities; data verification and validation; data storage, retrieval and assessment and monitoring program evaluation and improvement. Once completed the Quality Assurance Project Plan shall be included in the updated Plan of Operation.
CA-077-03 Plan due prior to conducting seepage test **Test results due prior to November 2011**	Submit a seepage testing plan to DEQ for review and approval that describes the procedures to be used to conduct seepage testing of Secondary Lagoon number two (LG-007702). Upon approval of the plan, conduct the seepage testing of Secondary Lagoon number two (LG-007702) in accordance with the approved plan and submit test results to DEQ. The seepage performance standard is 0.25 inches per day. If a properly tested lagoon leaks more than 0.25 inches per day, the permittee shall either: 1) Submit, for DEQ approval, a plan and schedule to either retest, repair, replace, or decommission structures not meeting this standard, or 2) Develop an assessment based on ground water sampling and analyses and/or modeling to determine the effect of the lagoon leakage on the local ground water. If actual or predicted impacts do not comply with IDAPA 58.01.11 as determined by DEQ, the permittee shall comply with 1) above.

E. Compliance Schedule for Required Activities

Compliance Activity Number Completion Date	Compliance Activity Description
CA-077-04 **Plan due prior to conducting seepage test** **Test results due prior to November 2012**	Submit a seepage testing plan to DEQ for review and approval that describes the procedures to be used to conduct seepage testing of Primary Lagoon number one (LG-007701). Upon approval of the plan, conduct the seepage testing of Primary Lagoon number one (LG-007701) in accordance with the approved plan and submit test results to DEQ. The seepage performance standard is 0.25 inches per day. If a properly tested lagoon leaks more than 0.25 inches per day, the permittee shall either: 1) Submit, for DEQ approval, a plan and schedule to either retest, repair, replace, or decommission structures not meeting this standard, or 2) Develop an assessment based on ground water sampling and analyses and/or modeling to determine the effect of the lagoon leakage on the local ground water. If actual or predicted impacts do not comply with IDAPA 58.01.11 as determined by DEQ, the permittee shall comply with 1) above.
CA-077-05 **One (1) Month after Permit Issuance**	Submit for DEQ review and approval a Crop Management Plan that describes the best management practices for increasing crop yield and nutrient uptake. In addition the crop management plan shall include a description of how crop yields are estimated.
CA-077-06 **Prior to wastewater irrigation**	Upgrade the current disinfection system to a system capable of meeting Class D wastewater disinfection requirements as specified in IDAPA 58.01.17.600.07.d. No wastewater shall be applied prior to completion of the disinfection system upgrades. Submit plans and specification for DEQ review and approval prior to disinfection system improvements.
CA-077-07 **Prior to wastewater irrigation**	Submit for DEQ review and approval, a Buffer Zone Plan that contains all the components described for Buffer Zone Plans in the latest revision of the Plan of Operation checklist found in the guidance. In addition describe the control measures used to prevent public access to within 300 feet of the application site. The plan shall describe the steps taken to mitigate effluent exposure for personnel planting, maintaining or harvesting adjacent fields. The plan shall also describe the measures taken to minimize wastewater irrigation drift to adjacent fields. Once completed the Buffer Zone Plan shall be included in the updated Plan of Operation.

F. Permit Limits and Conditions

The Permittee is allowed to apply wastewater and treat it on a land application site as prescribed in the table below and in accordance with all other applicable permit conditions and schedules.

Category	Permit Limits and Conditions
Type of Wastewater	Municipal, Class D
Application Site Area	13 acres
Application Season	Growing Season, April 1 - October 31 (214 days)
Reporting Year for Annual Loading Rates	November 1 - October 31
Growing Season Hydraulic Loading Rate, each HMU (Applies to wastewater and supplemental irrigation water)	Growing Season Hydraulic Loading Rate shall be substantially equal to the Irrigation Water Requirement (IWR) throughout the growing season.
Livestock Grazing	No grazing is allowed and animals shall not be fed harvested vegetation within two weeks of wastewater application.
Ground Water Quality	Wastewater land application activities shall not cause a violation of the *Ground Water Quality Rule* (GWQR), IDAPA 58.01.11.
Maximum Nitrogen Loading Rate, pounds/acre-year	150% of typical crop uptake (see Section C definitions) or loading rates specified in the University of Idaho Fertility Guides.
Maximum Phosphorus Loading Rate, pounds/acre-year	150% of typical crop uptake (see Section C definitions) or loading rates specified in the University of Idaho Fertility Guides.
Total Coliform Disinfection Level	The median value of the last three (3) results must not exceed 230 total coliform organisms/100ml and no confirmed sample shall exceed 2300 total coliform organisms/100ml.

F. Permit Limits and Conditions

Category	Permit Limits and Conditions
Buffer Zones From Reuse Site	All buffer zones must comply with, at a minimum, local zoning ordinances. Other minimum buffer zones are as follows: • 500 feet to inhabited dwellings • 300 feet to areas accessible by the public • 100 feet to permanent and intermittent surface water • 50 feet to irrigation ditches and canals • 500 feet to private water supply wells • 1000 feet to public water supply wells
Fencing and Posting	The application site shall be enclosed with a woven pasture fence or equivalent approved by DEQ. Signs should read 'Irrigated with Reclaimed Wastewater - Stay Back 300 Feet - Do Not Drink' or equivalent to be posted every 500 feet and at each corner of the application site. An additional sign shall be posted directly adjacent to the lagoon access road at the outer perimeter of the 300 foot public access buffer zone.
Allowable Crops	Crops grown for direct human consumption (those crops that are not processed prior to consumption) are not allowed.
Supplemental Irrigation Water Protection	For systems with wastewater and fresh irrigation water interconnections, DEQ-approved backflow prevention devices are required.
Odor Management	The land application facilities and other operations associated with the facility shall not create a public health hazard or nuisance conditions including odors. These facilities shall be managed in accordance with a DEQ approved Odor Management Plan as required by section E, CA-077-01. In the event that nuisance odors, verified by DEQ, occur, the Plan shall be revised as necessary to eliminate or minimize the reoccurrence of nuisance odors.
Runoff Control	No runoff is allowed.

G. Monitoring Requirements

1) Appropriate analytical methods, as given in the *Idaho Guidance for Reclamation and Reuse of Municipal and Industrial Wastewater*, or as approved by the Idaho Department of Environmental Quality (hereinafter referred to as DEQ), shall be employed. A description of approved sample collection methods, appropriate analytical methods and companion QA/QC protocol shall be included in the facility's Quality Assurance Project Plan (QAPP), which shall be part of the Operation and Maintenance Manual.

2) The permittee shall monitor and measure parameters as stated in the Facility Monitoring Table in this section.

3) Samples shall be collected at times and locations that represent typical environmental and process parameters being monitored.

4) Unless otherwise agreed to in writing by DEQ, data collected and submitted shall include, but not be limited to, the parameters and frequencies in the Facility Monitoring Table on the following pages. Wastewater monitoring is required at the frequency show in the table below if wastewater is applied anytime during the time period shown.

5) Five (5) soil sample locations shall be selected for each SMU. Three (3) soil samples shall be collected at each sample location, one at 0-12 inches, one at 12-24 inches, and one at 24-36 inches, or refusal. The soil samples collected at each depth shall be composited to yield three (3) samples for analysis from each SMU.

6) Annual reporting of monitoring requirements is described in Section H, Standard Reporting Requirements.

7) Monitoring locations are defined in Appendix 1, "Environmental Monitoring Serial Numbers".

Facility Monitoring Table

Frequency	Monitoring Point	Description/Type of Monitoring	Parameters
Daily	Hydraulic management unit	Estimated volume of supplemental irrigation water applied	Volume (million gallons) record daily, compile monthly
Daily	Flow meter	Flow of wastewater into lagoons	Volume (million gallons) record daily, compile monthly
Daily	Flow meter	Flow of wastewater into land application system	Volume (million gallons and acre-inches) to each hydraulic management unit (HMU), record daily, compile monthly
Monthly	Effluent to land application	Grab sample	total coliform bacteria (CFU/100 ml)

G. Monitoring Requirements

Frequency	Monitoring Point	Description/Type of Monitoring	Parameters
Monthly	Effluent to land application	Grab sample	pH, nitrate and nitrite nitrogen, total Kjeldahl nitrogen, total phosphorus, electrical conductivity
Annually	Hydraulic management unit	Acres used for land application	Acres
Annually	Hydraulic Management Unit	Calculate wastewater nitrogen and phosphorus loading rates	Pounds/acre-year
Annually	Hydraulic management unit	Report nitrogen and phosphorus fertilizer application rates	Type and Pounds/acre-year
Annually	Hydraulic management unit	Crop type and yield	Pounds/acre-year (specify moisture basis)
Annually	Hydraulic management unit	Crop Nutrient Uptake from Crop Tissue Analysis or from standard tables for Crop Type and yield	Nitrogen and phosphorus uptake in lbs/ac-year
Annually	Hydraulic management unit	Calculate Month-Specific Irrigation Water Requirement for comparison with GS hydraulic loading	Inches/acre-month for each crop type
Annually	All supplemental irrigation pumps directly connected to the wastewater distribution system	Backflow testing	Document the testing of all backflow prevention devices. Report the testing date(s) and results of the test (pass or fail). If any test failed, report the date of repair or replacement of backflow prevention device, and if the repaired/replaced device is operating correctly
Annually	All flow measurement locations.	Flow measurement calibration of all flows to land application.	Document the flow measurement calibration of all flow meters and pumps used directly or indirectly to measure all wastewater, tail water, flushing water, and supplemental irrigation water flows applied.

G. Monitoring Requirements

Frequency	Monitoring Point	Description/Type of Monitoring	Parameters
May 2014	Soil Monitoring Unit	Soil monitoring (see note 5 above)	Nitrate and nitrite nitrogen, electrical conductivity, ammonia nitrogen, plant available phosphorus

H. Standard Reporting Requirements

1.) The Permittee shall submit an Annual Wastewater Reuse Site Performance Report ("Annual Report") prepared by a competent environmental professional no later than January 31 of each year, which shall cover the previous reporting year. The Annual Report shall include an interpretive discussion of monitoring data (ground water, soils, hydraulic loading, wastewater etc.) with particular respect to environmental impacts by the facility.

2.) The annual report shall contain the results of the required monitoring as described in *Section G. Monitoring Requirements*. If the permittee monitors any parameter more frequently than required by this permit, the results of this monitoring shall be included in the calculation and reporting of the data submitted in the annual report.

3.) The annual report shall be submitted to the DEQ Engineering Manager in the Twin Falls Regional Office.

Twin Falls Regional Office
1363 Fillmore St.
Twin Falls, ID 83301
208-736-2190

A copy of the annual report shall also be mailed to:

Richard Huddleston, P.E.
Wastewater Program Manager
1410 N. Hilton
Boise, ID 83706
208-373-0561

4.) Notice of completion of any work described in *Section E. Compliance Schedule for Required Activities* shall be submitted to the Department within 30 days of activity completion. The status of all other work described in Section E shall be submitted with the Annual Report.

5.) All laboratory reports containing the sample results for monitoring required by *Section G. Monitoring Requirements* of this permit shall be submitted with the Annual Report.

I. Standard Permit Conditions: Procedures and Reporting

1. The permittee shall at all times properly maintain and operate all structures, systems, and equipment for treatment, operational controls and monitoring, which are installed or used by the permittee to comply with all conditions of the permit or the Wastewater Reuse Permit Regulations, in conformance with a DEQ approved, current Plan of Operations (Operations and Maintenance Manual) which describes in detail the operation, maintenance, and management of the wastewater treatment system. This Plan of Operations shall be updated as necessary to reflect current operations.

2. Wastewater(s) or recharge waters applied to the land surface must be restricted to the premises of the application site. Wastewater discharges to surface water that require a permit under the Clean Water Act must be authorized by the U.S. Environmental Protection Agency.

3. Wastewater must not create a public health hazard or nuisance condition as stated in IDAPA 58.01.16.600.03. In order to prevent public health hazards and nuisance conditions the permittee shall:

   a. Apply wastewater as evenly as practicable to the treatment area;
   b. Prevent organic solids (contained in the wastewater) from accumulating on the ground surface to the point where the solids putrefy or support vectors or insects; and
   c. Prevent wastewater from ponding in the fields to the point where the ponded wastewater putrefies or supports vectors or insects.

4. The permittee shall:

   a. Manage the wastewater reuse treatment site as an agronomic operation where vegetative cover is grown and harvested or grazed to utilize the nutrients and minerals in the wastewater, and,
   b. Not hydraulically overload any particular areas of the wastewater reuse treatment site.

5. All waste solids, including dredgings and sludges, shall be utilized or disposed in a manner which will prevent their entry, or the entry of contaminated drainage or leachate therefrom, into the waters of the state such that health hazards and nuisance conditions are not created; and to prevent impacts on designated beneficial uses of the ground water and surface water. The permittee's management of waste solids shall be governed by the terms of the DEQ approved Waste Solids Management Plan, which upon approval shall be an enforceable portion of this permit.

6. If the permittee intends to continue operation of the permitted facility after the expiration of an existing permit, the permittee shall apply for a new permit at least six months prior to the expiration date of the existing permit in accordance with the Wastewater Reuse Permit Regulations and include seepage tests on all lagoons per latest DEQ procedures.

7. The permittee shall allow the Director of the Idaho Department of Environmental Quality or the Director' designee (hereinafter referred to as Director), consistent with Title 39, Chapter 1, Idaho Code, to:

   a. Enter the permitted facility,
   b. Inspect any records that must be kept under the conditions of the permit.
   c. Inspect any facility, equipment, practice, or operation permitted or required by the permit.
   d. Sample or monitor for the purpose of assuring permit compliance, any substance or any parameter at the facility.

8. The permittee shall report to the Director under the circumstances and in the manner specified in this section:

   a. In writing thirty (30) days before any planned physical alteration or addition to the permitted facility or activity if that alteration or addition would result in any significant change in information that was submitted during the permit application process.

b. In writing thirty (30) days before any anticipated change which would result in non-compliance with any permit condition or these regulations.

c. Orally within twenty-four (24) hours from the time the permittee became aware of any non-compliance which may endanger the public health or the environment at telephone numbers provided in the permit by the Director (see below)

DEQ Regional Office: see Permit Certificate Page
Emergency 24 Hour Number: 1-800-632-8000

d. In writing as soon as possible but within five (5) days of the date the permittee knows or should know of any non-compliance unless extended by the DEQ. This report shall contain:

i. A description of the non-compliance and its cause;
ii. The period of non-compliance including to the extent possible, times and dates and, if the non-compliance has not been corrected, the anticipated time it is expected to continue; and
iii. Steps taken or planned to reduce or eliminate reoccurrence of the non-compliance.

e. In writing as soon as possible after the permittee becomes aware of relevant facts not submitted or incorrect information submitted, in a permit application or any report to the Director. Those facts or the correct information shall be included as a part of this report.

9. The permittee shall take all necessary actions to prevent or eliminate any adverse impact on the public health or the environment resulting from permit noncompliance.

10. The permittee shall determine (on an on-going basis) if any noxious weed problems relate to the permitted sites. If problems are present, coordinate with the Idaho Department of Agriculture or the local County authority regarding their requirements for noxious weed control. Also address these control operations in an update to the Operations and Maintenance Manual.

J. Standard Permit Conditions: Modifications, Violation, and Revocation

1. The permittee shall furnish to the Director within reasonable time, any information including copies of records, which may be requested by the Director to determine whether cause exists for modifying, revoking, re-issuing, or terminating the permit, or to determine compliance with the permit or these regulations.

2. Both minor and major modifications may be made to this permit as stated in IDAPA 58.01.17.700.01 and 02 with respect to any conditions stated in this permit upon review and approval of the DEQ.

3. Whenever a facility expansion, production increase or process modification is anticipated which will result in a change in the character of pollutants to be discharged or which will result in a new or increased discharge that will exceed the conditions of this permit, or if it is determined by the DEQ that the terms or conditions of the permit must be modified in order to adequately protect the public health or environment, a request for either major or minor modifications must be submitted together with the reports as described in Section I. *Standard Reporting Requirements*, and plans and specifications for the proposed changes. No such facility expansion, production increase or process modification shall be made until plans have been reviewed and approved by the DEQ and a new permit or permit modification has been issued.

4. Permits shall be transferable to a new owner or operator provided that the permittee notifies the Director by requesting a minor modification of the permit before the date of transfer.

5. Any person violating any provision of the Wastewater Reuse Permit Regulations, or any permit or order issued thereunder shall be liable for a civil penalty not to exceed ten thousand dollars ($10,000) or one thousand dollars ($1,000) for each day of a continuing violation, whichever is greater. In addition, pursuant to Title 39, Chapter 1, Idaho Code, any willful or negligent violation may constitute a misdemeanor.

6. The Director may revoke a permit if the permittee violates any permit condition or the Wastewater Reuse Permit Regulations.

7. Except in cases of emergency, the Director shall issue a written notice of intent to revoke to the permittee prior to final revocation. Revocation shall become final within thirty-five (35) days of receipt of the notice by the permittee, unless within that time the permittee request an administrative hearing in writing to the Board of Environmental Quality pursuant to the Rules of Administrative Procedures contained in IDAPA 58.01.23.

8. If, pursuant to Idaho Code, 67-5247, the Director finds the public health, safety or welfare requires emergency action, the Director shall incorporate findings in support of such action in a written notice of emergency revocation issued to the permittee. Emergency revocation shall be effective upon receipt by the permittee. Thereafter, if requested by the permittee in writing, a revocation hearing before the Board of Environmental Quality shall be provided. Such hearings shall be conducted in accordance with the Rules of Administrative Procedures contained in IDAPA 58.01.23.

9. The provisions of this permit are severable and if a provision or its application is declared invalid or unenforceable for any reason, that declaration will not affect the validity or enforceability of the remaining provisions.

10. The permittee shall notify the DEQ at least six (6) months prior to permanently removing any permitted reuse facility from service, including any treatment, storage, or other facilities or equipment associated with the reuse site. Prior to commencing closure activities, the permittee shall: a) participate in a pre-site closure meeting with the DEQ; b) develop a site closure plan that identifies specific closure, site characterization, or cleanup tasks with scheduled task completion dates in accordance with agreements made at the pre-site closure meeting; and c) submit the completed site

closure plan to the DEQ for review and approval within forty-five (45) days of the pre-site closure meeting. The permittee must complete the DEQ approved site closure plan.

Appendix 1
Environmental Monitoring Serial Numbers

HYDRAULIC MANAGEMENT UNITS

Serial Number	Description	Acres
MU-007701	City of Albion land application site	13

WASTEWATER SAMPLING POINTS

Serial Number	Description
WW-007701	Effluent to land application system

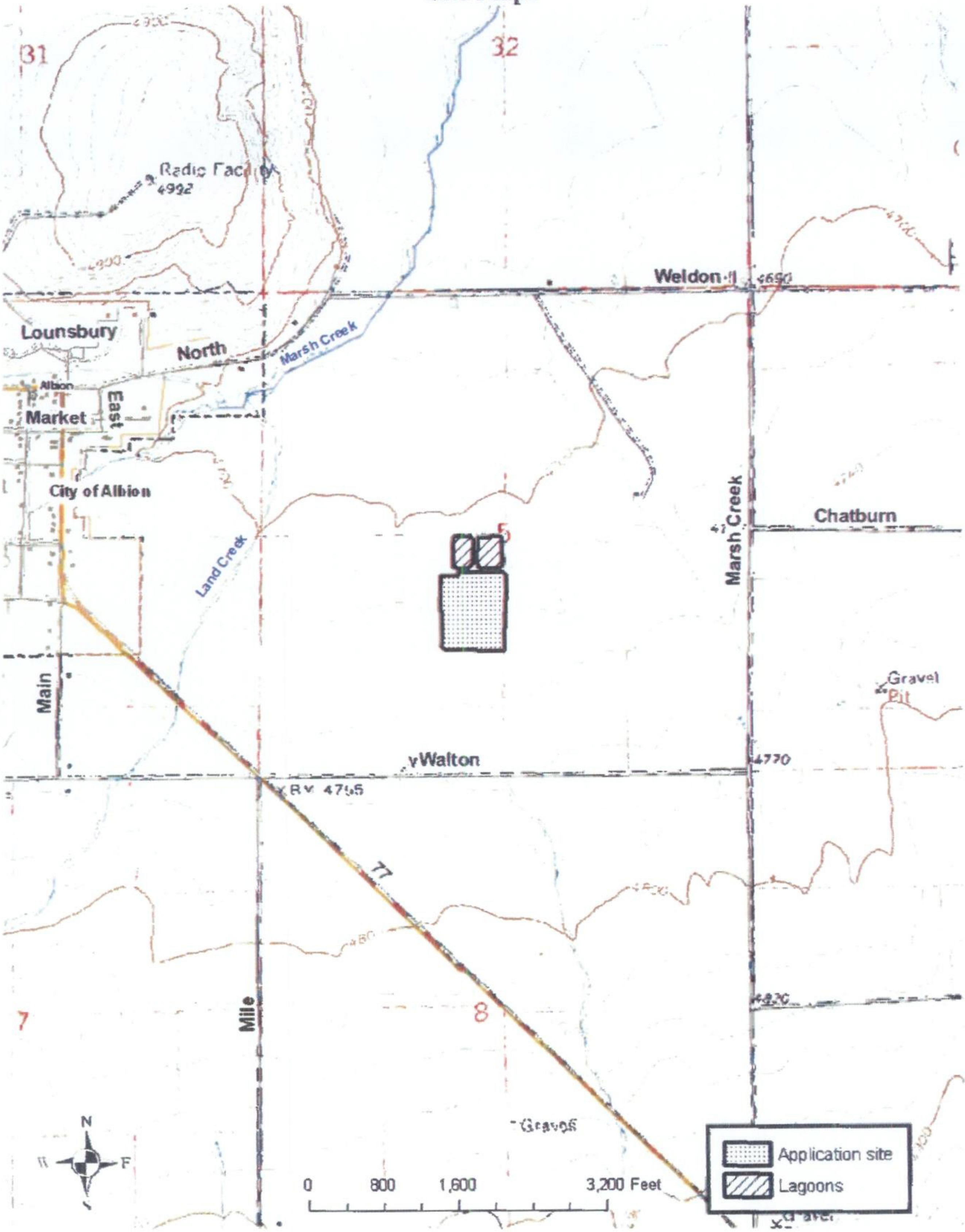
SOIL MONITORING UNITS

Serial Number	Description	Associated MU
SU-007701	Land Application acreage	MU-007701

LAGOONS

Serial Number	Description	Capacity (million gallons)
LG-007701	Primary lagoon one (east)	5.1
LG-007702	Secondary lagoon two (west)	3.9

Appendix 2
Site Maps



Appendix C

Calculations

CITY OF ALBION
2012 WASTEWATER FACILITIES PLAN
INFLUENT FLOWS AND LOADS
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J-U-B installed flow monitoring equipment that recorded flow at the influent lift station from July 6 - July 29, 2011

Average flow rate	44.7	gpm
	64,368	gal/d

This flowrate is significantly higher than would be expected for a community of this size.

Expected flow rate	100	gal/person/d (typical design value for residential and commercial flows, the City does not have any industrial flows)
2010 population	267	
Expected flow rate	26,700	gal/d

It is assumed the difference between the actual flow rate and expected domestic flow rate is due to infiltration and inflow.

	Average Day Flow		
Domestic	26,700	gal/day	(residential and commercial flows)
Infiltration and Inflow	37,668	gal/day	
Total Flow	64,368	gal/day	

PEAKING FACTORS APPLIED TO DOMESTIC FLOWS ONLY

Location	Maximum Month	Minimum Month	Peak Week	Peak Day	Peak Hour
Kimberly	1.18	-	-	1.89	3.31
Filer	1.47	0.85	-	2.11	2.65
Hagerman	1.14	0.81	-	-	-
Carey	1.30	0.59	-	-	-
Literature	-	-	-	-	3.21[a]
Albion	**1.27**	**0.75**	**1.75**	**2.00**	**3.21**

a. Estimated using 10 State Standards Equation modified to local conditions $(14+P^{0.5})/(4+P^{0.5})$ where P equals population in thousands (0.267 in 2010)

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I&I PEAKING FACTORS FROM PUMP RUN TIME DATA

Wet weather peaking factor			Dry weather peaking factor		
Month	**Run time hours**	**Maximum month PF**	**Month**	**Run time**	**Minimum month PF**
May-09	13.4	1.15	Sep-09	11.4	0.97
Jun-09	15.5	1.33	Aug-09	10.0	0.86
Jul-09	11.6		Jul-09	11.6	
2009 average run time		12.0	Annual - July PF		1.03
May-10	17.5	1.40	Sep-10	10.4	0.83
Jun-10	18.6	1.48	Oct-10	11.5	0.92
Jul-10	12.5		Jul-10	12.5	
2010 average run time		13.2	Annual - July PF		1.06
May-11	17.5	1.02	Sep-11	12.0	0.70
Jun-11	18.6	1.08	Jul-11	17.2	
Jul-11	17.2				
2011 average run time		16.4	Annual - July PF		0.96
2009-2011 average		**1.30**	**2009-2011 average**		**0.80**

size of service area	120	acres
inflow rate	250	gal/acre/d
peak day and peak hour inflow	30,000	gal/d

Parameter	Unit	Domestic	I&I	Total	
Average Day flow	gpd	26,700	37,700	64,400	
Maximum Month[a]	gpd	33,900	49,000	82,900	
Minimum Month[a]	gpd	20,000	30,000	50,000	
Peak Day[a]	gpd	53,400	67,700	121,100	I&I = average day + 30,000 gal/d inflow
Peak Hour[a]	gpd	85,700	79,000	164,700	I&I = max month + 30,000 gal/d inflow
Average Day Per Capita[b]	gpcd	100	141	241	

a. Based on the peaking factors shown above.
b. Based on a population of 267 in 2010.

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INFLUENT SAMPLING DATA AT LIFT STATION

Date	Day of Week	Time	BOD mg/L	TSS mg/L	TKN mg/L	Nitrate-N mg/L	Total P mg/L
10/13/2011	Thursday	9:05 AM	104	111	14.1	0.3	3.71
10/19/2011	Wednesday	9:15 AM	275	601	24.6	0.3	5.07
10/26/2011	Wednesday	10:15 AM	151	136	20.4	0.3	6.44
		AVERAGE	177	283	19.7	0.3	5.07

Since the influent sampling data consists only of 3 datapoints taken within 2 weeks of eachother, and one of the datapoints is significantly different than the other two, it is recommended to use typical wastewater constituent concentrations.

TYPICAL WASTEWATER CONCENTRATIONS

	BOD	TSS	Total P	TKN	Nitrate-N
Domestic [1]	300	300	8	50	0.5
I&I	5	5	0.5	1	1.3

Drinking water system reports 1.33 mg/L nitrates.

1. Typical values in southern Idaho based on sampling data at similar communities (Filer, Wendell, Murtaugh, and Hazelton).

TYPICAL WASTE LOAD PEAKING FACTORS

Parameter	Kimberly	Filer	Hagerman	M&E [b]	Average Used For Albion
COD/BOD[a]					
MM	1.30	1.40	1.69	1.25	1.41
PD	2.27	3.20	-	2.50	2.66
TSS					
MM	1.51	1.50	1.50	1.30	1.45
PD	2.30	4.20	-	2.70	3.07
TKN					
MM	1.30	1.30	1.51	1.30	1.35
PD	2.17	2.20	-	2.17	2.18
Nitrate-N					
MM	1.30	1.30	1.95	1.30	1.46
PD	2.17	2.20	-	2.17	2.18
Total P					
MM	1.25	1.30	1.47	1.25	1.32
PD	1.75	1.80	-	1.75	1.77

a. Peaking factors for COD and BOD assumed to be the same.

b. Metcalf & Eddy, 2003, Figure 3-8.

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Existing Wasteload Summary

			Domestic	I&I	Total
BOD	Avg Day	mg/L	300	5	127
		ppd	67	2	69
		ppcd	0.25	0.01	0.26
	Max Month	ppd	95	2	97
	Peak Day	ppd	178	3	181
TSS	Avg Day	mg/L	300	5	127
		ppd	67	2	69
		ppcd	0.25	0.01	0.26
	Max Month	ppd	97	2	99
	Peak Day	ppd	205	3	208
TKN	Avg Day	mg/L	50	1	21
		ppd	11.1	0.3	11.4
		ppcd	0.042	0.001	0.043
	Max Month	ppd	15.0	0.4	15.4
	Peak Day	ppd	24.2	0.6	24.8
Nitrate-N	Avg Day	mg/L	0.5	1.3	1.0
		ppd	0.1	0.4	0.5
		ppcd	0.0004	0.0015	0.0019
	Max Month	ppd	0.1	0.5	0.6
	Peak Day	ppd	0.2	0.8	1.0
Total-P	Avg Day	mg/L	8.0	0.5	3.6
		ppd	1.8	0.2	2.0
		ppcd	0.0067	0.0007	0.0074
	Max Month	ppd	2.4	0.2	2.6
	Peak Day	ppd	3.2	0.3	3.5

Historical Growth

Year	Population	% Growth	Overall % Growth
1960	415		
1970	229	-5.78%	
1980	286	2.21%	
1990	305	0.64%	
2000	262	-1.52%	
2010	267	0.19%	-0.87%

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Option 3 - assume 2% growth rate for both residential and commercial (alternative selected by City)

2.0%	growth rate

Year	Population	Avg Day Domestic Flow [1] gal/d	Avg Day I&I Flow gal/d	Total Annual Avg Day Flow gal/d	Max Month Flow gal/d	Peak Day Flow gal/d	Peak Hour Flow gal/d	BOD lbs/d	TSS lbs/d	Total P lbs/d	TKN lbs/d	Nitrate-N lbs/d
2010	267	26,700	37,700	64,400	82,900	121,100	164,700	68	68	1.9	11	0.5
2011	272	27,200	37,700	64,900	83,500	122,100	166,300	70	70	2.0	12	0.5
2012	278	27,800	37,700	65,500	84,300	123,300	168,200	71	71	2.0	12	0.5
2013	283	28,300	37,700	66,000	84,900	124,300	169,800	72	72	2.0	12	0.5
2014	289	28,900	37,700	66,600	85,700	125,500	171,800	74	74	2.1	12	0.5
2015	295	29,500	37,700	67,200	86,500	126,700	173,700	75	75	2.1	13	0.5
2016	301	30,100	37,700	67,800	87,200	127,900	175,600	77	77	2.2	13	0.5
2017	307	30,700	37,700	68,400	88,000	129,100	177,500	78	78	2.2	13	0.5
2018	313	31,300	37,700	69,000	88,800	130,300	179,500	80	80	2.2	13	0.5
2019	319	31,900	37,700	69,600	89,500	131,500	181,400	81	81	2.3	14	0.6
2020	325	32,500	37,700	70,200	90,300	132,700	183,300	83	83	2.3	14	0.6
2021	332	33,200	37,700	70,900	91,200	134,100	185,600	85	85	2.4	14	0.6
2022	339	33,900	37,700	71,600	92,100	135,500	187,800	86	86	2.4	14	0.6
2023	345	34,500	37,700	72,200	92,800	136,700	189,700	88	88	2.5	15	0.6
2024	352	35,200	37,700	72,900	93,700	138,100	192,000	90	90	2.5	15	0.6
2025	359	35,900	37,700	73,600	94,600	139,500	194,200	91	91	2.6	15	0.6
2026	367	36,700	37,700	74,400	95,600	141,100	196,800	93	93	2.6	16	0.6
2027	374	37,400	37,700	75,100	96,500	142,500	199,100	95	95	2.7	16	0.6
2028	381	38,100	37,700	75,800	97,400	143,900	201,300	97	97	2.7	16	0.6
2029	389	38,900	37,700	76,600	98,400	145,500	203,900	99	99	2.8	17	0.6
2030	397	39,700	37,700	77,400	99,400	147,100	206,400	101	101	2.8	17	0.6
2031	405	40,500	37,700	78,200	100,400	148,700	209,000	103	103	2.9	17	0.6
2032	413	41,300	37,700	79,000	101,500	150,300	211,600	105	105	2.9	18	0.6
2033	421	42,100	37,700	79,800	102,500	151,900	214,100					
2034	429	42,900	37,700	80,600	103,500	153,500	216,700					
2035	438	43,800	37,700	81,500	104,600	155,300	219,600					
2036	447	44,700	37,700	82,400	105,800	157,100	222,500					
2037	456	45,600	37,700	83,300	106,900	158,900	225,400					
2038	465	46,500	37,700	84,200	108,100	160,700	228,300					
2039	474	47,400	37,700	85,100	109,200	162,500	231,200					
2040	484	48,400	37,700	86,100	110,500	164,500	234,400					
2041	493	49,300	37,700	87,000	111,600	166,300	237,300					

F:\Projects\JUB\60-11-041-City_of_Albion_2011_Wastewater_Facilities_Plan_&_EID\Spreadsheets\Albion flows and loads - lagoon analysis.xlsx

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2042	503	50,300	37,700	88,000	112,900	168,300	240,500
2043	513	51,300	37,700	89,000	114,200	170,300	243,700
2044	524	52,400	37,700	90,100	115,500	172,500	247,200
2045	534	53,400	37,700	91,100	116,800	174,500	250,400
2046	545	54,500	37,700	92,200	118,200	176,700	253,900
2047	556	55,600	37,700	93,300	119,600	178,900	257,500
2048	567	56,700	37,700	94,400	121,000	181,100	261,000
2049	578	57,800	37,700	95,500	122,400	183,300	264,500
2050	590	59,000	37,700	96,700	123,900	185,700	268,400
2051	601	60,100	37,700	97,800	125,300	187,900	271,900
2052	613	61,300	37,700	99,000	126,900	190,300	275,800

1. 100 gal/d/person (typical design value for domestic wastewater)

AVERAGE DAY CONCENTRATION (MG/L) 159 159 4.42 26.6 0.90

Parameter	Unit	Domestic	I&I	Total	
Average Day flow	gpd	41,300	37,700	79,000	
Maximum Month[a]	gpd	52,500	49,000	101,500	
Minimum Month[a]	gpd	31,000	30,000	61,000	
Peak Day[a]	gpd	82,600	67,700	150,300	I&I = average day + 30,000 gal/d inflow
Peak Hour[a]	gpd	132,600	79,000	211,600	I&I = max month + 30,000 gal/d inflow
Average Day Per Capita[b]	gpcd	100	91	191	

a. Based on the peaking factors shown above.
b. Based on a population of 608 in 2031.

annual 28,835,000 Mgal

CITY OF ALBION
2012 WASTEWATER FACILITIES PLAN
INFLUENT FLOWS AND LOADS
60-11-041

Future Wasteload Summary

			Domestic	I&I	Total
BOD	Avg Day	mg/L	300	5	159
		ppd	103	2	105
		ppcd	0.25	0.01	0.26
	Max Month	ppd	145	2	147
	Peak Day	ppd	274	3	277
TSS	Avg Day	mg/L	300	5	159
		ppd	103	2	105
		ppcd	0.25	0.01	0.26
	Max Month	ppd	150	2	152
	Peak Day	ppd	316	3	319
TKN	Avg Day	mg/L	50	1	27
		ppd	17.2	0.3	17.5
		ppcd	0.042	0.001	0.043
	Max Month	ppd	23.3	0.4	23.7
	Peak Day	ppd	37.5	0.6	38.1
Nitrate-N	Avg Day	mg/L	0.5	1.3	0.9
		ppd	0.2	0.4	0.6
		ppcd	0.0005	0.0010	0.0015
	Max Month	ppd	0.3	0.5	0.8
	Peak Day	ppd	0.4	0.8	1.2
Total-P	Avg Day	mg/L	8.0	0.5	4.4
		ppd	2.8	0.2	3.0
		ppcd	0.0068	0.0005	0.0073
	Max Month	ppd	3.7	0.2	3.9
	Peak Day	ppd	4.9	0.3	5.2

CITY OF ALBION
2012 WASTEWATER FACILITIES PLAN
LAGOON CALCS
60-11-041

1) Lagoon Sludge Volume

	Water Surface Area (acres)	Water Surface Area (sq ft)	Bottom Surface Area (acres)	Bottom Surface Area (sq ft)	Design Water Depth (ft)	Calc'd Volume (Mgal)	Average Sludge Depth [1] (ft)	Sludge Volume (Mgal)	Effective Lagoon Volume (Mgal)	% Sludge
Cell #1	2.63	114,563	2.10	91,260	6.0	4.62	0.67	0.46	4.16	10.0
Cell #2	2.00	87,120	1.53	66,586	6.0	3.45	0.67	0.33	3.12	9.6
Total	4.63	201,683	3.6	157,846		8.07		0.79	7.28	

[1] Sludge depth informally measured by Operator... ponds were iced over so he couldn't use a sludge judge.

2) Facultative Lagoon Hydraulic Retention Time

	Avg Day	Max Mon	
Existing Flow (2012) =	0.0644	0.083	MGD
Projected Flow (2032) =	0.0790	0.102	MGD

	Lagoon Volume (mgal)	Sludge Volume (mgal)	Operating Volume (mgal)	Average Day 2012 Operating HRT (d)	Average Day 2032 Operating HRT (d)	Max Month 2012 Operating HRT (d)	Max Month 2032 Operating HRT (d)
Cell #1	4.62	0.46	4.16	64.6	52.7	50.2	41.0
Cell #2	3.45	0.33	3.12	48.4	39.5	37.6	30.7
	8.07	0.79	7.28	113.0	92.2	87.8	71.7

*** Hydraulic retention time of existing lagoon system appears to be acceptable through 2032**

Aerated Lagoons:
- 8 to 20 days in cold weather climates to reduce algal growth (Chapte
- 5 to 30 days - M&E, 1991, p. 645, Table 10-20
- 4 to 10 days - M&E, 2004, p. 841, Table 8-29

Facultative Lagoons:
- 20 to 180 days (EPA Tech. Fact Sheet)
- 5 to 30 days warm climate, 180 days cold (Chpt. 6, p. 238)
- 180 days between high WSL & 2 foot - controlled discharge - entire :
- 90 to 120 days - flow through - entire system (10 State Standards, p.
- 80 to 180 days at winter air temps of <33 F - total system (Chpt. 10,
- 30 to 80 days at winter air temps of <33 F - first cell (Chpt. 10, p. 10-
- 60 to 120 days (M&E, 1991, p. 642)

3) Facultative Lagoon BOD Loading

Year	Population	Avg Day BOD (lbs/d)	Cell #1 SLR (lbs/ac-d)	Total SLR (lbs/ac-d)
2010	267	68.4	26.0	14.8
2011	272	69.6	26.5	15.0
2012	278	71.1	27.0	15.4
2013	283	72.4	27.5	15.6
2014	289	73.9	28.1	16.0
2015	295	75.4	28.7	16.3
2016	301	76.9	29.2	16.6
2017	307	78.4	29.8	16.9
2018	313	79.9	30.4	17.3
2019	319	81.4	30.9	17.6

Facultative Lagoons:
- 13 to 71 lbs/acre-d (EPA Tech. Fact Sheet)
- 15 to 50 lbs/acre-d (Chpt 6., p. 238 & 249)
- 15 to 35 lbs/acre-d - primary pond (10 State Standards, p. 90-15)
- 20 to 25 lbs/acre-d (Chpt. 10, p. 10-10)
- 20 to 40 lbs/acre-d at winter air temps of 33-59 F - total system (Chpt. 10, p. 10-13)
- 60 to 120 lbs/acre-d at winter air temps of 33-59 F - first cell (Chpt. 10, p. 10-13)
- 25 to 35 lbs/acre-d - single pond or parallel (Reynolds, p. 560)
- 75 to 80 lbs/acre-d - primary pond in series operation (Reynolds, p. 560)
- 25 to 35 lbs/acre-d - secondary ponds in series operation (Reynolds, p. 560)
- 16.7 to 60 lbs/acre-d (M&E, p. 645)

CITY OF ALBION
2012 WASTEWATER FACILITIES PLAN
LAGOON CALCS
60-11-041

2020	325	82.9	31.5	17.9
2021	332	84.6	32.2	18.3
2022	339	86.4	32.8	18.7
2023	345	87.9	33.4	19.0
2024	352	89.6	34.1	19.4
2025	359	91.4	34.8	19.7
2026	367	93.4	35.5	20.2
2027	374	95.1	36.2	20.6
2028	381	96.9	36.8	20.9
2029	389	98.9	37.6	21.4
2030	397	100.9	38.4	21.8
2031	405	102.9	39.1	22.2
2032	413	104.9	39.9	22.7

Aeration Requirements

Oxygen Requirement = 1.5 lbs O_2/lbs BOD

1.5 - 10 State Std, 92.331, p. 90-9, act. sludge
2 - Chapter 10, p. 10-37
1.75 - 2.5 - Chapter 6, p. 238
2.25 - Chapter 6, p. 250
1.75 - 2.25 - M&E, p. 648

Max Month BOD Load	147	lbs BOD/d
Oxygen Demand	221	lbs O_2/d
	9.2	lbs O_2/hr
HP Required	6.1	hp

Assume

Splasher Aerators = 1.50 lbs O_2/hp-hr

Cell		HP per Unit	Number of Units	Total HP	Total O_2 Supplied (lbs O_2/d)	BOD Treatment Capacity (lbs BOD/d)
#1	Splasher	5	2	10	360	240
	Total		**2**	**10**	**360**	**240**

TOTAL	**360**	**240**
Oxygen Demand	221	

CITY OF ALBION
2012 WASTEWATER FACILITIES PLAN
DISINFECTION CALCS
60-11-041

Contact Time

	Avg Pumping Rate Max Lagoon WSL	Avg Pumping Rate Min Lagoon WSL	Max Pumping Rate Max Lagoon WSL	Max Pumping Rate Min Lagoon WSL		
Contact Basin						
Basin Length	11.25	11.25	11.25	11.25	ft	From 1974 design drawings
Basin Width	7	7	7	7	ft	From 1974 design drawings
Basin Floor Elevation	4706.5	4706.5	4706.5	4706.5	ft	From 1974 design drawings
Water Surface Elevation	4713	4709	4713	4709	ft	From 1974 design drawings
Water Depth	6.5	2.5	6.5	2.5	ft	From 1974 design drawings
Volume	3,829	1,473	3,829	1,473	gal	
Pump Wet-Well						
Basin Length	3.42	3.42	3.42	3.42	ft	From 1974 design drawings
Basin Width	7	7	7	7	ft	From 1974 design drawings
Basin Floor Elevation	4702.5	4702.5	4702.5	4702.5	ft	From 1974 design drawings
Water Surface Elevation	4713	4709	4713	4709	ft	From 1974 design drawings
Water Depth	10.5	6.5	10.5	6.5	ft	From 1974 design drawings
Volume	1,879	1,163	1,879	1,163	gal	
Total Volume	**5,708**	**2,636**	**5,708**	**2,636**	**gal**	
Pumping Rate	***129***	***129***	***164***	***164***	***gpm***	
Contact Time	***44***	***20***	***35***	***16***	***min***	

Chlorine Pump Sizing

	Min Pump Rate	**Max Pumping Rate**	
Max Chlorine Concentration	8.0	8.0	mg/L
Pumping Rate	129	164	gpm
Max Daily Chlorine Dose	12.4	15.8	lbs/d
Hypochlorite Concentration	10.0	10.0	%
Chlorine Mass/Volume	0.83	0.83	lbs/gal
Volumetric Hypochlorite Pumping Rate	14.9	19.0	gpd
Volumetric Hypochlorite Pumping Rate	**0.62**	**0.79**	**gph**

Estimated NaOCl Use

	Existing	Future	
Average Chlorine Concentration	3.0	3.0	mg/L
Effluent Volume	6.82	12.14	mgal
Annual Mass of Chlorine	171	304	lbs
Annual Volume of Hypochlorite	206	366	gal

CITY OF ALBION
2012 WASTEWATER FACILITIES PLAN
WATER BALANCE - EXISTING FLOWS
60-11-041

Effluent Flows & Waste Loads

Annual Effluent Volume Applied to Land App Site = 9.81 mgal/yr

	mg/L	lbs/yr	lbs/ac
COD	108	8,818	678
Total-N	11.5	941	72
Total-P	2.9	234	18

Irrigation Water Requirement

Assume:				
	Crop =	Alfalfa - Frequent Cuttings		
	Type of Irrigation =	Handlines		
	Irrigation Efficiency =	67.5	%	Guidance for Reclamation and Reuse of Municipal and Industrial Wastewater, Table 4-22, p. 4-79.
	Growing Season (GS) =	214	d	(April 1 - October 31)
	Non-Growing Season (NGS) =	151	d	(November 1 - March 31)
	Land Application Acreage =	13	acres	

Parameter	Units	Apr	May	Jun	Jul	Aug	Sep	Oct	Total
Average Precipitation[1]	inches	1.18	1.54	1.12	0.72	0.68	0.77	0.89	6.90
Consumptive Use[2]	inches	4.21	7.45	6.93	8.19	7.46	5.72	2.93	42.89
Effective Precipitation[3]	inches	0.88	1.36	0.98	0.67	0.61	0.63	0.62	5.75
Carryover Soil Moisture[4]	inches	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Leaching Requirement[4]	inches	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Net Irrigation Requirement	inches	3.33	6.09	5.95	7.52	6.85	5.09	2.31	37.14
Total Irrigation Requirement	inches	4.93	9.02	8.82	11.14	10.15	7.54	3.42	55.02
Total Irrigation Requirement	mgal								19.42

1 Precipitation data from the WRCC for monitoring stations at Albion College of Education, Malta 2E, Malta Aviation, and Oakley. http://www.wrcc.dri.edu/summary/Climsmsid.h
2 "Evapotranspiration and Consumptive Irrigation Water Requirements for Idaho" Allen, Richard G. and Clarence W. Robison, 2006 (Revised 2007). http://www.kimberly.uidaho.edu/ETIdaho/
3 Calculated based on SCS Formula in IDEQ Guidance for Reclamation and Reuse of Municipal and Industrial Wastewater, Table 4-8, p. 4-53.
4 Assumed to be 0 inches

Crop Consumptive Use Data & Conversion (Alfalfa - Frequent Cuttings)

Monitoring Station	Units	Apr	May	Jun	Jul	Aug	Sep	Oct
Oakley (NWS - 106542)	mm/day	3.96	6.03	5.63	6.05	5.42	4.67	3.05
	in/day	0.16	0.24	0.22	0.24	0.21	0.18	0.12
	inches	4.68	7.36	6.65	7.38	6.61	5.52	3.72
Malta (Agrimet - MALI)	mm/day	2.93	5.46	5.42	6.79	6.29	4.56	1.83
	in/day	0.12	0.21	0.21	0.27	0.25	0.18	0.07
	inches	3.46	6.66	6.40	8.29	7.68	5.39	2.23
Malta 1NE (NWS - 105563)	mm/day	3.79	6.82	6.55	7.29	6.63	5.28	2.32
	in/day	0.15	0.27	0.26	0.29	0.26	0.21	0.09
	inches	4.48	8.32	7.74	8.90	8.09	6.24	2.83

CITY OF ALBION
2012 WASTEWATER FACILITIES PLAN
WATER BALANCE - EXISTING FLOWS
60-11-041

Allowable Loadings

Parameter	Units	
Crop Yield	tons/acre	4
Crop Yield	lbs/acre	8,000
Nitrogen		
% Dry Mass Basis[1]	%	2.789
Crop Uptake	lbs/acre-yr	223
Allowable Application Rate[2]	lbs/acre-yr	335
Phosphorus		
% Dry Mass Basis[1]	%	0.261
Crop Uptake	lbs/acre-yr	21
Allowable Application Rate[2]	lbs/acre-yr	32
Organic (COD)		
Allowable Application Rate[3]	lbs/acre-d	50

1 Value from USDA NRCS Nutrients Available from Livestock Manure Relative to Crop Growth Requirements, C.H. Lander, D. Moffitt, and K. Alt, February 1998 Resource Assessment and Strategic Planning Workshop Paper 98-1, Table A-1, Appendix 1
2 Per Section F of Reuse Permit - 150% of crop uptake
3 IDEQ Guidance for Reclamation and Reuse of Municipal and Industrial Wastewater

Storage and Land Area Requirements

A) Make an initial estimate at the irrigated land area requirements based on one of the following limiting factors:

Hydraulic

Annual Effluent Volume	9.81	Mgal
Allowable Hydraulic Loading	55.02	inches
Acreage Required	6.6	acres

Nutrients

	Total-N	Total-P	
Average Annual Effluent Load	941	234	lbs/yr
Allowable Loading	335	32.0	lbs/acre-yr
Acreage Required	2.9	7.4	acres

Organic

Average Annual Effluent Load	8,818	lbs/yr
Allowable Loading	50	lbs/acre-d
Acreage Required	0.9	acres

Initial Estimate of Irrigated Acreage Requirement

Sub-Total	7.4	acres
Contingency (10%)	0.8	acres
Total	8.2	acres

CITY OF ALBION
2012 WASTEWATER FACILITIES PLAN
WATER BALANCE - EXISTING FLOWS
60-11-041

B) Calculate the lagoon storage volume and surface area requirements.

Water Balance Calculations - Alfalfa

Influent Flow =	64,400	gpd
Avg Lagoon Depth =	6.00	ft
Irrigated Acreage =	13.0	acres
Weighted Seepage Rate =	0.220	in/d

Cell	Surface Area	Seepage
#1	2.63	0.22
#2	2.00	0.22
Wtd Avg		0.2200

Existing Lagoon Surface Area =	4.63	acres
Iterative Additional Storage Lagoon Surface Area =	0.00	acres
Additional Storage Lagoon Surface Area =	0.00	acres
Total Lagoon Surface Area =	4.63	acres

Existing Effective Lagoon Volume =	7.28	Mgal
Max Lagoon Volume Required =	5.38	Mgal
Additional Storage Lagoon Volume =	0.00	Mgal
Total Lagoon Volume =	7.28	Mgal

Month	Days	Inflow	Precipitation		Evaporation		Seepage		Net Inflow	Crop Requirement		WW Applied to Crop	Discharge to River	Change in Storage	Net Accum. Storage	Supplemental Irrigation Water	
		Mgal	Inches	Mgal	Inches	Mgal	Inches	Mgal	Mgal	Inches	Mgal	Mgal	Mgal	Mgal	Mgal	Mgal	MGD
Oct	31	2.00	0.89	0.11	2.40	0.30	6.82	0.86	0.95	3.42	1.21	0.95		0.00	0.00	0.26	0.01
Nov	30	1.93	0.89	0.11	2.00	0.25	6.60	0.83	0.96	0.00	0.00	0.00		0.96	0.96	0.00	0.00
Dec	31	2.00	0.84	0.11	1.20	0.15	6.82	0.86	1.10	0.00	0.00	0.00		1.10	2.06	0.00	0.00
Jan	31	2.00	0.92	0.12	0.20	0.03	6.82	0.86	1.23	0.00	0.00	0.00		1.23	3.29	0.00	0.00
Feb	28	1.80	0.73	0.09	0.60	0.08	6.16	0.77	1.04	0.00	0.00	0.00		1.04	4.33	0.00	0.00
Mar	31	2.00	0.90	0.11	1.60	0.20	6.82	0.86	1.05	0.00	0.00	0.00		1.05	5.38	0.00	0.00
Apr	30	1.93	1.18	0.15	3.20	0.40	6.60	0.83	0.85	4.93	1.74	1.74		0.00	4.49	0.00	0.00
May	31	2.00	1.54	0.19	5.60	0.70	6.82	0.86	0.63	9.02	3.19	3.19		0.00	1.93	0.00	0.00
Jun	30	1.93	1.12	0.14	6.00	0.75	6.60	0.83	0.49	8.82	3.11	2.42		0.00	0.00	0.69	0.02
Jul	31	2.00	0.72	0.09	6.80	0.85	6.82	0.86	0.38	11.14	3.93	0.38		0.00	0.00	3.55	0.11
Aug	31	2.00	0.68	0.09	6.40	0.80	6.82	0.86	0.43	10.15	3.58	0.43		0.00	0.00	3.15	0.10
Sep	30	1.93	0.77	0.10	4.00	0.50	6.60	0.83	0.70	7.54	2.66	0.70		0.00	0.00	1.96	0.07
Total		23.52	11.18	1.41	40.00	5.01	80.30	10.11	9.81	55.02	19.42	9.81	0.00			9.61	
													9.81			19.42	

CITY OF ALBION
2012 WASTEWATER FACILITIES PLAN
WATER BALANCE - PROJECTED FLOWS
60-11-041

Effluent Flows & Waste Loads

Annual Effluent Volume Applied to Land App Site = 15.13 mgal/yr

	mg/L	lbs/yr	lbs/ac
COD	108	13,600	1046
Total-N	11.5	1,451	112
Total-P	2.9	361	28

Irrigation Water Requirement

Assume:			
	Crop =	Alfalfa - Frequent Cuttings	
	Type of Irrigation =	Handlines	
	Irrigation Efficiency =	67.5 %	Guidance for Reclamation and Reuse of Municipal and Industrial Wastewater, Table 4-22, p. 4-79.
	Growing Season (GS) =	214 d	(April 1 - October 31)
	Non-Growing Season (NGS) =	151 d	(November 1 - March 31)
	Land Application Acreage =	13 acres	

Parameter	Units	Apr	May	Jun	Jul	Aug	Sep	Oct	Total
Average Precipitation[1]	inches	1.18	1.54	1.12	0.72	0.68	0.77	0.89	6.90
Consumptive Use[2]	inches	4.21	7.45	6.93	8.19	7.46	5.72	2.93	42.89
Effective Precipitation[3]	inches	0.88	1.36	0.98	0.67	0.61	0.63	0.62	5.75
Carryover Soil Moisture[4]	inches	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Leaching Requirement[4]	inches	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Net Irrigation Requirement	inches	3.33	6.09	5.95	7.52	6.85	5.09	2.31	37.14
Total Irrigation Requirement	inches	4.93	9.02	8.82	11.14	10.15	7.54	3.42	55.02
Total Irrigation Requirement	mgal								19.42

1 Precipitation data from the WRCC for monitoring stations at Albion College of Education, Malta 2E, Malta Aviation, and Oakley. http://www.wrcc.dri.edu/summary/Climsmsid.h
2 "Evapotranspiration and Consumptive Irrigation Water Requirements for Idaho" Allen, Richard G. and Clarence W. Robison, 2006 (Revised 2007). http://www.kimberly.uidaho.edu/ETIdaho/
3 Calculated based on SCS Formula in IDEQ Guidance for Reclamation and Reuse of Municipal and Industrial Wastewater, Table 4-8, p. 4-53.
4 Assumed to be 0 inches

Crop Consumptive Use Data & Conversion (Alfalfa - Frequent Cuttings)

Monitoring Station	Units	Apr	May	Jun	Jul	Aug	Sep	Oct
Oakley (NWS - 106542)	mm/day	3.96	6.03	5.63	6.05	5.42	4.67	3.05
	in/day	0.16	0.24	0.22	0.24	0.21	0.18	0.12
	inches	4.68	7.36	6.65	7.38	6.61	5.52	3.72
Malta (Agrimet - MALI)	mm/day	2.93	5.46	5.42	6.79	6.29	4.56	1.83
	in/day	0.12	0.21	0.21	0.27	0.25	0.18	0.07
	inches	3.46	6.66	6.40	8.29	7.68	5.39	2.23
Malta 1NE (NWS - 105563)	mm/day	3.79	6.82	6.55	7.29	6.63	5.28	2.32
	in/day	0.15	0.27	0.26	0.29	0.26	0.21	0.09
	inches	4.48	8.32	7.74	8.90	8.09	6.24	2.83

CITY OF ALBION
2012 WASTEWATER FACILITIES PLAN
WATER BALANCE - PROJECTED FLOWS
60-11-041

Allowable Loadings

Parameter	Units	
Crop Yield	tons/acre	4
Crop Yield	lbs/acre	8,000
Nitrogen		
% Dry Mass Basis[1]	%	2.789
Crop Uptake	lbs/acre-yr	223
Allowable Application Rate[2]	lbs/acre-yr	335
Phosphorus		
% Dry Mass Basis[1]	%	0.261
Crop Uptake	lbs/acre-yr	21
Allowable Application Rate[2]	lbs/acre-yr	32
Organic (COD)		
Allowable Application Rate[3]	lbs/acre-d	50

1 Value from USDA NRCS Nutrients Available from Livestock Manure Relative to Crop Growth Requirements, C.H. Lander, D. Moffitt, and K. Alt, February 1998
Resource Assessment and Strategic Planning Workshop Paper 98-1, Table A-1, Appendix 1
2 Per Section F of Reuse Permit - 150% of crop uptake
3 IDEQ Guidance for Reclamation and Reuse of Municipal and Industrial Wastewater

Storage and Land Area Requirements

A) Make an initial estimate at the irrigated land area requirements based on one of the following limiting factors:

Hydraulic

Annual Effluent Volume	15.13	Mgal
Allowable Hydraulic Loading	55.02	inches
Acreage Required	10.2	acres

Nutrients

	Total-N	Total-P	
Average Annual Effluent Load	1,451	361	lbs/yr
Allowable Loading	335	32.0	lbs/acre-yr
Acreage Required	4.4	11.3	acres

Organic

Average Annual Effluent Load	13,600	lbs/yr
Allowable Loading	50	lbs/acre-d
Acreage Required	1.3	acres

Initial Estimate of Irrigated Acreage Requirement

Sub-Total	11.3	acres
Contingency (10%)	1.2	acres
Total	12.5	acres

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WATER BALANCE - PROJECTED FLOWS
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B) Calculate the lagoon storage volume and surface area requirements.

Water Balance Calculations - Alfalfa

Influent Flow =	79,000	gpd
Avg Lagoon Depth =	6.00	ft
Irrigated Acreage =	13.0	acres
Weighted Seepage Rate =	0.220	in/d

Cell	Surface Area	Seepage
#1	2.63	0.22
#2	2.00	0.22
Wtd Avg		0.2200

Existing Lagoon Surface Area =	4.63	acres
Iterative Additional Storage Lagoon Surface Area =	0.25	acres
Additional Storage Lagoon Surface Area =	0.00	acres
Total Lagoon Surface Area =	4.63	acres

Existing Effective Lagoon Volume =	7.28	Mgal
Max Lagoon Volume Required =	7.77	Mgal
Additional Storage Lagoon Volume =	0.49	Mgal
Total Lagoon Volume =	7.77	Mgal

Month	Days	Inflow Mgal	Precipitation Inches	Precipitation Mgal	Evaporation Inches	Evaporation Mgal	Seepage Inches	Seepage Mgal	Net Inflow Mgal	Crop Requirement Inches	Crop Requirement Mgal	WW Applied to Crop Mgal	Discharge to River Mgal	Change in Storage Mgal	Net Accum. Storage Mgal	Supplemental Irrigation Water Mgal	Supplemental Irrigation Water MGD
Oct	31	2.45	0.89	0.11	2.40	0.30	6.82	0.86	1.40	3.42	1.21	1.21		0.19	0.19	0.00	0.00
Nov	30	2.37	0.89	0.11	2.00	0.25	6.60	0.83	1.40	0.00	0.00	0.00		1.40	1.59	0.00	0.00
Dec	31	2.45	0.84	0.11	1.20	0.15	6.82	0.86	1.55	0.00	0.00	0.00		1.55	3.14	0.00	0.00
Jan	31	2.45	0.92	0.12	0.20	0.03	6.82	0.86	1.68	0.00	0.00	0.00		1.68	4.82	0.00	0.00
Feb	28	2.21	0.73	0.09	0.60	0.08	6.16	0.77	1.45	0.00	0.00	0.00		1.45	6.27	0.00	0.00
Mar	31	2.45	0.90	0.11	1.60	0.20	6.82	0.86	1.50	0.00	0.00	0.00		1.50	7.77	0.00	0.00
Apr	30	2.37	1.18	0.15	3.20	0.40	6.60	0.83	1.29	4.93	1.74	1.74		0.00	7.32	0.00	0.00
May	31	2.45	1.54	0.19	5.60	0.70	6.82	0.86	1.08	9.02	3.19	3.19		0.00	5.21	0.00	0.00
Jun	30	2.37	1.12	0.14	6.00	0.75	6.60	0.83	0.93	8.82	3.11	3.11		0.00	3.03	0.00	0.00
Jul	31	2.45	0.72	0.09	6.80	0.85	6.82	0.86	0.83	11.14	3.93	3.86		0.00	0.00	0.07	0.00
Aug	31	2.45	0.68	0.09	6.40	0.80	6.82	0.86	0.88	10.15	3.58	0.88		0.00	0.00	2.70	0.09
Sep	30	2.37	0.77	0.10	4.00	0.50	6.60	0.83	1.14	7.54	2.66	1.14		0.00	0.00	1.52	0.05
Total		28.84	11.18	1.41	40.00	5.01	80.30	10.11	15.13	55.02	19.42	15.13	0.00			4.29	
													15.13			19.42	

CITY OF ALBION
2012 WASTEWATER FACILITIES PLAN
LIFT STATION CALCULATIONS - EXISTING CONDITIONS
60-11-041

Flows & Velocities

1 Velocities

Flow (mgd)	0.06	0.09	0.13	0.17
Flow (gpm)	40	65	90	115
Flow (cfs)	0.09	0.15	0.20	0.26
Pipe Diameter (in)	6	6	6	6
Velocity (fps)	**0.45**	**0.74**	**1.02**	**1.30**

2 Barometric Pressure

	Elevation (ft amsl)	Barometric Pressure (psi)		
Standard Elevation (Low)	4000	12.692		Environmental Engineering Reference Manual, Appendix 18A, Lindeburg, 2001
Standard Elevation (High)	5000	12.225		Environmental Engineering Reference Manual, Appendix 18A, Lindeburg, 2001
Site Elevation	4695	12.367	psi	
		28.57	ft	
	=>	**24.28**	ft	Use 85% of recommended value for fluctuations in pressure

3 Vapor Pressure (85 °F)

0.596	psi	Cameron Hydraulic Data, Westaway and Loomis, 1979, p. 4-4.
1.38	ft	

Discharge Head

1 Pipe Friction Loss

Assumptions:
- Full pipe flow.
- Clean water headloss is calculated using Hazen-Williams equation.
- A Hazen-Williams coefficient of 120 is used for clean water headloss calc's (conservative, but required by R317)

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$$h_f = \frac{10.44LQ^{1.85}}{C^{1.85}d^{4.8655}}$$

where:
h_f = friction losses (ft)
L = length of pipe (ft)
Q = flow (gpm)
C = Hazen-Williams coefficient for ductile iron pipe (140)
d = pipe diameter (in)

Flow	40	65	90	115	gpm
Discharge Pipe Length	40	40	40	40	ft
Discharge Pipe Dia.	4	4	4	4	in
Hazen-Williams Coeff.	120	120	120	120	-
Discharge Pipe Length	4866	4866	4866	4866	ft
Discharge Pipe Dia.	6	6	6	6	in
Hazen-Williams Coeff.	120	120	120	120	-
Pipe h_f (clean water)	**1.15**	**2.83**	**5.17**	**8.14**	**ft**

4" inside the pump station will incr

2 Minor Friction Loss

Assumptions:
- Loss coefficients assumed to be the same as clean water. A correction factor will be applied to approximate biosolids headloss (Sanks, 1998, chpt 19).

$$h_f = K\frac{V^2}{2g}$$

where:
h_f = friction losses (ft)
K = loss coefficient
V = velocity (fps)
g = 32.2 ft/s^2

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2012 WASTEWATER FACILITIES PLAN
LIFT STATION CALCULATIONS - EXISTING CONDITIONS
60-11-041

Fitting	Size (in)	No.	K	h_f	h_f	h_f	h_f		
			Flow (gpm)	40	65	90	115	gpm	
			Velocity (fps)	0.45	0.74	1.02	1.3	fps	
90 deg elbow	6	8	0.80	0.02	0.05	0.10	0.17	ft	Pumping Station Design, Sanks, 1
Check valve	6	1	1.70	0.01	0.01	0.03	0.04	ft	Pumping Station Design, Sanks, 1
Plug valve	6	2	0.50	0.00	0.01	0.02	0.03	ft	Pumping Station Design, Sanks, 1
Tee (branch)	6	2	0.75	0.00	0.01	0.02	0.04	ft	Pumping Station Design, Sanks, 1
Flow meter	6	0	0.01	0.00	0.00	0.00	0.00	ft	Very low; similar to a 1' long pipe s
Tee (through)	6	2	0.30	0.00	0.01	0.01	0.02	ft	Pumping Station Design, Sanks, 1
45 deg elbow	6	4	0.20	1.85	1.85	1.85	1.85	ft	Pumping Station Design, Sanks, 1
Outlet	6	1	1.00	0.00	0.01	0.02	0.03	ft	Pumping Station Design, Sanks, 1
Minor h_f (clean water)				**1.88**	**1.95**	**2.05**	**2.18**	ft	

3 Elevation Head

	High WSL	Mid WSL	Low WSL		Lagoon Water Level
High Point in Pipeline	4714.00	4712.00	4710.00	ft	Design WSE in primary pond= 47
Pump discharge elevation	4693.66	4693.66	4693.66	ft	avg level of pump1 on/OFF, from
Elevation Head	**20.34**	**18.34**	**16.34**	**ft**	

4 Total Clean Water Headloss

Flow	40	65	90	115	gpm
Pipe Friction Loss	1.15	2.83	5.17	8.14	ft
Minor Friction Loss	1.88	1.95	2.05	2.18	ft
Elevation Head	20.34	20.34	20.34	20.34	ft
Total Clean Water Discharge Head	**23.37**	**25.12**	**27.56**	**30.66**	ft

Total Dynamic Head

Total Dynamic Head (High WSL)	**23.37**	**25.12**	**27.56**	**30.66**	**ft**
	10.1	**10.9**	**11.9**	**13.3**	**psi**
Total Dynamic Head (Mid WSL)	**21.37**	**23.12**	**25.56**	**28.66**	**ft**
	9.3	**10.0**	**11.1**	**12.4**	**psi**
Total Dynamic Head (Low WSL)	**19.37**	**21.12**	**23.56**	**26.66**	**ft**
	8.4	**9.1**	**10.2**	**11.5**	**psi**

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LIFT STATION CALCULATIONS - EXISTING CONDITIONS
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Pump Curve

Flow (gpm)	Head (ft)
40	26
72	24.1
108	22.4
144	21.3

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LIFT STATION CALCULATIONS - FUTURE CONDITIONS
60-11-041

Flows & Velocities

1 Velocities

Flow (mgd)	0.23	0.26	0.29
Flow (gpm)	160	180	200
Flow (cfs)	0.36	0.40	0.45
Pipe Diameter (in)	6	6	6
Velocity (fps)	**1.81**	**2.04**	**2.27**

2 Barometric Pressure

	Elevation (ft amsl)	Barometric Pressure (psi)		
Standard Elevation (Low)	4000	12.692		Environmental Engineering Reference Manual, Appendix 18A, Lindeburg, 2001
Standard Elevation (High)	5000	12.225		Environmental Engineering Reference Manual, Appendix 18A, Lindeburg, 2001
Site Elevation	4695	12.367	psi	
		28.57	ft	
	=>	**24.28**	ft	Use 85% of recommended value for fluctuations in pressure

3 Vapor Pressure (85 °F)

0.596	psi	Cameron Hydraulic Data, Westaway and Loomis, 1979, p. 4-4.
1.38	ft	

Discharge Head

1 Pipe Friction Loss

Assumptions:
- Full pipe flow.
- Clean water headloss is calculated using Hazen-Williams equation.
- A Hazen-Williams coefficient of 120 is used for clean water headloss calc's (conservative, but required by R317)

$$h_f = \frac{10.44 L Q^{1.85}}{C^{1.85} d^{4.8655}}$$

where: h_f = friction losses (ft)
L = length of pipe (ft)

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	Q =	flow (gpm)
	C =	Hazen-Williams coefficient for ductile iron pipe (140)
	d =	pipe diameter (in)

Flow	160	180	200	gpm	
Discharge Pipe Length	40	40	40	ft	
Discharge Pipe Dia.	4	4	4	in	
Hazen-Williams Coeff.	120	120	120	-	
Discharge Pipe Length	4866	4866	4866	ft	
Discharge Pipe Dia.	6	6	6	in	4" inside the pump station will increase head loss slightly
Hazen-Williams Coeff.	120	120	120	-	
Pipe h_f (clean water)	**14.99**	**18.64**	**22.65**	**ft**	

2 Minor Friction Loss

Assumptions:
- Loss coefficients assumed to be the same as clean water. A correction factor will be applied to approximate biosolids headloss (Sanks, 1998, chpt 19).

$$h_f = K\frac{V^2}{2g}$$

where:	h_f =	friction losses (ft)
	K =	loss coefficient
	V =	velocity (fps)
	g =	32.2 ft/s^2

			Flow (gpm)	160	180	200	gpm	
			Velocity (fps)	1.81	2.04	2.27	fps	
Fitting	Size (in)	No.	K	h_f	h_f	h_f		
90 deg elbow	6	8	0.80	0.33	0.41	0.51	ft	Pumping Station Design, Sanks, 1998, p. 898-899.
Check valve	6	1	1.70	0.09	0.11	0.14	ft	Pumping Station Design, Sanks, 1998, p. 898-899.
Plug valve	6	2	0.50	0.05	0.06	0.08	ft	Pumping Station Design, Sanks, 1998, p. 898-899.
Tee (branch)	6	2	0.75	0.08	0.10	0.12	ft	Pumping Station Design, Sanks, 1998, p. 898-899.
Flow meter	6	0	0.01	0.00	0.00	0.00	ft	Very low; similar to a 1' long pipe spool
Tee (through)	6	2	0.30	0.03	0.04	0.05	ft	Pumping Station Design, Sanks, 1998, p. 898-899.
45 deg elbow	6	4	0.20	1.85	1.85	1.85	ft	Pumping Station Design, Sanks, 1998, p. 898-899.
Outlet	6	1	1.00	0.05	0.06	0.08	ft	Pumping Station Design, Sanks, 1998, p. 898-899.
Minor h_f (clean water)				**2.48**	**2.63**	**2.83**	**0.00**	ft

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LIFT STATION CALCULATIONS - FUTURE CONDITIONS
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3 Elevation Head

	High WSL	Mid WSL	Low WSL		Lagoon Water Level
High Point in Pipeline	4714.00	4712.00	4710.00	ft	Design WSE in primary pond= 4714.00
Pump discharge elevation	4693.66	4693.66	4693.66	ft	avg level of pump1 on/OFF, from orig plans.
Elevation Head	**20.34**	**18.34**	**16.34**	**ft**	

4 Total Clean Water Headloss

Flow	160	180	200	gpm
Pipe Friction Loss	14.99	18.64	22.65	ft
Minor Friction Loss	2.48	2.63	2.83	ft
Elevation Head	20.34	20.34	20.34	ft
Total Clean Water Discharge Head	**37.81**	**41.61**	**45.82**	ft

Total Dynamic Head

Total Dynamic Head (High WSL)	**37.81**	**41.61**	**45.82**	**ft**
	16.4	**18.0**	**19.8**	**psi**
Total Dynamic Head (Mid WSL)	**35.81**	**39.61**	**43.82**	**ft**
	15.5	**17.1**	**19.0**	**psi**
Total Dynamic Head (Low WSL)	**33.81**	**37.61**	**41.82**	**ft**
	14.6	**16.3**	**18.1**	**psi**

Appendix D

Cost Estimates

CITY OF ALBION
2012 WASTEWATER FACILITIES PLAN
GRAVITY COLLECTION SYTEM IMPROVEMENTS
OPINION OF PROBABLE COSTS

Gravity Collection System - Cleaning & Video Inspection

Item	Estimated Quantity	Unit	Unit Price	Total Price
Clean and Video Inspect Gravity Collection System	17,500.00	LF	$0.80	$14,000
Sub-Total Construction Costs				**$14,000**
Contractor Mob/Demob, Bonding, Admin, Insurance (10%)				$1,400
Contingency (25%)				$3,500
Total Construction Costs				**$18,900**
Engineering & Construction Admin. (20%)				$3,800
Administration & Funding (5%)				$0
Inflation (4% for 2 Years)				$0
TOTAL PROJECT COST				***$22,700***

Gravity Collection System - Replace Trunk Line North of Wet-Well

Item	Estimated Quantity	Unit	Unit Price	Total Price
Mobilization/Demobilization (5%)	1	LS	$2,100.00	$2,100
Traffic Control	1	LS	$2,000.00	$2,000
8" PVC Gravity Sewer Main	480	LF	$36.00	$17,300
Standard 4' Dia. Manhole with Ring and Cover	1	EA	$2,500.00	$2,500
Connection to Existing Manhole	2	EA	$1,000.00	$2,000
Disconnect/Abandon Existing Gravity Sewer	1	LS	$1,000.00	$1,000
Remove/Dispose of Existing Manhole	1	EA	$500.00	$500
Water/Nonpotable Water Crossing with Sleeving	1	EA	$1,000.00	$1,000
Sewer Service Line Re-Connection	3	EA	$400.00	$1,200
Asphalt Surface Repair	240	LF	$35.00	$8,400
Gravel Surface Repair	240	LF	$14.00	$3,400
Bypass Pumping	10	Days	$250.00	$2,500
Post Install Cleaning/Video Inspection	480	LF	$2.00	$1,000
Sub-Total Construction Costs				**$44,900**
Contractor Mob/Demob, Bonding, Admin, Insurance (10%)				$4,500
Dewatering (10%)				$4,500
Contingency (25%)				$11,200
Total Construction Costs				**$65,100**
Engineering & Construction Admin. (20%)				$13,000
Administration & Funding (5%)				$3,300
Inflation (4% for 2 Years)				$5,200
TOTAL PROJECT COST				***$86,600***

CITY OF ALBION
2012 WASTEWATER FACILITIES PLAN
GRAVITY COLLECTION SYTEM IMPROVEMENTS
OPINION OF PROBABLE COSTS

Gravity Collection System - Replace Sewer Mains to Reduce I&I

Item	Estimated Quantity	Unit	Unit Price	Total Price
Mobilization/Demobilization (5%)	1	LS	$18,500.00	$18,500
Traffic Control	1	LS	$5,000.00	$5,000
8" PVC Gravity Sewer Main	4,400	LF	$36.00	$158,400
Standard 4' Dia. Manhole with Ring and Cover	11	EA	$2,500.00	$27,500
Connection to Existing Manhole	22	EA	$1,000.00	$22,000
Disconnect/Abandon Existing Gravity Sewer	11	LS	$1,000.00	$11,000
Remove/Dispose of Existing Manhole	11	EA	$500.00	$5,500
Water/Nonpotable Water Crossing with Sleeving	11	EA	$1,000.00	$11,000
Sewer Service Line Re-Connection	33	EA	$400.00	$13,200
Asphalt Surface Repair	2,200	LF	$35.00	$77,000
Gravel Surface Repair	2,200	LF	$14.00	$30,800
Post Install Cleaning/Video Inspection	4,400	LF	$2.00	$8,800
Sub-Total Construction Costs				**$388,700**
Contractor Mob/Demob, Bonding, Admin, Insurance (10%)				$38,900
Dewatering (10%)				$38,900
Contingency (25%)				$97,200
Total Construction Costs				**$563,700**
Engineering & Construction Admin. (20%)				$112,700
Administration & Funding (5%)				$28,200
Inflation (4% for 2 Years)				$45,100
TOTAL PROJECT COST				***$749,700***

CITY OF ALBION
2012 WASTEWATER FACILITIES PLAN
PRESSURE SEWER MAIN IMPROVEMENTS
OPINION OF PROBABLE COSTS

Item	Estimated Quantity	Unit	Unit Price	Total Price
Clean Pressure Sewer Main	4,600	LF	$2.00	$9,200
Install Clean-Outs	3	EA	$1,000.00	$3,000
Replace Air-Vacuum Valves/Vaults	3	EA	$11,000.00	$33,000
Bypass Pumping	15	Days	$500.00	$7,500
Sub-Total Construction Costs				**$52,700**
Contractor Mob/Demob, Bonding, Admin, Insurance (10%)				$5,300
Dewatering (10%)				$5,300
Contingency (25%)				$13,200
Total Construction Costs				**$76,500**
Engineering & Construction Admin. (20%)				$15,300
Administration & Funding (5%)				$3,800
Inflation (4% for 2 Years)				$6,100
TOTAL PROJECT COST				***$101,700***

CITY OF ALBION
2012 WASTEWATER FACILITIES PLAN
LIFT STATION
OPINION OF PROBABLE COSTS

Duplex Submersible Lift Station

Item	Estimated Quantity	Unit	Unit Price	Total Price
Site Work and Remove Existing Lift Station				**$9,300**
Site Grading and Clearing	1	LS	$5,000.00	$5,000
Remove and Dispose of Existing Lift Station	1	LS	$3,000.00	$3,000
Bypass Pumping	5	Days	$250.00	$1,300
Wastewater Structures/Vaults				**$31,500**
Wet-Well (8' Diameter, Pre-Cast Concrete)	1	EA	$11,000.00	$11,000
Valve Vault (6' Diameter, Pre-Cast Concrete)	1	EA	$6,000.00	$6,000
Flow Meter Vault (4' Diameter, Pre-Cast Concrete)	1	EA	$2,500.00	$2,500
Access Hatches	4	EA	$3,000.00	$12,000
Mechanical Piping/Equipment				**$51,900**
Non-Clog Submersible Pumps (5-hp)	2	EA	$10,000.00	$20,000
6" Check Valves	2	EA	$2,100.00	$4,200
6" Plug Valves	2	EA	$1,700.00	$3,400
Piping and Fittings	1	LS	$15,000.00	$15,000
Magnetic Flow Meter	1	EA	$5,300.00	$5,300
Jib Crane	1	LS	$4,000.00	$4,000
Electrical/Controls				**$15,000**
Electrical/Controls	1	LS	$15,000.00	$15,000
Back-Up Generator				**$20,000**
Back-Up Generator	1	LS	$15,000.00	$15,000
Transfer Switch	1	LS	$5,000.00	$5,000
1,000 Gallon Diesel Storage Tank				**$17,900**
1,000 Gallon Diesel Storage Tank	1	LS	$10,200.00	$10,200
Tax (6%)	1	LS	$600.00	$600
Contractor Mark-Up (10%)	1	LS	$1,000.00	$1,000
Shipping/Installation (25%)	1	LS	$2,600.00	$2,600
Concrete Pad for Tank	7	CY	$500.00	$3,500
Building for Generator and Panels				**$42,000**
Metal Building (20' x 20')	400	SF	$50.00	$20,000
Footing/Foundation	9	CY	$500.00	$4,500
Floor Slab	15	CY	$500.00	$7,500
HVAC - Building	1	LS	$5,000.00	$5,000
Electrical - Building	1	LS	$5,000.00	$5,000
Sub-Total Construction Costs				**$187,600**
Contractor Mob/Demob, Bonding, Admin, Insurance (10%)				$18,800
Dewatering (10%)				$18,800
Contingency (25%)				$46,900
Total Construction Costs				**$272,100**
Engineering & Construction Admin. (20%)				$54,400
Administration & Funding (5%)				$13,600
Inflation (4% for 2 Years)				$21,800
TOTAL PROJECT COST				***$361,900***

CITY OF ALBION
2012 WASTEWATER FACILITIES PLAN
LIFT STATION
OPINION OF PROBABLE COSTS

Triplex Submersible Lift Station

Item	Estimated Quantity	Unit	Unit Price	Total Price
Site Work and Remove/Dispose of Existing Lift Station				**$11,500**
Site Grading and Clearing	1	LS	$7,000.00	$7,000
Remove and Dispose of Existing Lift Station	1	LS	$3,000.00	$3,000
Bypass Pumping	6	Days	$250.00	$1,500
Wastewater Structures/Vaults				**$45,500**
Wet-Well (12' Diameter, Pre-Cast Concrete)	1	EA	$20,000.00	$20,000
Valve Vault (8' Diameter, Pre-Cast Concrete)	1	EA	$8,000.00	$8,000
Flow Meter Vault (4' Diameter, Pre-Cast Concrete)	1	EA	$2,500.00	$2,500
Access Hatches	5	EA	$3,000.00	$15,000
Mechanical Piping/Equipment				**$71,200**
Non-Clog Submersible Pumps (5-hp)	3	EA	$10,000.00	$30,000
6" Check Valves	3	EA	$2,100.00	$6,300
6" Plug Valves	3	EA	$1,700.00	$5,100
Piping and Fittings	1	LS	$20,000.00	$20,000
Magnetic Flow Meter	1	EA	$5,300.00	$5,300
Jib Crane	1	LS	$4,500.00	$4,500
Electrical/Controls				**$20,000**
Electrical/Controls	1	LS	$20,000.00	$20,000
Back-Up Generator				**$20,000**
Back-Up Generator	1	LS	$15,000.00	$15,000
Transfer Switch	1	LS	$5,000.00	$5,000
1,000 Gallon Diesel Storage Tank				**$17,900**
1,000 Gallon Diesel Storage Tank	1	LS	$10,200.00	$10,200
Tax (6%)	1	LS	$600.00	$600
Contractor Mark-Up (10%)	1	LS	$1,000.00	$1,000
Shipping/Installation (25%)	1	LS	$2,600.00	$2,600
Concrete Pad for Tank	7	CY	$500.00	$3,500
Building				**$42,000**
Metal Building (20' x 20')	400	SF	$50.00	$20,000
Footing/Foundation	9	CY	$500.00	$4,500
Floor Slab	15	CY	$500.00	$7,500
HVAC - Building	1	LS	$5,000.00	$5,000
Electrical - Building	1	LS	$5,000.00	$5,000
Sub-Total Construction Costs				**$228,100**
Contractor Mob/Demob, Bonding, Admin, Insurance (10%)				$22,800
Dewatering (10%)				$22,800
Contingency (25%)				$57,000
Total Construction Costs				**$330,700**
Engineering & Construction Admin. (20%)				$66,100
Administration & Funding (5%)				$16,500
Inflation (4% for 2 Years)				$26,500
TOTAL PROJECT COST				***$439,800***

CITY OF ALBION
2012 WASTEWATER FACILITIES PLAN
LIFT STATION
OPINION OF PROBABLE COSTS

Duplex Wet/Dry-Well Lift Station

Item	Estimated Quantity	Unit	Unit Price	Total Price
Site Work and Remove/Dispose of Existing Lift Station				**$12,800**
Site Grading and Clearing	1	LS	$8,000.00	$8,000
Remove and Dispose of Existing Lift Station	1	LS	$3,000.00	$3,000
Bypass Pumping	7	Days	$250.00	$1,800
Wastewater Structures/Vaults				**$52,500**
Wet-Well (8' Diameter, Pre-Cast Concrete)	1	EA	$11,000.00	$11,000
Dry-Well (10' Diameter, Pre-Cast Concrete)	1	EA	$16,000.00	$16,000
Valve Vault (8' Diameter, Pre-Cast Concrete)	1	EA	$8,000.00	$8,000
Flow Meter Vault (4' Diameter, Pre-Cast Concrete)	1	EA	$2,500.00	$2,500
Access Hatches	5	EA	$3,000.00	$15,000
Mechanical Piping/Equipment				**$54,400**
Non-Clog Submersible Pumps (5-hp)	2	EA	$10,000.00	$20,000
6" Check Valves	2	EA	$2,100.00	$4,200
6" Plug Valves	2	EA	$1,700.00	$3,400
Piping and Fittings	1	LS	$17,500.00	$17,500
Magnetic Flow Meter	1	EA	$5,300.00	$5,300
Jib Crane	1	LS	$4,000.00	$4,000
Electrical/Controls				**$17,500**
Electrical/Controls	1	LS	$17,500.00	$17,500
Back-Up Generator				**$20,000**
Back-Up Generator	1	LS	$15,000.00	$15,000
Transfer Switch	1	LS	$5,000.00	$5,000
1,000 Gallon Diesel Storage Tank				**$17,900**
1,000 Gallon Diesel Storage Tank	1	LS	$10,200.00	$10,200
Tax (6%)	1	LS	$600.00	$600
Contractor Mark-Up (10%)	1	LS	$1,000.00	$1,000
Shipping/Installation (25%)	1	LS	$2,600.00	$2,600
Concrete Pad for Tank	7	CY	$500.00	$3,500
Building				**$42,000**
Metal Building (20' x 20')	400	SF	$50.00	$20,000
Footing/Foundation	9	CY	$500.00	$4,500
Floor Slab	15	CY	$500.00	$7,500
HVAC - Building	1	LS	$5,000.00	$5,000
Electrical - Building	1	LS	$5,000.00	$5,000
Sub-Total Construction Costs				**$217,100**
Contractor Mob/Demob, Bonding, Admin, Insurance (10%)				$21,700
Dewatering (10%)				$21,700
Contingency (25%)				$54,300
Total Construction Costs				**$314,800**
Engineering & Construction Admin. (20%)				$63,000
Administration & Funding (5%)				$15,700
Inflation (4% for 2 Years)				$25,200
TOTAL PROJECT COST				***$418,700***

CITY OF ALBION
2012 WASTEWATER FACILITIES PLAN
OPTIMIZE EXISTING TREATMENT FACILITIES
OPINION OF PROBABLE COSTS

Item	Estimated Quantity	Unit	Unit Price	Total Price
Reconstruct Inlet Force Mains and Valves				**$16,400**
6" Force Main	75	LF	$20.00	$1,500
6" Plug Valves w/ Valve Box and Cover	2	EA	$1,950.00	$3,900
6" Pressure Main Fittings	4	EA	$250.00	$1,000
Gravel Surface Repair	1	LS	$2,500.00	$2,500
Reconstruct Lagoon Embankment	1	LS	$5,000.00	$5,000
Bypass Pumping	10	Days	$250.00	$2,500
Screening Structure and Gravity Inlet Lines				**$84,500**
Screening Structure	1	LS	$20,000.00	$20,000
8" PVC Gravity Sewer Main	1,000	LF	$36.00	$36,000
Connection to Structure	2	EA	$1,000.00	$2,000
Standard 4' Dia. Manhole with Ring and Cover	4	EA	$2,500.00	$10,000
Lagoon Inlet Pads	2	EA	$6,000.00	$12,000
Post Install Cleaning/Video Inspection	1,000	LF	$2.00	$2,000
Bypass Pumping	10	Days	$250.00	$2,500
Replace 8" Outlet Line Valves				**$5,300**
Remove/Dispose of Existing Valves	2	EA	$500.00	$1,000
8" Plug Valves w/ Valve Box and Cover	2	EA	$2,150.00	$4,300
Sub-Total Construction Costs				**$106,200**
Contractor Mob/Demob, Bonding, Admin, Insurance (10%)				$10,600
Contingency (25%)				$26,600
Total Construction Costs				**$143,400**
Engineering & Construction Admin. (20%)				$28,700
Administration & Funding (5%)				$7,200
Inflation (4% for 2 Years)				$11,500
TOTAL PROJECT COST				***$190,800***

Appendix E

Example Lift Station Layouts

CONSTRUCTION NOTES:

1. PUMP STATION MECHANICAL MATERIAL/EQUIPMENT SHOWN IS SCHEMATIC. CONTRACTOR IS RESPONSIBLE FOR CORRECT QUANTITIES. CONSULT MANUFACTURERS DETAIL DRAWINGS FOR DIMENSIONS AND INSTALLATION DETAILS. VERIFY ALL DIMENSIONS (BOTH VERTICAL AND HORIZONTAL). VERIFY MANUFACTURERS CONNECTION DETAILS AND INSTALLATION REQUIREMENTS. PROVIDE A DIMENSIONED DRAWING SHOWING ALL VALVES, GATES, FITTINGS, PIPE SPOOLS, AND PUMP CONNECTIONS WITH SHOP DRAWING SUBMITTAL.
2. INITIAL LEVEL CONTROLLER SETTINGS (FIELD ADJUST TO OPTIMIZE PERFORMANCE AS REQUIRED):

ELEVATION (FEET)	CONTROLLER FUNCTION FOR RISING LEVEL	CONTROLLER FUNCTION FOR FALLING LEVEL
XXX.XX	HIGH WATER ALARM ON	HIGH WATER ALARM OFF
XXX.XX	LAG PUMP ON	
XXX.XX	LEAD PUMP ON	
XXX.XX		BOTH PUMPS OFF AND ALTERANATE LEAD/LAG
XXX.XX	LOW WATER ALARM OFF	LOW WATER ALARM ON

3. INSTALL PUMP AND ALL RELATED PUMP EQUIPMENT IN STRICT ACCORDANCE WITH THE DRAWINGS, SPECIFICATIONS, AND MANUFACTURERS RECOMMENDATIONS.
4. PROTECT BUILDINGS, FENCES, CURBS, AND SIDEWALKS ADJACENT TO THE SITE, UNLESS NOTED OTHERWISE. DAMAGE BY CONTRACTORS OPERATIONS SHALL BE REPAIRED AT CONTRACTORS EXPENSE.
5. UNLESS NOTED OTHERWISE, ALL PIPING AND FITTINGS FROM THE PUMPS THROUGH THE VALVE VAULT SHALL BE DUCTILE IRON AND HAVE A 2-PART HIGH BUILD COAL TAR EPOXY COATING (40 MIL THICKNESS) ON INTERIOR AND EXTERIOR SURFACES.
6. ALL BOLTS, NUTS, WASHERS, FASTENERS, ETC. SHALL BE STAINLESS STEEL.
7. INSTALL EXPANSION JOINT MATERIAL BETWEEN CONCRETE STRUCTURES AND ANY CONCRETE SLABS.
8. NOT ALL FEATURES ARE SHOWN IN BOTH PLAN AND SECTION VIEWS FOR CLARITY.

LEGEND:

(1) SEE NEXT SHEET FOR MISCELLANEOUS COMMENTS. DELETE BEFORE COMPLETING DRAWING.

XXXXXX OPTIONAL ITEMS ARE SHOWN IN RED. DELETE THOSE NOT USED BEFORE COMPLETING DRAWING.

KEYED NOTES:

(1) SUBMERSIBLE PUMP
(2) 4" PUMP QUICK DISCONNECT DISCHARGE ELBOW AND MOUNTING BASE WITH EPOXY-SET ST.ST. ANCHOR BOLTS, VERIFY SIZE WITH PUMP MFTR.
(3) 4" UNI-FLANGE
(4) 4" RESTRAINED FLANGE COUPLING ADAPTER
(5) 4" PIPE SPOOL (FLGxPE)
(6) 4" 90° LONG RADIUS ELBOW, FLGxFLG
(7) WATERTIGHT WALL PENETRATION, MANHOLE ADAPTER "A-LOK", "KOR-N-SEAL" OR EQUIVALENT
(8) 4" PIPE SPOOL (FLGxPE)
(9) DUAL 4" FLEXIBLE SLEEVE-TYPE PIPE COUPLINGS
(10) 4" PIPE SPOOL (PExPE)
(11) 4" RUBBER FLAPPER SWING CHECK VALVE (EPOXY-LINED) (FLGxFLG)
(12) 4"x4" CROSS (FLG)
(13) STAINLESS STEEL VALVE/PIPE SUPPORT, SEE DETAIL
(14) 6'-0"x6'-0"x6'-6" HIGH PRECAST CONCRETE VALVE VAULT WITH PLASTIC COATED STEPS. HS-20 RATED. MODIFY OPENINGS AS REQ'D TO ACCOMMODATE PIPING AND ACCESS HATCH AS SHOWN. COAT EXTERIOR WITH WATERPROOFING TREATMENT.
(15) ELECTRICAL OUTLET, SEE ELECTRICAL DRAWINGS
(16) 72"ø PRECAST CONCRETE MANHOLE HS-20 RATED WITH 12" THICK MONOLITHIC BASE. COAT EXTERIOR WITH WATERPROOF TREATMENT.
(17) STAINLESS STEEL PUMP REMOVAL SYSTEM, COMPLETE WITH MOUNTING BRACKETS AND INTERMEDIATE SUPPORT BRACES.
(18) 4" PIPE SPOOL (FLGxPE)
(19) SCREENED VENT, SEE DETAIL
(20) 8" INLET SEWER
(21) 4" PLUG VALVE WITH HANDWHEEL (FLGxFLG)
(22) 30"x48" MIN. ALUMINUM DOUBLE LEAF ACCESS DOOR WITH STAINLESS STEEL HARDWARE - OPENING DIMENSIONS AND DOOR LOCATION SHALL BE IN ACCORDANCE WITH PUMP MANUFACTURERS REQUIREMENTS. HATCH SHALL BE HS-20 TRAFFIC RATED AND WATER-TIGHT. PROVIDE RECESSED, LOCKABLE HASP COVERED WITH HINGED LID FLUSH WITH SURFACE. INSTALL DOOR SUCH THAT ENTRY SYSTEM IS NOT IN CONFLICT WITH DOOR
(23) 4" PIPE SPOOL (FLGxPE)
(24) CABLE SUPPORT BRACKET. SEE DETAIL. SEE ELECTRICAL DRAWINGS FOR ADDITIONAL REQUIREMENTS.
(25) 30"x30" MIN. ALUMINUM SINGLE LEAF ACCESS DOOR WITH STAINLESS STEEL HARDWARE. HATCH SHALL BE HS-20 TRAFFIC RATED AND WATER-TIGHT. PROVIDE RECESSED, LOCKABLE HASP COVERED WITH HINGED LID FLUSH WITH SURFACE. INSTALL DOOR SUCH THAT ENTRY SYSTEM IS NOT IN CONFLICT WITH DOOR. PLUMB HATCH RIM DRAIN TO VAULT FLOOR DRAIN.
(26) STAINLESS STEEL LIFTING CHAIN OR CABLE (MIN. STRENGTH 6,000 LBS.) WITH S.S. CLEVIS FITTING AT EACH END
(27) THRUST RESTRAINT PIPE SUPPORT, SEE DETAIL.
(28) MANHOLE JOINT WITH EXTRUDED BUTYL RUBBER SEAL. GROUT JOINT INSIDE AND OUT, TYP.
(29) CRUSHED SURFACING BASE COMPACTED TO 95%
(30) 4" CAM-LOCK FITTING WITH PRESSURE CAP
(31) 24"ø LOCKABLE MANHOLE LID CAST IN TOP SLAB. CENTER OVER CAM-LOCK FITTING
(32) PIPE BRACE. SEE DETAIL.
(33) LOCATING WIRE. PROVIDE SUFFICIENT LENGTH FOR WIRE TO BE PULLED TO GROUND SURFACE.
(34) 4" FLAP GATE
(35) 8"x4" ECCENTRIC REDUCER, (FLGxFLG)
(36) 8" 90° ELBOW (FLGxFLG) WITH FLANGED COUPLING ADAPTER AND THRUST BLOCK
(37) 6"ø CAST IRON FLOOR DRAIN, P-TRAP, & 4" PVC DRAIN TO WETWELL SLOPED AT 2% MIN.
(38) ULTRASONIC LEVEL TRANSMITTER AND BRACKET. SEE DETAIL. SEE ELECTRICAL DRAWING FOR ADDITIONAL REQUIREMENTS.
(39) ITT FLYGT MIX FLUSH VALVE, ONE PUMP ONLY
(40) CONFINED SPACE ENTRY SYSTEM-LIFTING SUPPORT PEDESTAL RECESSED UNI-SLEEVE AS MANUFACTURED BY LIFE PROTECTION, INC., 22-360 KEWATIN ST., WINNIPEG, CANADA (204) 633-0511 NO SUBSTITUTE. INSTALL IN ACCORDANCE WITH MANUFACTURERS RECOMMENDATIONS.
(41) LINE INTERIOR WITH T-LOCK

NOTE

THE LOCATION OF ALL EXISTING UNDERGROUND UTILITIES IS SHOWN IN AN APPROXIMATE WAY ONLY. THE CONTRACTOR SHALL DETERMINE THE EXACT LOCATION OF ALL EXISTING UTILITIES BEFORE COMMENCING WORK. HE AGREES TO BE FULLY RESPONSIBLE FOR ANY AND ALL DAMAGES WHICH MIGHT BE OCCASIONED BY HIS FAILURE TO EXACTLY LOCATE AND PRESERVE ANY AND ALL UNDERGROUND UTILITIES.

CALL 48 HOURS BEFORE YOU DIG
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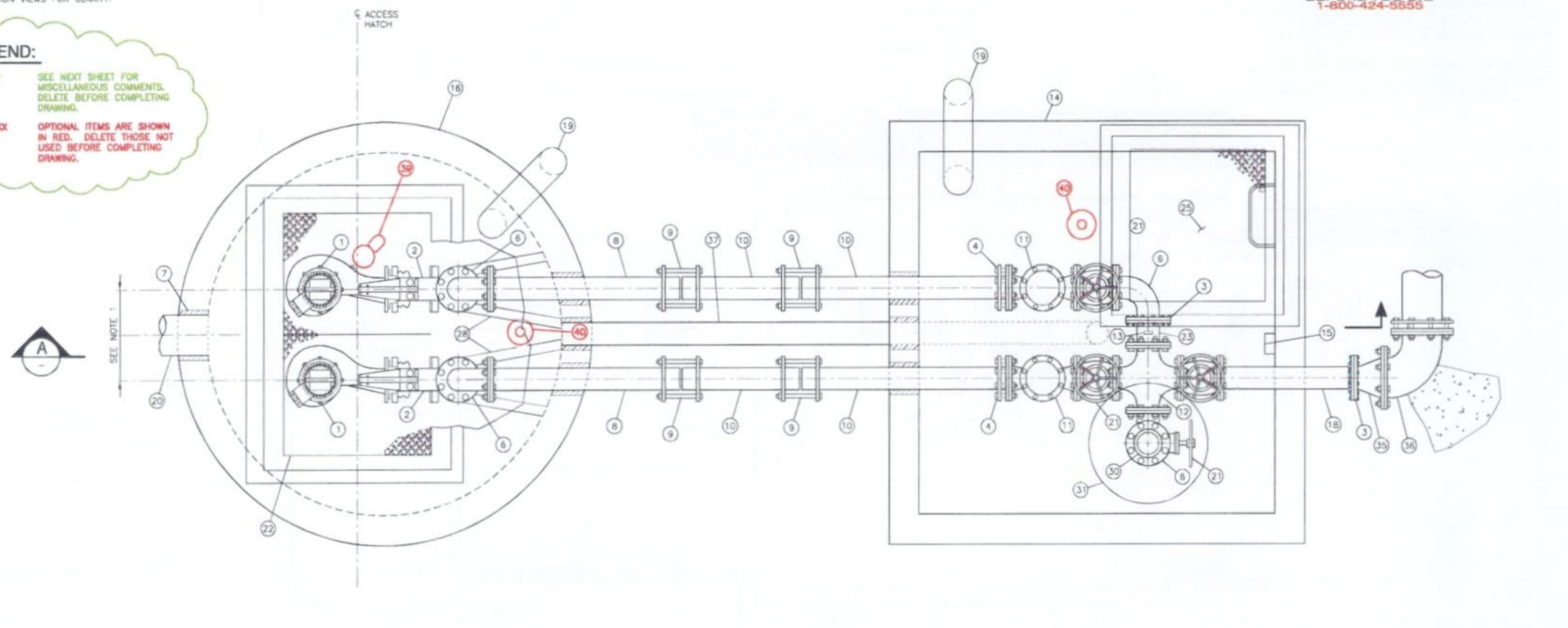

LIFT STATION - MECHANICAL PLAN

1 0 1 2
SCALE IN FEET

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Kennewick, Washington 99336
Phone: 509.783.2144
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PRELIMINARY PLANS
NOT FOR CONSTRUCTION

REVISION

NO.	DESCRIPTION	BY	APR.	DATE

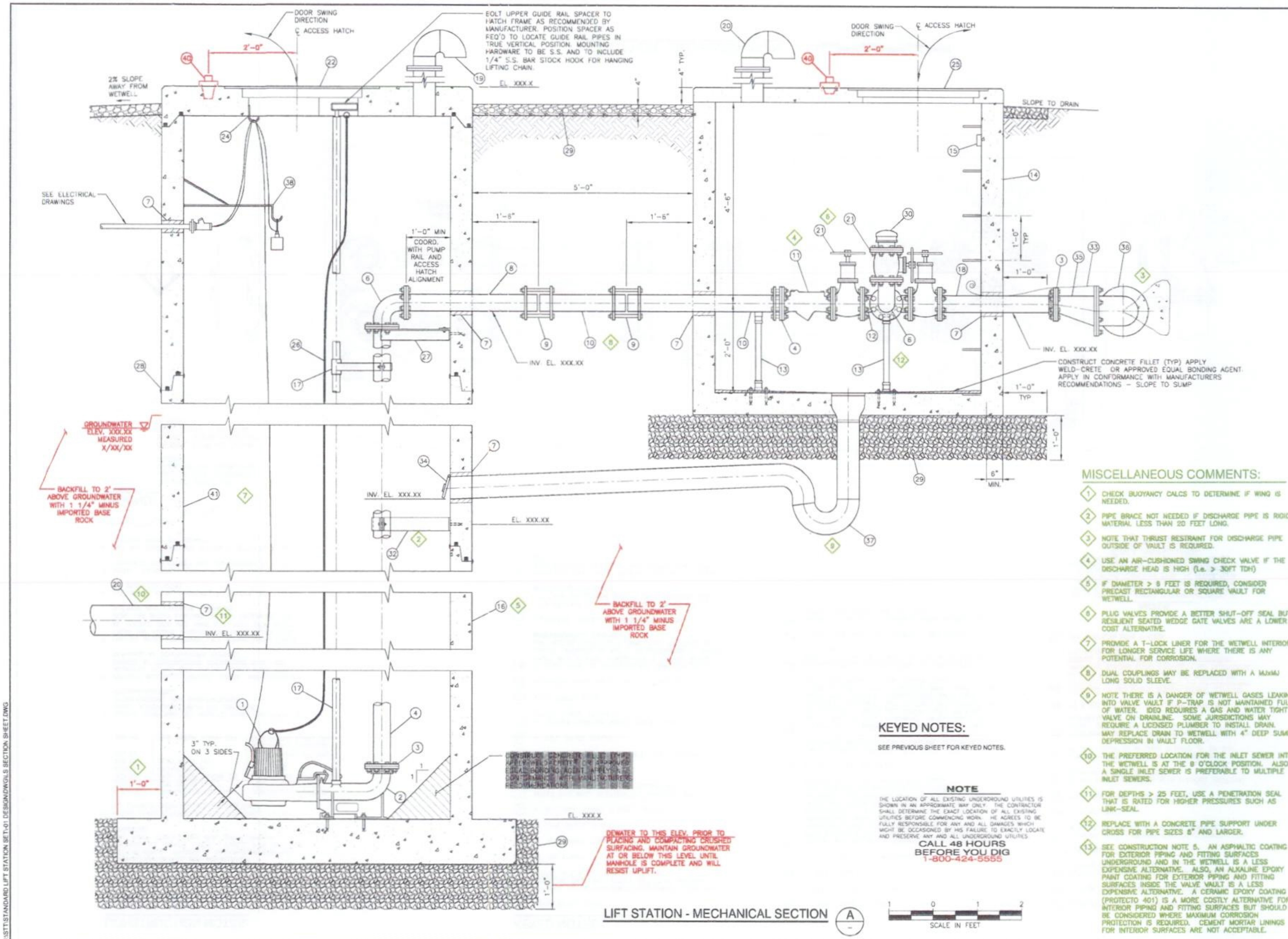
J-U-B ENGINEERS, INC.
TYPICAL LIFT STATION

LIFT STATION
MECHANICAL PLAN

FILE: LS MECHANICAL SHEET
DRAWN BY:
DESIGN BY:
CHECKED BY:
SCALE OF SHEET
HOR SCALE: AS SHOWN
VER SCALE: NONE
LAST UPDATED: 1/23/2007
DATE PLOTTED: 1/23/2007

SHEET XX OF XX

U:\STT\STANDARD LIFT STATION SET\-01 DESIGN\DWG\LS MECHANICAL SHEET.DWG



LIFT STATION - MECHANICAL SECTION (A)

SCALE IN FEET

MISCELLANEOUS COMMENTS:

1. CHECK BUOYANCY CALCS TO DETERMINE IF WING IS NEEDED.
2. PIPE BRACE NOT NEEDED IF DISCHARGE PIPE IS RIGID MATERIAL LESS THAN 20 FEET LONG.
3. NOTE THAT THRUST RESTRAINT FOR DISCHARGE PIPE OUTSIDE OF VAULT IS REQUIRED.
4. USE AN AIR-CUSHIONED SWING CHECK VALVE IF THE DISCHARGE HEAD IS HIGH (i.e. > 30FT TDH)
5. IF DIAMETER > 8 FEET IS REQUIRED, CONSIDER PRECAST RECTANGULAR OR SQUARE VAULT FOR WETWELL.
6. PLUG VALVES PROVIDE A BETTER SHUT-OFF SEAL BUT RESILIENT SEATED WEDGE GATE VALVES ARE A LOWER COST ALTERNATIVE.
7. PROVIDE A T-LOCK LINER FOR THE WETWELL INTERIOR FOR LONGER SERVICE LIFE WHERE THERE IS ANY POTENTIAL FOR CORROSION.
8. DUAL COUPLINGS MAY BE REPLACED WITH A MJxMJ LONG SOLID SLEEVE.
9. NOTE THERE IS A DANGER OF WETWELL GASES LEAKING INTO VALVE VAULT IF P-TRAP IS NOT MAINTAINED FULL OF WATER. IDEQ REQUIRES A GAS AND WATER TIGHT VALVE ON DRAINLINE. SOME JURISDICTIONS MAY REQUIRE A LICENSED PLUMBER TO INSTALL DRAIN. MAY REPLACE DRAIN TO WETWELL WITH 4" DEEP SUMP DEPRESSION IN VAULT FLOOR.
10. THE PREFERRED LOCATION FOR THE INLET SEWER INTO THE WETWELL IS AT THE 8 O'CLOCK POSITION. ALSO, A SINGLE INLET SEWER IS PREFERABLE TO MULTIPLE INLET SEWERS.
11. FOR DEPTHS > 25 FEET, USE A PENETRATION SEAL THAT IS RATED FOR HIGHER PRESSURES SUCH AS LINK-SEAL.
12. REPLACE WITH A CONCRETE PIPE SUPPORT UNDER CROSS FOR PIPE SIZES 8" AND LARGER.
13. SEE CONSTRUCTION NOTE 5. AN ASPHALTIC COATING FOR EXTERIOR PIPING AND FITTING SURFACES UNDERGROUND AND IN THE WETWELL IS A LESS EXPENSIVE ALTERNATIVE. ALSO, AN ALKALINE EPOXY PAINT COATING FOR EXTERIOR PIPING AND FITTING SURFACES INSIDE THE VALVE VAULT IS A LESS EXPENSIVE ALTERNATIVE. A CERAMIC EPOXY COATING (PROTECTO 401) IS A MORE COSTLY ALTERNATIVE FOR INTERIOR PIPING AND FITTING SURFACES BUT SHOULD BE CONSIDERED WHERE MAXIMUM CORROSION PROTECTION IS REQUIRED. CEMENT MORTAR LININGS FOR INTERIOR SURFACES ARE NOT ACCEPTABLE.

KEYED NOTES:

SEE PREVIOUS SHEET FOR KEYED NOTES.

NOTE

THE LOCATION OF ALL EXISTING UNDERGROUND UTILITIES IS SHOWN IN AN APPROXIMATE WAY ONLY. THE CONTRACTOR SHALL DETERMINE THE EXACT LOCATION OF ALL EXISTING UTILITIES BEFORE COMMENCING WORK. HE AGREES TO BE FULLY RESPONSIBLE FOR ANY AND ALL DAMAGES WHICH MIGHT BE OCCASIONED BY HIS FAILURE TO EXACTLY LOCATE AND PRESERVE ANY AND ALL UNDERGROUND UTILITIES.

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PRELIMINARY PLANS
NOT FOR CONSTRUCTION

REVISION				
NO.	DESCRIPTION	BY	APR.	DATE

J-U-B ENGINEERS, INC.
TYPICAL LIFT STATION

LIFT STATION
MECHANICAL SECTION

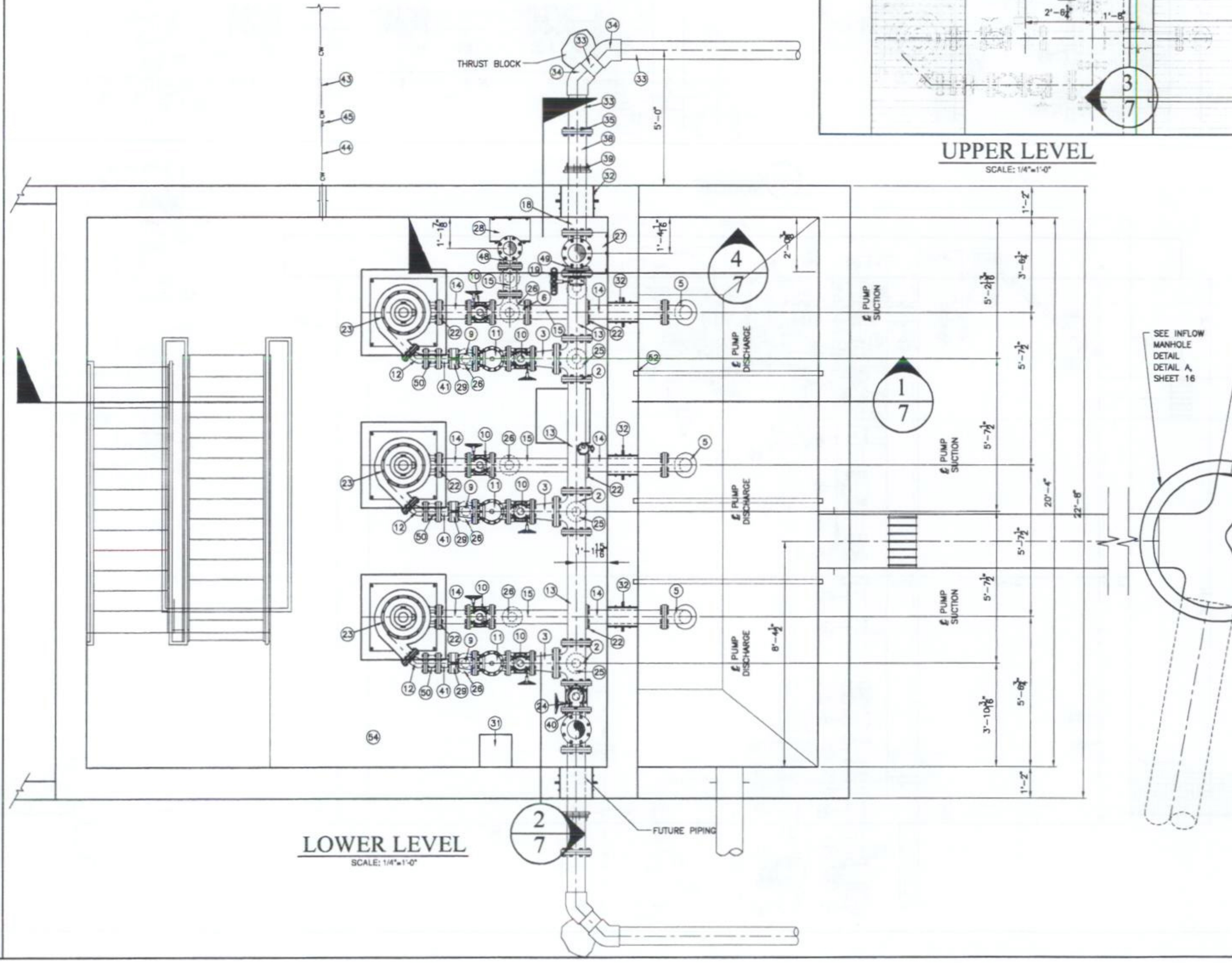
FILE: LS SECTION SHEET
DRAWN BY:
DESIGN BY:
CHECKED BY:
SCALE OF SHEET
HOR SCALE: AS SHOWN
VER SCALE: NONE
LAST UPDATED: 1/23/2007
DATE PLOTTED: 1/23/2007

SHEET XX OF XX

PUMP AND PIPING GENERAL NOTES

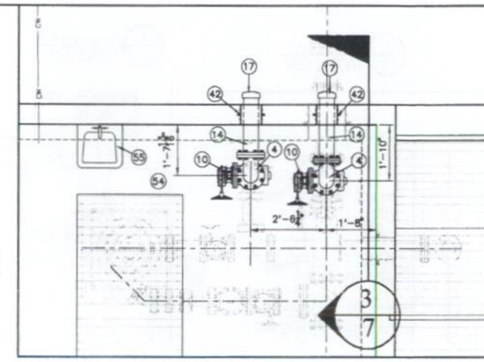
1. NOT USED
2. 8" D.I. TEE (FLG)
3. 8"x6" D.I. ECC. RED. (FLG)
4. 6" 90° D.I. BEND (FLG)
5. 6" 90° D.I. BEND (FLG x FLARE)
6. 6" D.I. TEE (FLG)
7. NOT USED
8. NOT USED
9. 6"x4" ECC. REDUCER (FLG x FLG)
10. 6" DZURIK ECCENTRIC PLUG VALVE W/ GEAR OPERATOR (FLG)
11. 6" VALMATIC SWING FLEX CHECK VALVE W/ LEVER POSITION INDICATOR (FLG) (OR EQUAL)
12. 4" 45° D.I. BEND (FLG)
13. 8" D.I. PIPE SPOOL (FLG x FLG)
14. 6" D.I. PIPE SPOOL (FLG x PE)
15. 6" D.I. PIPE SPOOL (FLG x FLG)
16. 2" P-TRAP FROM SINK
17. 6" ALUMINUM MALE CAMLOCK FITTING WITH CAMLOCK PLUG
18. 8" D.I. PIPE SPOOL (FLG x PE)
19. 6" DZURICK ECCENTRIC PLUG VALVE W/ CHAIN OPERATOR (FLG) (OR EQUAL)
20. 8"x6" D.I. CONC. RED. (FLG)
21. 8" D.I. FLANGE COUPLING ADAPTER RESTRAIN W/ 6 S.S. TIE RODS
22. 6" D.I. FLANGE COUPLING ADAPTER RESTRAIN W/ 6 S.S. TIE RODS
23. FAIRBANKS MORSE 4" 5434 WD PUMP WITH 15HP MOTOR
24. 8" DZURICK ECCENTRIC PLUG VALVE W/ GEAR OPERATOR (FLG)
25. 8" ADJUSTABLE PIPE SUPPORT (SEE DETAIL F SHEET 11)
26. 6" ADJUSTABLE PIPE SUPPORT (SEE DETAIL F SHEET 11)
27. 8" PIPE WALL ANCHOR (SEE DETAIL B SHEET 11)
28. 6" PIPE WALL ANCHOR (SEE DETAIL B SHEET 11)
29. RED VALVE SERIES 40 FLANGED PRESSURE SENSOR, CARBON STEEL BODY AND FLANGES, NEOPRENE LINER, WITH TEE FOR PRESSURE SENSOR, PRESSURE SWITCH AND STANDARD 2½" GAUGE, 0 TO 100 P.S.I. (OR EQUAL)
30. 6" ABB MAGMASTER MAGNETIC FLOW METER
31. 14½" SQUARE ALUMINUM AIR DUCT. EXTEND TO 12" ABOVE VAULT FLOOR.
32. LINK-SEAL WALL PENETRATION (SEE DETAIL D SHEET 11)
33. 8" HDPE IPS DR 17 (FUSION WELDED)
34. 8" HDPE 45° BEND (FUSION WELDED)
35. 8" HDPE FLANGE (FUSION WELDED)
36. 4" D.I. PIPE SPOOL (FLG x FLG)
37. SUMP PUMP (SEE DETAIL A SHEET 11)
38. 8" D.I. ADAPTER (FLG x MJ)
39. 8" MEGALUG RESTRAINT
40. 8" D.I. BLIND FLG.
41. 4" D.I. PIPE SPOOL (FLG x PE)
42. FLOOR PENETRATION (SEE DETAIL C SHEET 11)
43. 1" POLY CULINARY WATER SERVICE
44. 1" TYPE K COPPER CULINARY WATER SERVICE
45. COMPRESSION FITTING
46. HOSE BIB WITH BACKFLOW PREVENTER
47. WALL ANCHOR (SEE DETAIL B SHEET 11)
48. 6" D.I. 90° BASE BEND (FLG x FLG)
49. 8" D.I. 90° BASE BEND (FLG x FLG)
50. 4" D.I. FLANGE COUPLING ADAPTER RESTRAIN W/ 6" S.S. TIE RODS
51. 2" ABS DRAIN LINE FROM SINK
52. 2"x2" CONCRETE KEY WAY
53. COAT ALL CONCRETE SURFACES IN WET WELL AND INFLOW MANHOLE WITH TNEMEC 435 PERMACOAT, 2 COATS TOTALING 40 MIL DFT OR APPROVED EQUAL
54. PAINT CONCRETE FLOOR WITH EPOXY SEALANT, ONE COAT PRIMER/SEALER 7.0 TO 8.0 MILS DFT AND ONE COAT NON-SLIP EPOXY FINISH COAT (40-60 SQ. FT. PER GALLON)
55. SINGLE COMPARTMENT INDUSTRIAL STAINLESS STEEL SINK, 18x18x14 WITH 7" S.S. BACKSPLASH WITH FAUCET

F:\PROJECTS\JUB\CLINTON\55-05-074 SEWER LIFT\CAD\LIFT STATION\CAD\MECHANICAL 11-08-06.DWG



LOWER LEVEL

SCALE: 1/4"=1'-0"



UPPER LEVEL

SCALE: 1/4"=1'-0"

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J-U-B ENGINEERS, Inc.
466 North 900 West
Kaysville, Utah 84037

Phone: 801.547.0393
Fax: 801.547.0397
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FINAL PLANS

APPROVED FOR CONSTRUCTION

REVISION

NO.	DESCRIPTION	BY	APPR.	DATE

CRANEFIELD SEWER LIFT STATION PROJECT
CLINTON CITY CORPORATION

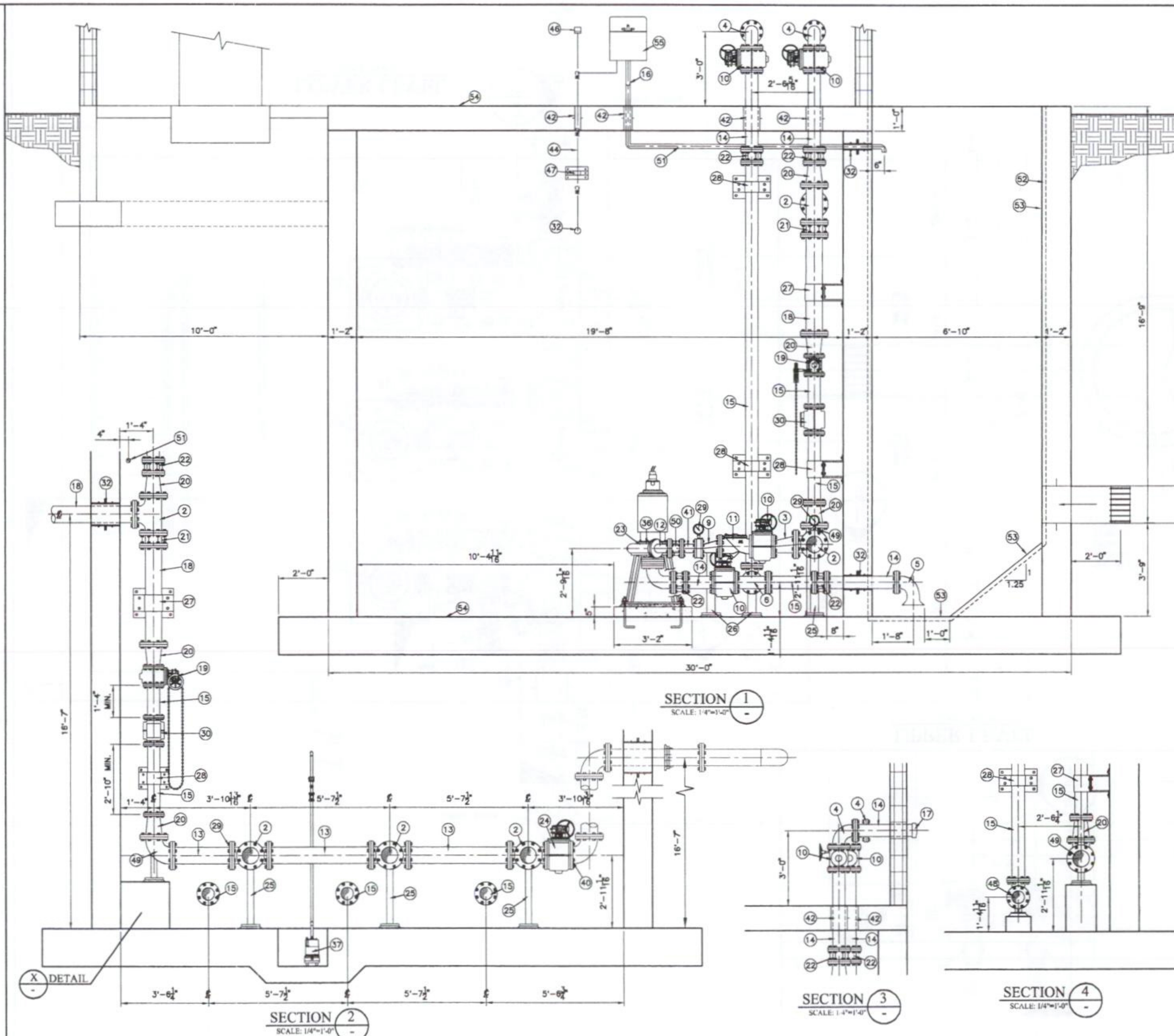
MAIN AND PUMP LEVEL PIPING PLAN

FILE: MECHANICAL 11-08-06
DRAWN BY: ELC
DESIGN BY: PJT
CHECKED BY: BKW
SCALE OF SHEET
HOR SCALE: AS SHOWN
VER SCALE: NONE
LAST UPDATED: 3/1/2007
DATE PLOTTED: 7/2/2007

SHEET 6 OF 17

PUMP AND PIPING GENERAL NOTES

1. NOT USED
2. 8" D.I. TEE (FLG)
3. 8"x6" D.I. ECC. RED. (FLG)
4. 6" 90° D.I. BEND (FLG)
5. 6" 90° D.I. BEND (FLG x FLARE)
6. 6" D.I. TEE (FLG)
7. NOT USED
8. NOT USED
9. 6"x4" ECC. REDUCER (FLG x FLG)
10. 6" DZURIK ECCENTRIC PLUG VALVE W/ GEAR OPERATOR (FLG)
11. 6" VALMATIC SWING FLEX CHECK VALVE W/ LEVER POSITION INDICATOR (FLG) (OR EQUAL)
12. 4" 45° D.I. BEND (FLG)
13. 8" D.I. PIPE SPOOL (FLG x FLG)
14. 6" D.I. PIPE SPOOL (FLG x PE)
15. 6" D.I. PIPE SPOOL (FLG x FLG)
16. 2" P-TRAP FROM SINK
17. 6" ALUMINUM MALE CAMLOCK FITTING WITH CAMLOCK PLUG
18. 8" D.I. PIPE SPOOL (FLG x PE)
19. 6" DZURICK ECCENTRIC PLUG VALVE W/ CHAIN OPERATOR (FLG) (OR EQUAL)
20. 8"x6" D.I. CONC. RED. (FLG)
21. 8" D.I. FLANGE COUPLING ADAPTER RESTRAIN W/ 6 S.S. TIE RODS
22. 6" D.I. FLANGE COUPLING ADAPTER RESTRAIN W/ 6 S.S. TIE RODS
23. FAIRBANKS MORSE 4" 5434 WD PUMP WITH 15HP MOTOR
24. 8" DZURICK ECCENTRIC PLUG VALVE W/ GEAR OPERATOR (FLG)
25. 8" ADJUSTABLE PIPE SUPPORT (SEE DETAIL F SHEET 11)
26. 6" ADJUSTABLE PIPE SUPPORT (SEE DETAIL F SHEET 11)
27. 8" PIPE WALL ANCHOR (SEE DETAIL B SHEET 11)
28. 6" PIPE WALL ANCHOR (SEE DETAIL B SHEET 11)
29. RED VALVE SERIES 40 FLANGED PRESSURE SENSOR, CARBON STEEL BODY AND FLANGES, NEOPRENE LINER, WITH TEE FOR PRESSURE SENSOR, PRESSURE SWITCH AND STANDARD 2½" GAUGE, 0 TO 100 P.S.I. (OR EQUAL)
30. 6" ABB MAGMASTER MAGNETIC FLOW METER
31. 14½" SQUARE ALUMINUM AIR DUCT. EXTEND TO 12" ABOVE VAULT FLOOR.
32. LINK-SEAL WALL PENETRATION (SEE DETAIL D SHEET 11)
33. 8" HDPE IPS DR 17 (FUSION WELDED)
34. 8" HDPE 45° BEND (FUSION WELDED)
35. 8" HDPE FLANGE (FUSION WELDED)
36. 4" D.I. PIPE SPOOL (FLG x FLG)
37. SUMP PUMP (SEE DETAIL A SHEET 11)
38. 8" D.I. ADAPTER (FLG x MJ)
39. 8" MEGALUG RESTRAINT
40. 8" D.I. BLIND FLG.
41. 4" D.I. PIPE SPOOL (FLG x PE)
42. FLOOR PENETRATION (SEE DETAIL C SHEET 11)
43. 1" POLY CULINARY WATER SERVICE
44. 1" TYPE K COPPER CULINARY WATER SERVICE
45. COMPRESSION FITTING
46. HOSE BIB WITH BACKFLOW PREVENTER
47. WALL ANCHOR (SEE DETAIL B SHEET 11)
48. 6" D.I. 90° BASE BEND (FLG x FLG)
49. 8" D.I. 90° BASE BEND (FLG x FLG)
50. 4" D.I. FLANGE COUPLING ADAPTER RESTRAIN W/ 6" S.S. TIE RODS
51. 2" ABS DRAIN LINE FROM SINK
52. 2"x2" CONCRETE KEY WAY
53. COAT ALL CONCRETE SURFACES IN WET WELL AND INFLOW MANHOLE WITH TNEMEC 435 PERMACOAT, 2 COATS TOTALING 40 MIL DFT OR APPROVED EQUAL
54. PAINT CONCRETE FLOOR WITH EPOXY SEALANT, ONE COAT PRIMER/SEALER 7.0 TO 8.0 MILS DFT AND ONE COAT NON-SLIP EPOXY FINISH COAT (40-60 SQ. FT. PER GALLON)
55. SINGLE COMPARTMENT INDUSTRIAL STAINLESS STEEL SINK, 18x18x14 WITH 7" S.S. BACKSPLASH WITH FAUCET



SECTION 1 — SCALE: 1/4"=1'-0"

SECTION 2 — SCALE: 1/4"=1'-0"

SECTION 3 — SCALE: 1/4"=1'-0"

SECTION 4 — SCALE: 1/4"=1'-0"

X DETAIL

J-U-B
Engineers • Surveyors • Planners
J-U-B ENGINEERS, Inc.
466 North 900 West
Kaysville, Utah 84037
Phone: 801.547.0393
Fax: 801.547.0397
www.jub.com

FINAL PLANS

APPROVED FOR CONSTRUCTION

NO.	DESCRIPTION	BY	APR.	DATE

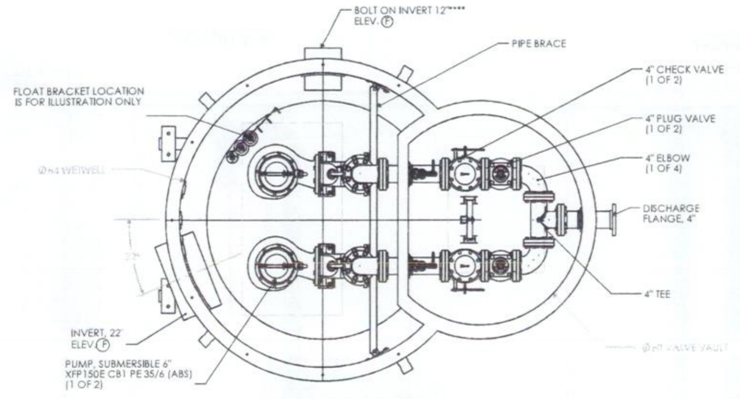
REVISION

CRANEFIELD SEWER LIFT STATION PROJECT
CLINTON CITY CORPORATION

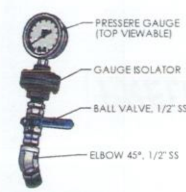
MAIN AND PUMP LEVEL PIPING PLAN

FILE: MECHANICAL 11-08-06
DRAWN BY: ELC
DESIGN BY: PJT
CHECKED BY: BKW
SCALE OF SHEET
HOR SCALE: AS SHOWN
VER SCALE: NONE
LAST UPDATED: 3/1/2007
DATE PLOTTED: 7/2/2007

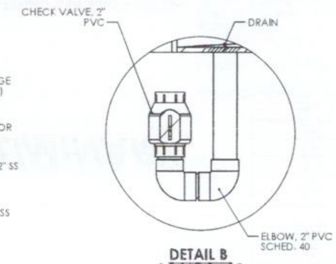
SHEET 7 OF 17



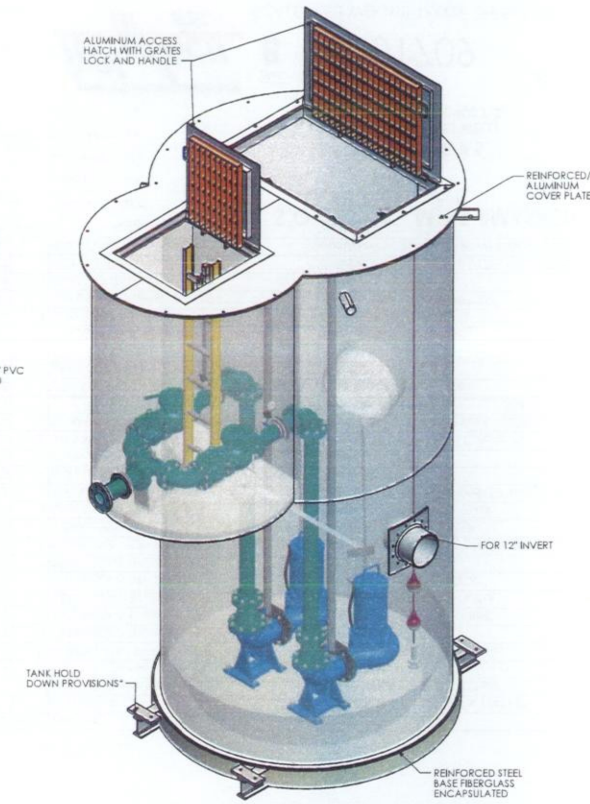
TOP VIEW DETAIL
SCALE 1:35



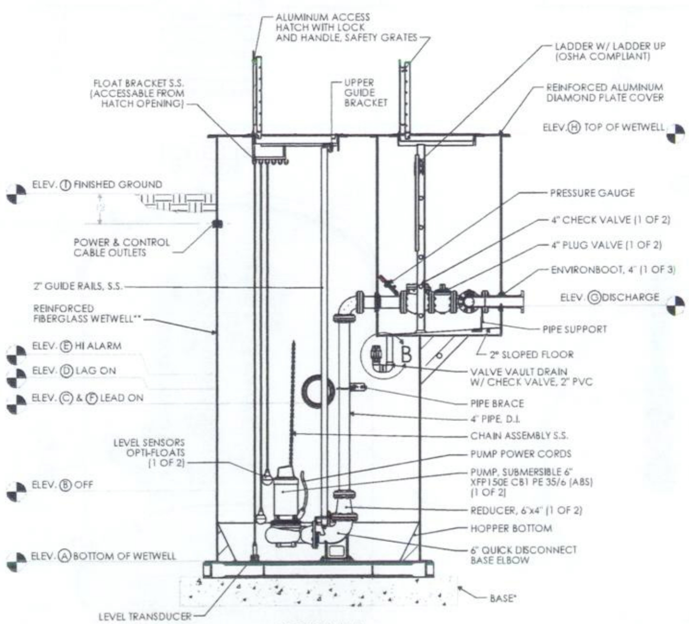
PRESSURE GAUGE
SCALE 1:8



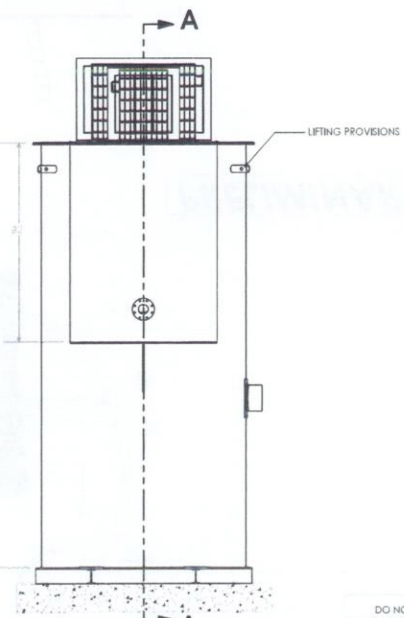
DETAIL B
SCALE 1 : 12



ISO VIEW



SECTION A-A
SCALE 1 : 50



FRONT VIEW

NOTES:

SOME FIELD ASSEMBLY MAYBE REQUIRED
* BY CONTRACTOR
** Ø 7FT x 14.5FT TALL (FROM BOTTOM OF WETWELL)
*** INVERT SIDE, SIZE & LOCATION T.B.D. BY CUSTOMER

CUSTOMER DRAWING APPROVAL	
APPROVED BY:	
INTIAL:	
DATE:	

	NAME	INIT.	DATE
DRAWN	MAO		06/20/11
CHECKED	A.B.		
SALES APPR.	G.C.		
PURCH. APPR.			
MFG APPR.			

S.O.# 91775 M.T.S.M67940

CONTRACTORS NW

ROSALIA P.S.

-

HYDRONIX MODEL 421 PUMP STATION

SIZE B DWG. NO. M01935 REV

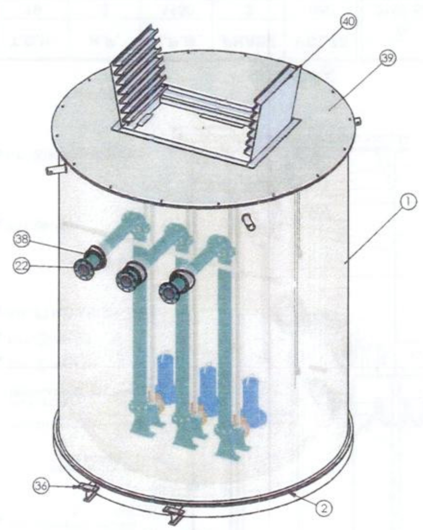
SCALE: NTS WEIGHT: ~7,800 SHEET 1 OF 1

DO NOT SCALE DRAWING

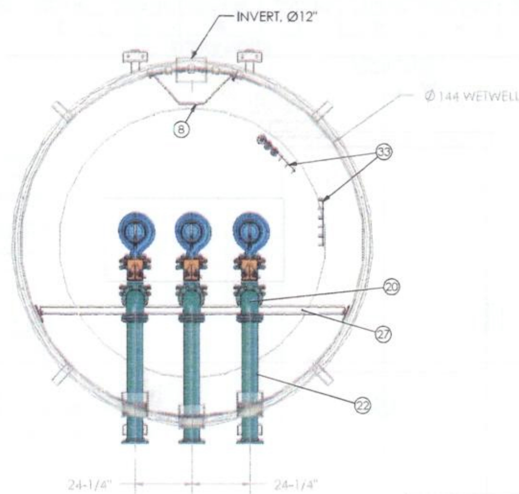
PumpTech Inc.

G.P.M.	T.D.H.	H.P.	R.P.M.	PHASE	VOLTS	ELEVATIONS A	B	C	D	E	F	G	H	I
500	19	5	1150	3	460	2198.50	2200.90	2203.90	2204.40	2204.90	2203.90	2207.20	2213.00	2211.00

PRINT DATE: 6/21/2011 8:25 AM



ISO VIEW
SCALE: 1:60

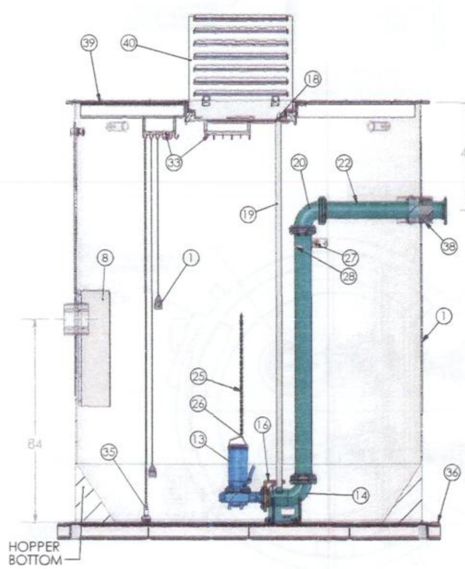


TOP VIEW

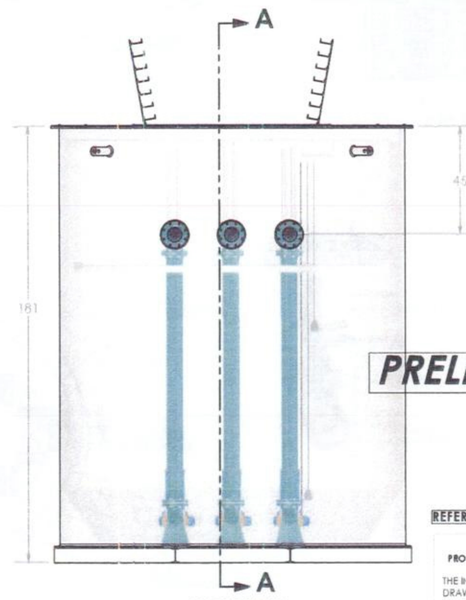
NOTES:
- NOT ALL OBJECTS IN THE B.O.M. ARE SHOWN FOR CLARITY
- HATCH IS H-20 RATED
- TANK IS Ø 12' x 14.5' TALL
- SOME ONSITE ASSEMBLY MAYBE REQUIRED

PRELIMINARY

ITEM NO.	QTY.	PART NO.	DESCRIPTION
1	1	M01690(MFC)	Valve Vault Tank, Ø144" x 174" FRP
2	1	M01692 (PumpTech, Inc.)	Base Plate, 0.50" xØ150" Steel
8	1	M01711 (General Sheet Metal)	Flow Deflector, SS (IF AVAILABLE)
13	3	XFP100E CB1 PE75/4 (ABS)	Pump, Submersible 4" ANSI Flg
14	3	13442-006-3 (Hydromatic)	Discharge Elbow, 6"
15	3	51833-004-5 (Hydromatic)	Discharge Elbow Accy Kit, 6"
16	3	13443-146-2 (Hydromatic)	Discharge Flange, 4"x 6" MTM
17	3	51834-006-5 (Hydromatic)	Sealing Flange Accy Kit, 6"
18	3	13444-004-5 (Hydromatic)	Upper Guide Bracket
19	6	Guide Rail , Ø2" (152" Long) Sched. 40 SS	Guide Rail Ø2" x 152" SS Sched. 40
20	3	Elbow, 90° Short 6" C.I.	Elbow, 90 Straight 6" 125# C.I.
21	3	Spool, Ø6" x 103" Long	Spool, Ø6"x103" FxF D.I. 250#
22	3	6" D.I. Spool Fx Loose End (52" Long)	Spool, Ø6" D.I. Fx Loose End (52" Long) 125#
23	9	6" Red Ring Gasket	Gasket, 6" 150#
24	3	4" Red Ring Gasket	Gasket, 4" 150#
25	15	5/16 in - SS - Chain (Conray) (45FT REQ'D)	Chain, 5/16" SS 1,800lbs Capacity
26	3	5/16in - SS - Shackle (Rigging Supply)	Shackle, 5/16" 316 SS Rated 1000lbs
27	1	____ (PumpTech, Inc.) (IF AVAILABLE)	Pipe Brace, 3x3x0.375x____ 304SS
28	3	U-Bolt 316 SS1-1/2 x 6" pipe (Fastenal)	U-Bolt 316 SS1.50 x 6" pipe
33	2	6AHB (Conery) (BY L2)	6-Float Bracket, 300 SS
34	2	1002170 (SJE Rhombus) (BY L2)	Float Sensor, 30 Sweno
35	1	EVA2000 (EnviroSense)	Submersible Transducer
36	4	M-00508 (PumpTech, Inc.) (IF AVAILABLE)	Anchor bar, 0.75x4x10 SS
37	4	Coupling Adapter, Ø2" NPT S.S.	Coupling Adapter, 2" NPT SS
38	3	LS-475-C-10 (Proctor Sales)	Link Seal, 6" D.I. pipe
39	1	M01693 (PumpTech, Inc.)	Lid Plate, 0.50"x Ø152" Al. Diamond Plate
40	1	AHD 40x85 (USF Fabrication)	Hatch, 34x75 Clear. Al H-20 w/ Safety Grates



SECTION A-A



FRONT VIEW

PRELIMINARY

REFERENCE DRAWING # M01689

PROPRIETARY AND CONFIDENTIAL

UNLESS OTHERWISE SPECIFIED:
DIMENSIONS ARE IN INCHES
TOLERANCES:
FRACTIONAL: +/- 1/4
ANGULAR: +/- 1
ONE PLACE DECIMAL +/- 0.2
TWO PLACE DECIMAL +/- 0.07
INTERPRET GEOMETRIC TOLERANCING PER:
MATERIAL
FINISH
DO NOT SCALE DRAWING

	NAME	INITIALS	DATE
DRAWN	MAO		05/06/10
CHECKED	A.B.		
SALES APPR.	J.D.		
PURCH.APPR.	B.R.		
MFG APPR.	S.B.		

PumpTech inc.

S.O.#83251 M.T.S.#M65430

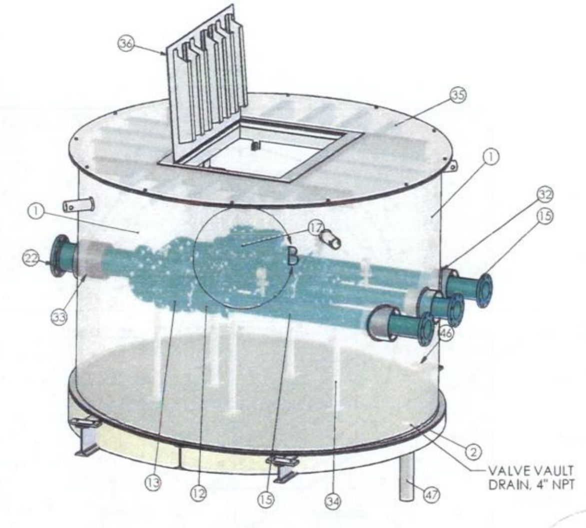
P.T.M.
CITY OF FILER P.S.
CITY OF FILER (WET WELL)
HYDRONIX MODEL 400 P.S.
B.O.M.

SIZE B | DWG. NO. M01709 | REV

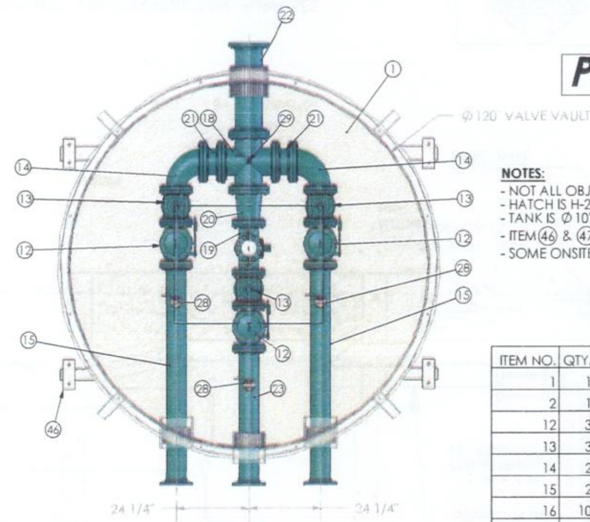
SCALE: 1:50 WEIGHT: 17,000 SHEET 1 OF 1

PRINT DATE: 6/17/2010 11:48

PRELIMINARY



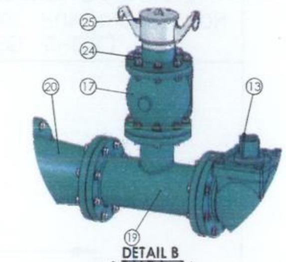
ISO VIEW



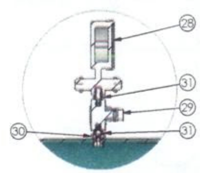
TOP VIEW

NOTES:

- NOT ALL OBJECTS IN THE B.O.M. ARE SHOWN FOR CLARITY
- HATCH IS H-20 RATED
- TANK IS Ø 10' x 7.08' TALL
- ITEM (46) & (47) ARE TO BE SHIPPED LOOSE
- SOME ONSITE ASSEMBLY MAYBE REQUIRED



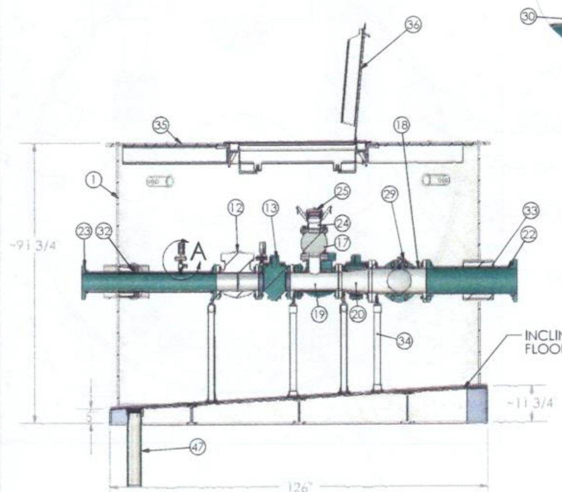
DETAIL B
SCALE 1 : 16



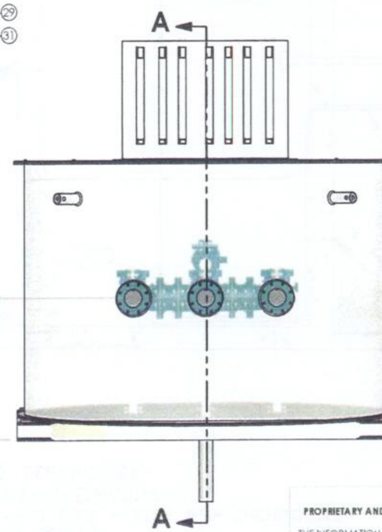
DETAIL A
SCALE 1 : 10

PRELIMINARY

ITEM NO.	QTY.	PART NO.	DESCRIPTION
1	1	M01691 (MFC)	Valve Vault Tank, Ø120" x 85" FRP
2	1	M01701 (PumpTech, Inc.)	Base Plate, 0.50" xØ126" Steel
12	3	Fig. 801 BBW 6" LH (Milliken)	Swing Check Valve 6" W/Outside Lever-Weight LH
13	3	601E-6" (Milliken Valve)	Plug Valve 6"
14	2	8"x6" Reducing Elbow C.I.	Taper Reducing Elbow 90° 8x6 125# Flg C.I.
15	2	6" D.I. Spool Fx Loose End (72" Long)	Spool, Ø6" D.I. Fx Loose End (72" Long) 125#
16	10	Ø6" Gasket	Gasket, Ring Flange Ø6" 150# 1/8" Thk.
17	1	601E 4" 125# Flg - (Milliken)	Plug Valve 4" 125# Flg
18	1	8" Cross C.I.	Cross, 8" 125# C.I.
19	1	Fig 812 (Grinnell)	Tee, Reducing 6x6x4 125# Flg
20	1	8"x6" Concentric Reducer C.I.	Concentric Reducer, 8x6 C.I. 125# Flg.
21	2	Ø8" D.I. Pipe FxF (6" Long)	Spool, Ø8" x 6" xFxF D.I. 125#
22	1	8" D.I. Pipe FxLoose End (30" Long)	Spool, 8" Dia.x 30" xFxLoose End D.I. 125#
23	1	6" D.I. Spool Fx Loose End (45" Long)	Spool, Ø6" D.I. Fx Loose End (45" Long) 125#
24	1	400FLAAL (APG/ Peterson Indust.)	Flange Adapter, 4" 150# Aluminum
25	1	440DCAL (APG/ Peterson Industrial)	Dust Cap Alumin 4"
26	6	Ø8" Gasket	Gasket, Ring Flange Ø8" 150# 3/32" Thk.
27	2	Ø4" Gasket	Gasket, Ring Flange Ø4" 150# 1/8" Thk.
28	3	Type 433.50 0-60psi (Wika)	Pressure Gauge w/ Diaphragm Liquid Filled 4" 0-60psi
29	4	FNW220-0.50 (FNW)	Ball Valve, 1/2" NPT 316 SS
30	4	Bushing 3/4"x1/2" 316 SS	Bushing 0.75 x 0.50 316 SS
31	7	1/2" Close Nipple 316 SS	Nipple, 1/2" Close Sched. 40 316SS
32	3	LS-475-C-10 (Proctor Sales)	Link Seal, 6" D.I. pipe
33	1	Model# CS-12-10/ Size: LS-400-C-9 (Proctor Sales)	Link Seal, 8" D.I. pipe
34	6	S89 Ø6" 150# Flange (Standon/Ferguson)	Adjustable Pipe Support, Ø6" 150# Flg. Galv./S.S.
35	1	M01694 (PumpTech, Inc.)	Lid Plate, 0.50"xØ128" Al. Diamond Plate
36	1	AHD 39x56 Al. (USF Fabrication)	Hatch, 39x56 Al. H-20,w/Springs & Rec. Padlock
46	4	M-00508 (PumpTech, Inc.)	Anchor bar, 0.75x4x10 SS
47	1	Pipe, Ø4" (24" Long) NPTxBlank Sched. 40 SS	Pipe, Ø4" x 24" Long NPTxBlank Sched. 40 SS



SECTION A-A



BACK VIEW

REFERENCE DRAWING # M01689

UNLESS OTHERWISE SPECIFIED:
DIMENSIONS ARE IN INCHES
TOLERANCES:
FRACTIONAL: +/- 1/4
ANGULAR: +/- 1
ONE PLACE DECIMAL +/- 0.2
TWO PLACE DECIMAL +/- 0.07

INTERPRET GEOMETRIC TOLERANCING PER:
MATERIAL
FINISH
DO NOT SCALE DRAWING

	NAME	INITIALS	DATE
DRAWN	MAO		05/06/10
CHECKED	A.B.		
SALES APPR.	J.D.		
PURCH.APPR.	B.R.		
MFG APPR.	S.B.		

PumpTech Inc.

S.O.#83251 MTS.#M65430

P.T.M.

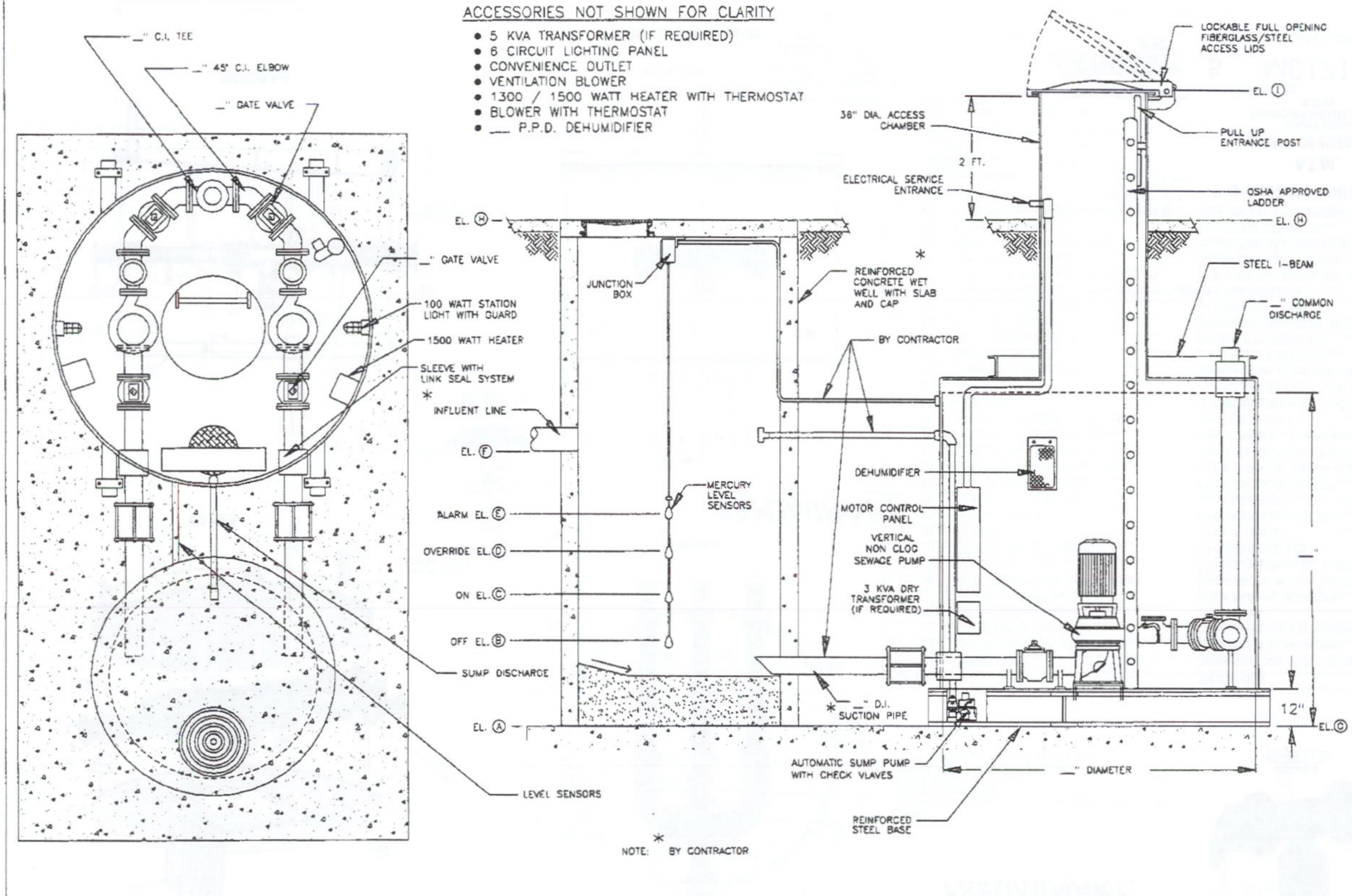
CITY OF FILER P.S.

VALVE VAULT
HYDRONIX MODEL 400 P.S.
B.O.M.

SIZE B DWG. NO. M01710 REV

SCALE: 1:40 WEIGHT: 9,000 SHEET 1 OF 1

PRINT DATE: 8/23/2010 9:45



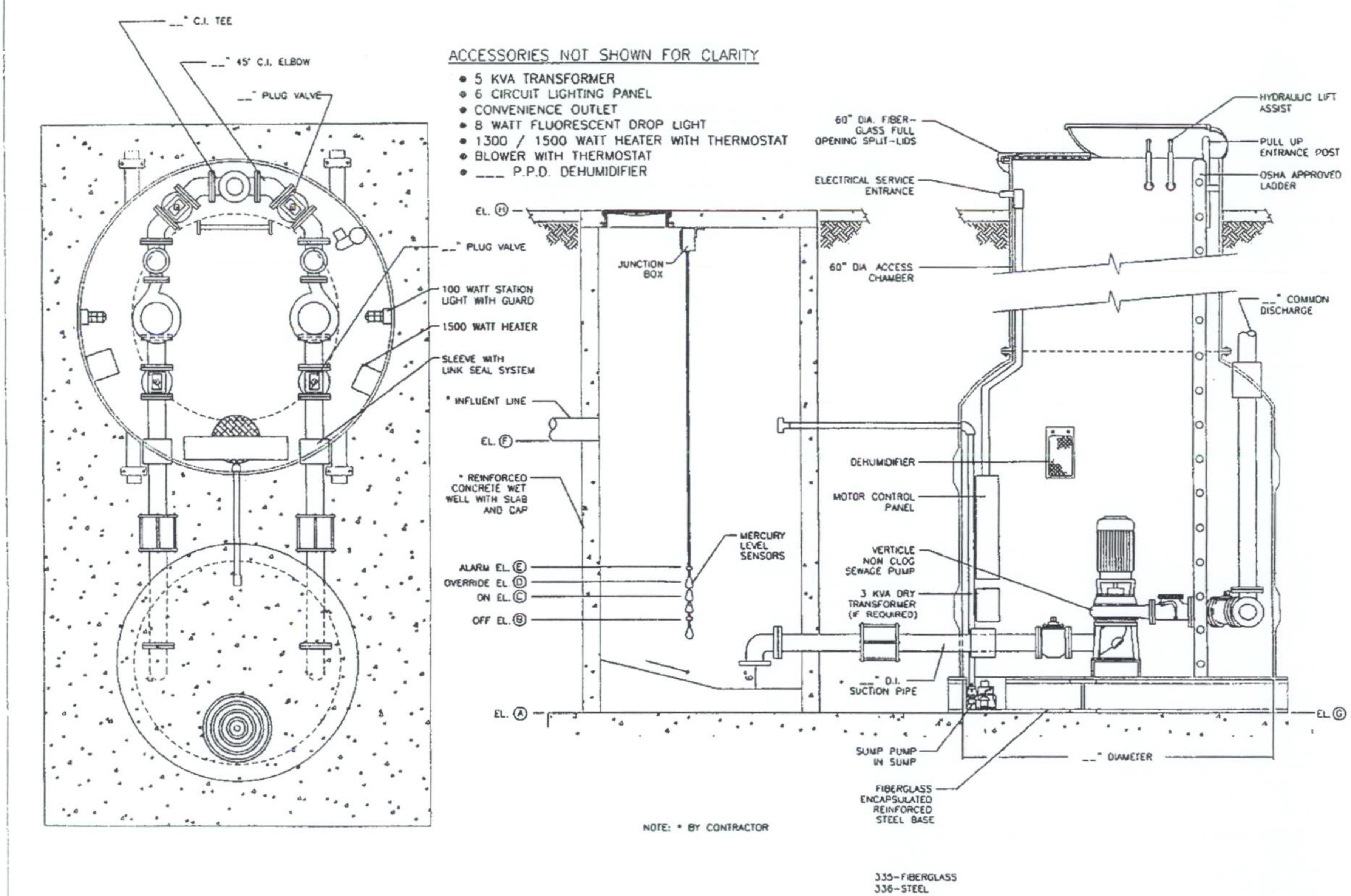
SCALES & DIMENSIONS ARE FOR REFERENCE ONLY, DRAWINGS MUST BE CERTIFIED CORRECT FOR CONSTRUCTION PURPOSES

G.P.M.	T.D.H.	H.P.	R.P.M.	PHASE	VOLTS	ELEVATIONS A	B	C	D	E	F	G	H	I

HYDRONIX

320 SERIES (STEEL SHELL)
FLOODED SUCTION, SEWAGE PUMPING STATION
BURIED, REDUCED OPENING, SINGLE LID

MODEL # 326 | DWG # 320-CAT-2 REV. 1

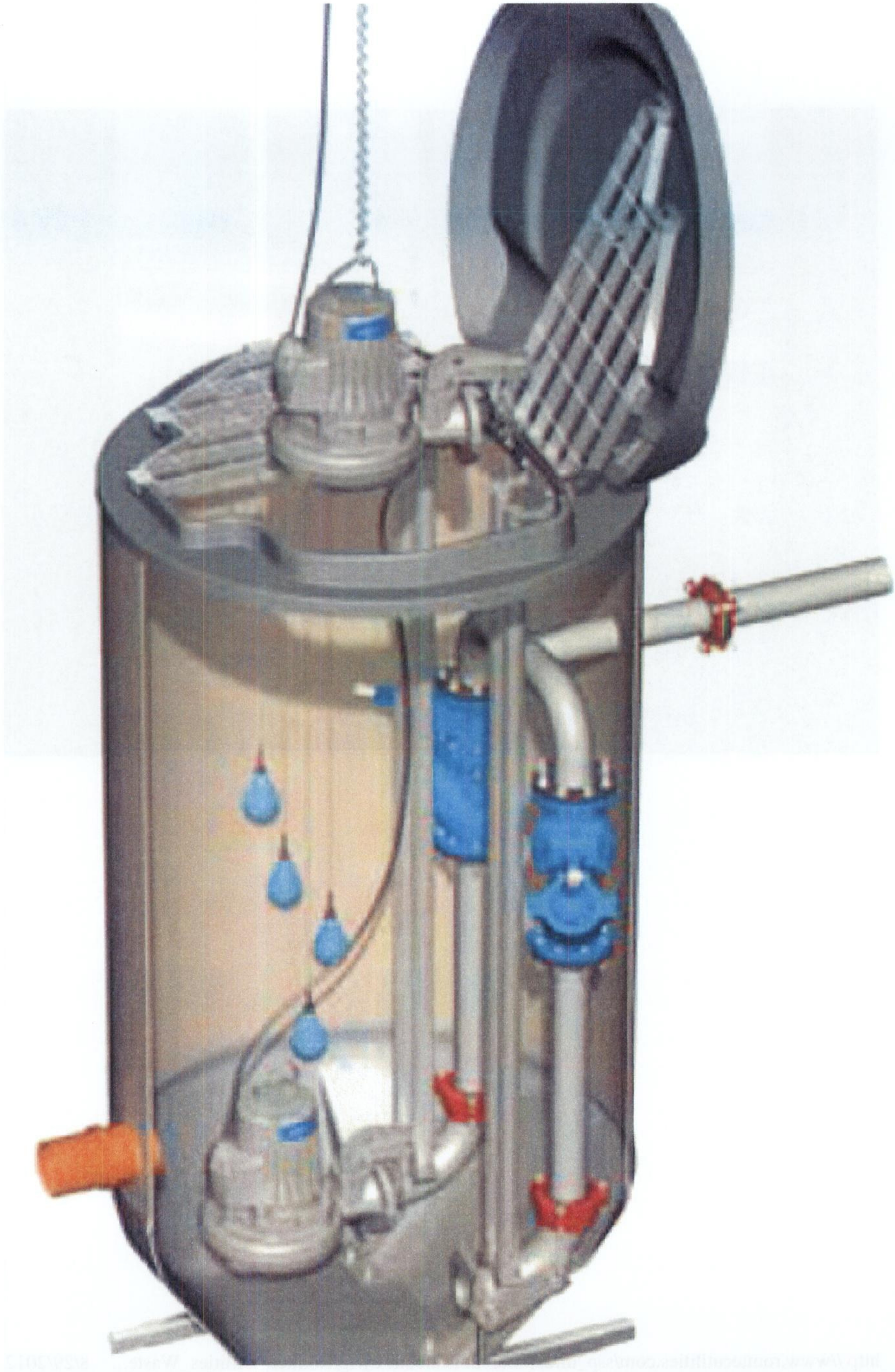


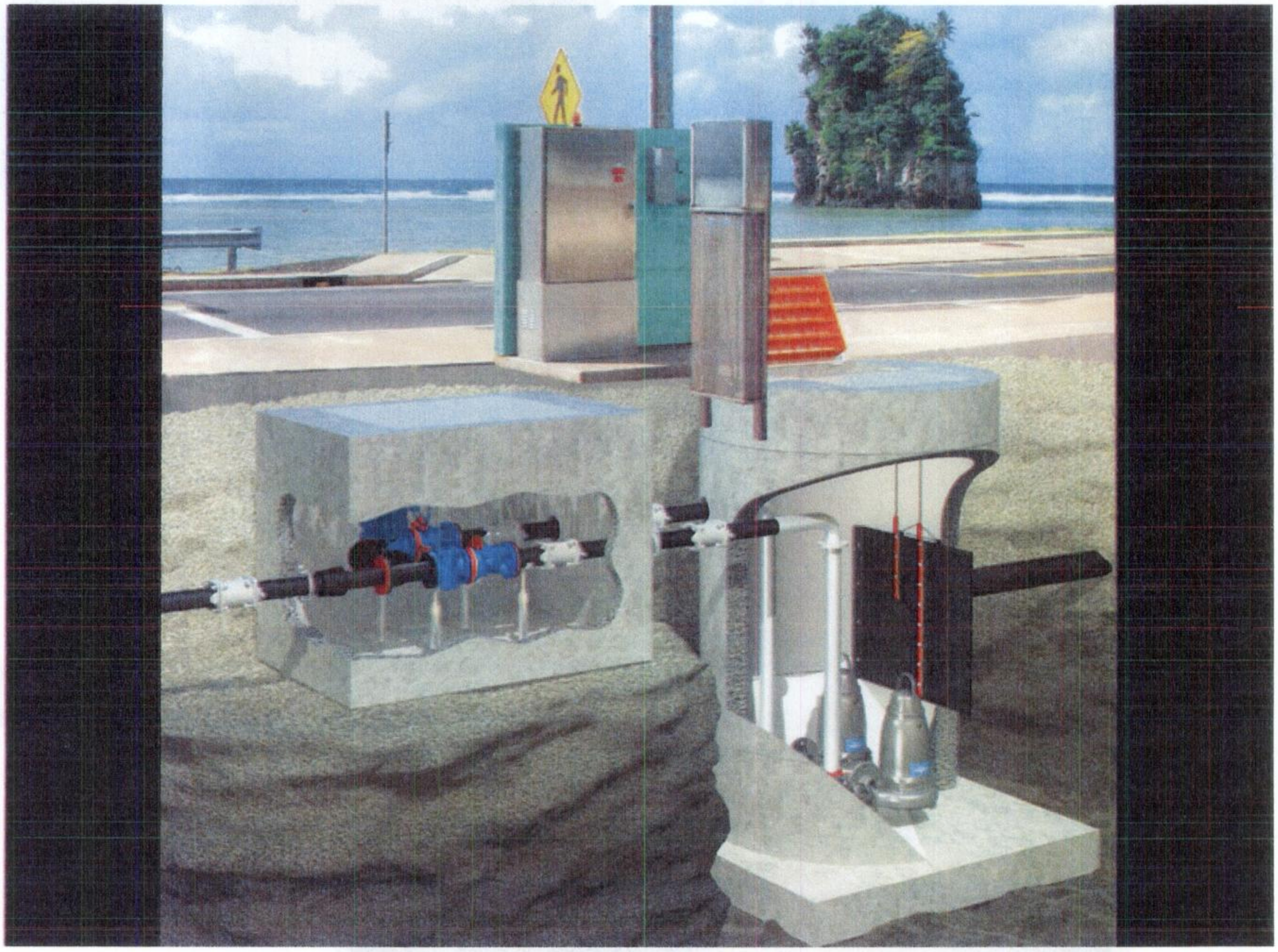
SCALES & DIMENSIONS ARE FOR REFERENCE ONLY. DRAWINGS MUST BE CERTIFIED CORRECT FOR CONSTRUCTION PURPOSES

G.P.M.	T.D.H.	H.P.	R.P.M.	PHASE	VOLTS	ELEVATIONS A	B	C	D	E	F	G	H

330 SERIES
FLOODED SUCTION, SEWAGE PUMPING STATION
BURIED, REDUCED OPENING, SPLIT LIDS

MODEL # 335/336 DWG # 330-CAT-R





Appendix F

Public Hearing Information