April 2009

Wastewater Treatment Facility Plan

WORKING TOGETHER TO ENHANCE THE CITY OF

HARRISBURG, SOUTH DAKOTA

Presented by:

Howard R. Green Company

Prepared for:

The City of Harrisburg, SD

CERTIFICATION

WASTEWATER TREATMENT FACILITY PLAN

CITY OF HARRISBURG, SOUTH DAKOTA

APRIL 2009

REGISTERED PROFESSIONAL ENGINEER REG. NO. 8326 TANYA L. MILLER SOUTH DAKOTA	I hereby certify that this engineering document was prepared by me or under my direct personal supervision and that I am a duly registered Professional Engineer under the laws of the State of South Dakota. Date: 4/15/09 **TANYA L. MILLER, P.E.** License No. **8326** My renewal date is **June 30, 2010** Pages or sheets covered by this seal: **Entire report.**

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I. EXECUTIVE SUMMARY

A. PURPOSE

In 2007, the City of Harrisburg realized that their existing sanitary sewer evaporation ponds would reach capacity much earlier than anticipated due to recent population growth. As a result, the hired Howard R. Green Company (HR Green) began a wastewater facility planning process to evaluate when the lagoons would reach capacity and propose solutions. Initial findings suggested that the lagoons would be full in 2011. A draft Facility Plan was prepared in 2007, which evaluated several options for future wastewater treatment. The report recommended a phased construction process with the first phase meeting the needs from 2011 to 2021, and a second phase meeting needs through 2031. Construction costs for the first phase were projected at $16.4 to $37.4 million with $1.3 to $14.6 million for the second phase based on 2007 dollars.

Harrisburg faces many challenges in trying to fund a project of this magnitude. Even with significant rate increases, financing the project proved to be a challenge. The City met with the South Eastern Council of Governments (SECOG), the South Dakota Department of Environment and Natural Resources (SD DENR), and the City of Sioux Falls to discuss and brainstorm all funding options.

Since 2007, the City has researched funding options and continued to monitor the lagoons. The most recent indications are that the lagoons are reaching capacity and an emergency discharge will be required in spring 2009. In the past few months, other treatment alternatives that facilitate ammonia removal in cold climates with lower capital construction costs have also been considered.

Because Harrisburg's existing evaporation ponds are projected to reach capacity in 2009, they must be modified or replaced. Failure to expand the ponds or provide another means of wastewater treatment would result in unauthorized discharge, potential environmental damage to the surrounding area, and State and Federal fines. Furthermore, economic growth and development will be forced to cease in Harrisburg and the area economy will be negatively impacted if the wastewater treatment capacity cannot be increased for the community. This Facility Plan provides Harrisburg with a planning guide for the safe treatment of the City's wastewater for the next 20-years.

B. EXISTING FACILITIES

A three cell, 63 acre, evaporation pond currently provides wastewater treatment for the City of Harrisburg. No wastewater is discharged from the ponds. At the time the ponds were constructed, they were projected to have capacity until 2017. Due to the recent population increase the City has experienced, the ponds are projected to be full in 2009.

C. PROJECTED FLOWS AND LOADINGS

The City of Harrisburg's population has more than tripled in the past six years, and the rapid growth has placed a strain on the City's existing wastewater treatment infrastructure. Conventional population projection methods could not be used because of the large, recent increases. Therefore, the future population projection was made by evaluating the recent trends in building permits for new homes in Harrisburg.

This Facility Plan was first drafted in 2007. Initially, it was thought that improvements would not be needed until 2011. Therefore, population projections were completed for a 20-year planning period between 2011 and 2031. A two-phased approach was used for planning purposes. Because of this, the projected flows and loadings in the report frequently refer to years 2021 and 2031.

In 2009, it was determined that the lagoons will reach capacity by spring, and construction of a new wastewater treatment system needs to begin later this year. As a result, a new population projection was prepared taking into consideration the recent downturn in building permits. The population projection for both scenarios is shown in Table IV-12.

Projected flows were calculated for average dry weather (ADW), average wet weather (AWW), maximum wet weather (MWW), and peak hourly wet weather (PHWW) flows. Projected influent flows were determined assuming 75 gallons per capita per day (gpcd) for average day dry weather (ADW) and 100 gpcd for average day wet weather (AWW). MWW flows were calculated by multiplying the AWW by an assumed peaking factor of two (2). PHWW flows were calculated by multiplying the AWW by a population based peaking factor as outlined in Ten States Standards. The projected influent wastewater flows are summarized in Table I-1 for the 2007 and 2009 Facility Plan designs.

Table I-1: Projected Influent Wastewater Flows

Condition	2007 Design Year		2009 Design Year	
	2021	2031	2019	2029
ADW, mgd	1.03	1.85	0.84	1.52
AWW, mgd	1.37	2.46	1.18	2.02
MWW, mgd	2.75	4.92	2.24	4.04
PHWW, mgd	3.87	6.31	3.25	5.35

Influent loading conditions for biological oxygen demand (BOD_5), total suspended solids (TSS), ammonia, and Total Kjeldahl Nitrogen (TKN) are presented in Table I-2. They were calculated using projected domestic populations and typical per capita loading rates of 0.20 pounds per day (ppd) for BOD_5, 0.22 ppd for TSS, 0.025 ppd for ammonia, and 0.038 ppd for TKN. Maximum values were calculated using the ratio of maximum concentration to average concentration from wastewater sampling shown in Table IV-1 in this report.

Table I-2: Projected Influent Wastewater Loadings

Condition	**2007 Design Year**		**2009 Design Year**	
	2021	**2031**	**2019**	**2029**
BOD_5 Average, ppd	2749	4,922	2,238	4,381
BOD_5 Max, ppd	3,646	6,530	2,969	5,811
TSS Average, ppd	3,024	5,415	2,462	4,819
TSS Max, ppd	3,870	6,931	3,152	6,168
NH_3-N Average, ppd	337	604	275	537
NH_3-N Max, ppd	413	698	343	621
TKN Average, ppd	550	929	456	827
TKN Max, ppd	636	1,074	527	956

D. PROJECT OPTIONS

The Facility Plan evaluated the following five (5) options for Harrisburg's future wastewater treatment. For the LEMNA alternative in Option 3, and Options 4 and 5, a phased approach was used with the infrastructure needed for the first 10 years of operation constructed first. Additional equipment and structures would be added for the second 10 years to provide treatment capacity through either year 2029 or 2031. The opinion of probable construction costs, and operation and maintenance (O&M) cost for each phase, and overall probable project present worth costs are provided in Table I-3. The costs assume a 20% contingency due to the preliminary nature of the design. Engineering, construction administration, and legal fees are expected to be 20% of the overall project cost.

1. Option 1: No Action

If the City does nothing, they will be forced to halt economic development to eliminate additional wastewater sources. In addition, the evaporation ponds would eventually fill and overflow resulting in environmental damage and fines. The City does not consider this an option.

2. Option 2: Expansion of the City's Existing Evaporation Ponds

Approximately 451 acres of additional land would be required to expand the City's evaporation ponds to meet future needs. The large land area required, siting constraints due to buffering requirements, and potential for odors make this an undesirable option, and it was not considered.

3. Option 3: Aerated Lagoons

The City of Harrisburg could convert their existing total containment ponds into aerated lagoons and discharge the effluent to a nearby waterway. It is expected that the 30-day average discharge limit for ammonia would be 1.0 mg/l during summer months and 2.0 mg/l during the winter months. Typically, aerated lagoons in cold climates, such as Harrisburg's, are not capable of this level of ammonia removal. Lagoon covers and other treatment options can be used to meet this low, year-round ammonia discharge limit. Recent technology developed in the upper Midwest and tested in Canada indicates that nitrogen removal can be accomplished in Submerged Attached Growth Reactors (SAGR) in cold climates. The two options considered for conversion of the existing ponds are:

- Lemna Technologies LEMTEC Process
- Nelson Environmental OPTAER Process with SAGR

Conversion of the lagoons to aerated ponds would create a complete mix zone, partial mix zone, quiescent/settling zone, and a submerged attached growth reactor. Fine-bubble diffused aeration is provided in the complete mix and partial mix zones and medium/coarse bubble diffused aeration is provided in the SAGR.

Discharge would be to a nearby receiving waterway. Harrisburg's nearest waterway, Ninemile Creek, discharges into Lake Alvin. Lake Alvin is a protected watershed and a typical treated wastewater discharge is not allowed into its tributaries within ten miles of the Lake. Since Harrisburg is over five miles from the lake inlet (6.3 miles approximately), a discharge permit that includes nitrogen and phosphorus removal would be considered for discharge to Ninemile Creek. Because of the phosphorus removal requirement, chemical addition and sand filtration will also be required. The details of the discharge location and permit would be coordinated with the South Dakota Department of Environment and Natural Resources (SD DENR) during design.

4. Option 4: New Mechanical Wastewater Treatment Plant (WWTP)

Construction of a new mechanical WWTP evaluates the gravity interceptor, force main, lift station, mechanical WWTP, and outfall required to convey wastewater from Harrisburg to the Big Sioux River. The WWTP is proposed near the Big Sioux River to maximize the future area the WWTP would eventually serve via a gravity collection system. Large diameter gravity sanitary sewer piping is proposed from the current total containment ponds to a lift station. Force main is proposed from the lift station to the WWTP.

Three treatment alternatives were evaluated for Option 4, the new mechanical WWTP, including:

- Sequencing Batch Reactor (SBR)
- Conventional Activated-Sludge
- Membrane Bioreactor (MBR)

5. Option 5: Regionalization

Several options for regionalization were considered including:

- Pumping wastewater to the City of Sioux Falls for treatment
- Building a larger WWTP than needed and selling excess capacity to the City of Sioux Falls or others
- Sioux Falls relocating the proposed WWTP further south of the City to accommodate Harrisburg
- Purchasing a portion of the proposed Sioux Falls WWTP located on the south side of the City
- Construction of a regional WWTP with the City of Tea

Of these options, the only one that was considered to be viable was pumping wastewater to the City of Sioux Falls for treatment. Harrisburg does not have the available capital or debt capacity to front the money needed to build a larger WWTP than needed and sell the excess capacity to the City of Sioux Falls. Sioux Falls has indicated that they are not interested in relocating their proposed WWTP further south due to the recent construction of Sioux Falls Lift Station #240. Sioux Falls has also indicated that they would prefer not to sell a portion of their new WWTP to Harrisburg. Finally, Tea recently upgraded their existing lagoons to aerated lagoons and can discharge to Ninemile Creek, since they are more than ten (10) miles from Lake Alvin. They have indicated that they have available capacity for several years and are not interested in regionalization at this time.

Harrisburg could pump their wastewater to the City of Sioux Falls for treatment. This would require Harrisburg to construct a small section of gravity sewer interceptor, a lift station, and approximately 10.5 miles of 16-inch force main.

Initially, the wastewater would be pumped to Sioux Falls' Lift Station #240 located near 57th Street and the Big Sioux River. This lift station would convey wastewater to Sioux Falls' current WWTP on the north side of the City.

The City of Sioux Falls plans to construct a new MBR WWTP, near Lift Station #240. At that time, flows from Harrisburg would be directed to the head of the WWTP.

The Sioux Falls MBR plant cannot tolerate rapid changes to influent flows. As a result, the existing evaporation ponds are proposed to be reused as an equalization basin to provide storage. The equalization basin will lessen the peak flows sent to Sioux Falls for treatment, reduce the needed pumping capacity and the overall size of the lift station, and offers Sioux Falls operational flexibility. The basins will likely need to be aerated to reduce odors; however, it is expected that some odor conditions would develop in the basins and affect residents in the area even with aeration.

Table I-3: Summary of Opinion of Probable Construction and O&M Costs

Treatment Process	Alternatives						
	Convert Existing Ponds to Aerated Lagoons with OPTAER Process and SAGR	Convert Existing Ponds to Aerated Lagoons with LEMNA Process	New Harrisburg WWTP			Pump to Sioux Falls: Can-Type LS and Use Existing Ponds for Equalization Storage	Pump to Sioux Falls: Can-Type LS and Construct New Lagoons Outside City Limits
			SBR	Conventional AS	MBR		
Phase One Total Construction Costs	$ 14,175,800	$ 12,181,800	$ 27,233,100	$ 29,451,800	$ 34,697,200	$ 9,853,000	$ 15,317,600
Phase Two Total Construction Costs	$ -	$ 4,997,000	$ 10,148,000	$ 7,851,000	$ 14,651,000	$ 2,238,000	$ 2,094,000
Phase One Annual O&M Costs	$ 291,200	$ 295,700	$ 370,900	$ 406,500	$ 410,000	$ 406,100	$ 406,100
Phase Two Annual O&M Costs	$ 385,400	$ 462,600	$ 388,300	$ 472,300	$ 481,100	$ 681,300	$ 681,300
Present Worth Project Costs	$ 20,291,000	$ 23,302,000	$ 42,021,000	$ 43,400,000	$ 54,587,000	$ 23,085,000	$ 28,611,000

E. SUMMARY AND RECOMMENDATIONS

After extensive review of the capital construction costs, long-term O&M costs, and advantages and disadvantages of each option, we recommend the City of Harrisburg proceed with construction of the lift station and force main to pump their wastewater to Sioux Falls for treatment. This option has the lowest capital cost and will be the easiest for the City to fund. The City of Harrisburg will pay Sioux Falls to treat the wastewater, and the 20-year present worth analysis indicates this option may be slightly more expensive than modifying their existing lagoons. However, this option allows the City to slowly increase rates to fund the rising costs for treatment as the flows increase annually. In addition, the City will need additional staff to operate and maintain a treatment facility. The City will not have to comply with a discharge permit requiring ammonia and phosphorus removal. Lagoons are typically not used to meet the low levels of ammonia and phosphorus proposed for discharge to Ninemile Creek and there was some concern with permit compliance. Finally, this option offers the fastest schedule and will allow wastewater to be discharged and pumped to Sioux Falls for treatment as early as the first half of 2010.

II. INTRODUCTION

A. BACKGROUND

The City of Harrisburg (City) is located approximately two and one-half miles south of Sioux Falls in Lincoln County in eastern South Dakota. The City's evaporation ponds treat wastewater from domestic, commercial, and industrial sources located within the City's corporate limits. Domestic wastewater accounts for the largest portion of the total wastewater flows and loads. Currently, no major industrial wastewater flows or loadings are received at the ponds. The current industries are considered "dry" industries, with their waste streams consisting mainly of sanitary flows from facility restrooms.

The City's first wastewater treatment system consisted of stabilization ponds with discharge to a ditch leading to Ninemile Creek. The stabilization ponds were constructed south of the City in 1974. They were abandoned in 1999, after the City's current evaporation ponds were constructed just to the east. The evaporation ponds consist of a series of three total containment ponds. Harrisburg is restricted from discharging treated wastewater to Ninemile Creek because is it a protected waterway that flows into the protected Lake Alvin approximately 6.3 miles downstream. The current evaporation ponds have a design average daily flow of 0.133 million gallons per day (mgd), maximum daily flow of 0.331 mgd, and an average BOD_5 loading capacity of 275 ppd. Harrisburg's existing evaporation ponds were designed to have capacity until 2017, but because of recent population increases they are projected to reach capacity in 2009.

Residential housing encompasses the current pond site to the north and east, and the Burlington Northern Santa Fe Railroad runs along its west side. This will limit future expansion due to South Dakota Department of Environment and Natural Resources (SD DENR) sighting requirements for wastewater treatment facilities and expansions. Regulatory requirements, aesthetic concerns, and available land for expansion will pose issues for the City in the future at this site.

B. PURPOSE AND SCOPE

The purpose of this Facility Plan is to provide the City with a guide to planning and design of the expansion and/or replacement of their existing ponds. The recommended alternative will meet proposed effluent limits, and current solids handling and disposal regulations.

The Facility Plan analyzes and develops opinions of probable cost for various treatment alternatives. Total capital costs, which include construction costs, engineering, administration and legal costs, have been developed for each alternative. The total present worth values prepared incorporate capital costs, and annual operation and maintenance (O&M) costs inflated over a 20-year period.

III. ENVIRONMENTAL REVIEW

A. ENVIRONMENTAL INFORMATION

As a part of the State Revolving Fund (SRF) loan application process for SD DENR, the following agencies were asked to comment on the proposed project: The South Dakota State Historical Preservation Office, the US Fish and Wildlife Service, the SD Department of Game, Fish, and Parks (GFP), the US Natural Resource Conservation Service (NRCS), and the US Army Corps of Engineers (USACE). Letters were sent to the various agencies with a short project description, and project location maps showing the relationship of the project to the City of Harrisburg. These maps are included in Appendix A.

B. HISTORICAL AND ARCHAEOLOGICAL SITES

The State Historical Society Archeological Research Center was sent requests to complete an archaeological and structural records search within a one-mile radius of the project location. Letters were sent February 27, 2009 and March 19, 2009, and copies of the letters are included in Appendix A. The State Historical Society Archeological Research Center does not have records of receiving these letters. An email requesting a records search was sent on April 15, 2009 and is included in Appendix A. The response letter and findings of the records search will be forwarded to the SD DENR as soon as it is received.

A Cultural Resources Effects Assessment Summary will be completed and sent to the SD DENR requesting a determination as soon as the records search is received from the State Historical Society Archeological Research Center.

C. FLOODPLAINS AND WETLANDS

The USACE was sent clearance letters for the project on March 13, 2009. Comments were received in a letter dated April 10, 2009, stating:

> “Your plans should be coordinated with the U.S. Environmental Protection Agency, which is currently involved in a program to protect groundwater resources. In addition, the South Dakota State Historic Preservation Office should be contacted for information and recommendations on potential cultural resources in the project area.
>
> We are not able to provide flood plain impact comments at this time. The project does not appear to be within Corps owned or operated land. To determine if the proposed project may impact areas designated as floodway please consult the following flood plain management offices.

NFIP Coordinator:
South Dakota, Division of Emergency Management
Nicole Prince
118 W. Capitol Ave.
Pierre. SD 57501-5070
Nicole.prince@state.sd.us
T-605-773-3238
F-605-773-3580

FEMA:
Ryan Pietramali
Federal Emergency Management Agency
Region VIII, Denver Federal Center
Building 710, P.O. 25267
Denver, CO 80225-0267
Ryan.pietramali@dhs.gov
T-303-235-4836
F-303-235-4849"

A copy of the letter is included in Appendix A.

D. AGRICULTURAL LANDS

The NRCS offered the following comments in a review letter dated March 30, 2009: "The project will have no effect on prime or important farmland." The complete letter has been included in Appendix A.

E. WILD AND SCENIC RIVERS

The South Dakota GFP has reviewed the project and stated the following in a letter signed and dated on March 18, 2009. "Due to the previously disturbed nature of these areas, the project described will have no significant impact on fish and wildlife resources. However, if the project design changes or if new information becomes available, please submit changes for review." The complete letter has been included in Appendix A.

F. FISH AND WILDLIFE PROTECTION

The US Fish and Wildlife Service indicated "no objection" in their response dated March 16, 2009 attached in Appendix A.

G. DIRECT AND INDIRECT IMPACTS

Indirect impacts will occur throughout the City of Harrisburg as a result of the proposed improvements. The project will result in expanded capacity of the system and will allow for increased residential, commercial, and industrial development in Harrisburg. This development will lead to additional underground utilities, roads, schools, and parks. Infrastructure for increased City services, as well as police, fire, and medical services will also be needed.

H. MITIGATING ADVERSE IMPACTS

Though adverse impacts are not expected as a part of this project, best management practices will be required to minimize impacts. The proper authorities will be contacted if issues arise.

IV. EXISTING AND FUTURE CONDITIONS

A. PROJECT NEED AND PLANNING AREA IDENTIFICATION

The existing zoning map is shown as Exhibit B-1 in Appendix B. The City of Harrisburg serves an area of approximately 1,500 acres.

Residential areas vary from low/medium density to high density with the majority of the existing residential development being low/medium density. Single and multi-family residential land uses comprise the greatest amount of land area in Harrisburg. Future residential development is anticipated to occur within areas currently annexed into the City to the south of the Industrial Park, on the south side of the City. These undeveloped areas are currently labeled Natural Resource Conservation (NRC) Districts on the zoning map in Exhibit B-1. The abandoned and existing wastewater ponds are also located in land zoned NRC. The City map is provided as Exhibit B-2 in Appendix B showing the location of the current and abandoned ponds.

Commercial development is located in the areas labeled Central Business Districts and General Business Districts in Exhibit B-1. Most of these areas are not fully developed at this time.

In discussions with the City, no significant industrial development is anticipated. The City of Harrisburg does have an Industrial Park on the north side of town; however, the current and anticipated businesses are considered dry industrial companies.

The 20-year project planning area extends beyond the current City limits of Harrisburg. The City plans to obtain this additional land through annexation. The Future Land Use Map provided at the end of Appendix B shows the 2025 planning area boundary and anticipated land use.

It should also be noted that a new high school is under construction one-half mile to the west of the City and projected to open in the fall of 2009. Additional residential development is projected to occur around this area.

B. CURRENT WASTEWATER CHARACTERISTICS

1. Hydraulic Load

The City of Harrisburg operates total containment lagoons and does not discharge. Flow into the existing evaporation ponds is measured at the influent Parshall flume with an ultrasonic level transducer. Totalized flow can be read at any time, and Harrisburg's supervisory control and data acquisition (SCADA) system has recently begun to store the data. Flow information is available for a portion of 2008, and to date for 2009. The data is provided in Appendix D and summarized in Table IV-1. Not enough flow data has been collected yet to use the information for future projections. However, the data can provide a good check for the assumptions used in projecting flow for 2009.

Table IV-1: Influent Flow Data for Harrisburg's Evaporation Ponds

	2008	2009
Total Annual Flow (gal)*	92,811,000	21,454,000
Average Day Flow (gpd)	369,765	346,032
Max Day Flow (gpd)	1,285,000	815,000
Ratio of Ave Day Flow to Max Day	3.48	2.36
Average Day Dry Weather Flow (gpd)**	315,563	
Max Day Dry Weather Flow (gpd)	680,000	
Ratio of Ave Day Dry to Max Day Dry Flow	2.2	
Average Day Wet Weather Flow (gpd)***	711,077	
Maximum Day Wet Weather Flow (gpd)	1,285,000	
Ratio of Ave Day Wet to Max Day Wet Weather Flow (gpd)	1.8	

*Flow data was available for 251 days in 2008 and from 01/01/09 to 03/03/09 for 2009.
**ADW flow for 2008 was determined with data from 09/11/08 through 12/31/08.
***AWW flow for 2008 was determined with data from 03/08/08 through 04/06/08. During that 30-day period, 26 days had data available.

2. Organic Load

Limited historical influent wastewater quality data was available for review. Thus, sampling and analysis was performed in 2007 to obtain data on the influent wastewater characteristics. City personnel and HR Green staff completed the sampling. Two independent certified laboratories, South Dakota State University (SDSU) and the South Dakota State Health Laboratory, performed the analysis.

Wastewater sampling was collected at the influent manhole with an Isco Composite Sampler. The sampler was programmed to collect a 250-milliliter (ml) sample every hour for 24 hours. Each 250-ml sample was collected in an individual one liter sample bottle, and all 24 samples were combined and mixed in a 20-liter plastic carboy. Representative samples were then placed in sample bottles provided by the certified laboratories. After the ammonia sample was taken, it was preserved with sulfuric acid. Samples were shipped next day air and packaged with ice to preserve them. The results of the sampling are presented in Tables IV-2 and IV-3. The City may want to consider periodic sampling and testing of the wastewater influent to verify the assumptions made from this limited data set.

The City tested the quality of the water in the third cell in the summer of 2008. Table IV-4 summarizes the results of this testing.

Table IV-2: Influent Water Quality Sampling Results Tested by South Dakota State University

Sampler Start Date	**Sampler End Date**	**BOD_5** *(mg/L)*	**TSS** *(mg/L)*	**Ammonia - N** *(mg/L)*
4/24/2007	4/25/2007	172	152	19.6
4/30/2007	5/1/2007	237	160	29.4
5/1/2007	5/2/2007	223	154	30.2
5/2/2007	5/3/2007	278	252	34.0
5/14/2007	5/15/2007	186	173	31.7
5/15/2007	5/16/2007	157	244	29.6
5/16/2007	5/17/2007	214	244	31.4
Maximum		278	252	34.0
Average		210	197	29.4
Standard Deviation		42	47	4.6
Maximum to Average Ratio		1.33	1.28	1.16

Table IV-3: Influent Water Quality Sampling Results Tested by the South Dakota State Health Lab (Sampled 4/25/07-4/26/07)

PARAMETER	**VALUE**
BOD_5, *mg/l*	185
CBOD, mg/l	184
COD, *mg/l*	258
Total Solids, *mg/l*	1356
TDS, *mg/l*	1086
TSS, *mg/l*	212
VTSS, *mg/l*	168
Ammonia - N, *mg/l*	24.9
TKN, *mg/l*	38
Alkalinity - M, *mg/l*	313
Alkalinity - P, *mg/l*	0
Magnesium, *mg/l*	56.5
Potassium, *mg/l*	9.1
Sodium, *mg/l*	179
Phosphorous, *mg/l*	5.03
Nitrate, *mg/l*	1.0
Chloride, *mg/l*	280
Iron, *mg/l*	0.27
Sulfate, *mg/l*	271

Table IV-4: Water Quality in Cell No. 3 Tested by the South Dakota State Health Lab (Sampled 6/16/08)

PARAMETER	VALUE
BOD_5, *mg/l*	<3
TSS, *mg/l*	<3
TDS, *mg/l*	815
Ammonia - N, *mg/l*	1.33
Total Coliform, per 100ml	1700
Fecal Coliform, per 100ml	490
Phosphorus, *mg/l*	1.42

C. EVALUATION OF TREATMENT SYSTEMS

1. Existing Wastewater Treatment System

The existing Wastewater Treatment System is located on the south side of town, in the W½ of the SE¼ of Section 1, Township 99 North, Range 50 West. The City's previous abandoned lagoon system is located adjacent to the existing treatment facility in the NE¼ of the SW¼ of Section 1. Both sites are shown on the City map in Figure B-2 in Appendix B.

The area of the existing wastewater ponds is approximately 63 acres. The aerial photo in Figure C-1 in Appendix C also shows the residential areas directly north and east of the site. The site is bordered to the west by an existing railroad right-of-way, and on the south by undeveloped land.

Influent flow is measured in a 72-inch precast manhole with a prefabricated Parshall flume. An ultrasonic level transducer is located directly upstream of the Parshall flume to measure the water depth. A Miltronics Multiranger Plus converts the signal from the level transducer to a flow rate. The manhole and Parshall flume were constructed at the same time as the existing containment ponds in 1999, and are in very good condition.

Harrisburg's treatment facility consists of three (3) total containment lagoons in series. The capacities of each cell are listed in Table IV-5 and were obtained from the original construction plans.

Table IV-5: Total Containment Lagoon Capacities

Parameter	**Cell No. 1**	**Cell No. 2**	**Cell No. 3**
Function	Primary	Secondary	Tertiary
Top Water Surface Area, acres	10.2	10.2	19.6
Middle Water Surface Area, acres	9.7	9.6	18.5
Bottom Water Surface Area, acres	9.2	9.0	17.5
Water Depth, ft	5.0	6.0	8.0
Total Volume (including 3-foot storage area), MG	15.8	18.7	48.4

The City of Harrisburg operates their containment lagoons in series to obtain the most efficient and highest degree of treatment. The current piping configuration does not allow parallel treatment, and limits operational flexibility should maintenance need to be performed. The treatment operation is summarized as:

a. The wastewater flows by gravity to the influent manhole near the northwest corner of Primary Cell No. 1 where it is metered. It then flows into Primary Cell No. 1. When Cell No. 1 reaches a set elevation, the City opens the valve between the primary and secondary cells. Flow enters the second cell and once the water levels have equalized, the valve between the primary and secondary cells is closed.

b. The City repeats the same process to transfer volumes between Cell No. 2 and Cell No. 3, opening and closing the valve between the two cells.

c. The process of transferring volume between the cells typically occurs at 0.5-foot increments until water levels approach the high water level (HWL). Once the cells approach their HWL, transfers occur at more frequent intervals. Once all the cells have reached their HWL, the capacity of the treatment facility has been reached.

Since the City operates total containment lagoons, authorized discharge is not allowed. The only time discharge would be conducted is during an emergency situation.

2. Existing Wastewater Treatment System Capacity

The existing influent piping consists of 12-inch PVC sewer pipe laid at a 0.22% slope. The hydraulic capacity of the influent pipe is 1.08 mgd.

The design capacity of the total containment lagoons were obtained from the “Operations and Maintenance Manual for Wastewater Treatment Facilities” compiled by Stockwell Engineers, Inc., in 2000, and from original construction plans. The hydraulic and organic loading design capacities for the lagoon system are listed in Table IV-6.

Table IV-6: Total Containment Lagoon Design Capacities

Parameter	**Unit**	**Value**
Design Data		
Design Population	people	1718
Waste Flow	gal/cap/day	75
BOD_5	lb/cap/day	0.16
SS	lb/cap/day	0.20
Total BOD_5 Load	lb/acre/day	20
Storage Capacity at Design Flow	years	20
Design Flow		
Average Daily	gallon/day	132,750
Maximum Daily	gallon/day	331,200
BOD_5, Average Daily	lb/day	275
SS, Average Daily	lb/day	344
Primary Cell No. 1		
Water Surface Area	acres	10.21
Maximum Liquid Depth	feet	5.0
Minimum Liquid Depth	feet	2.0
Effective Storage Volume	MG	6.6
BOD_5 Loading	lb/acre/day	26.9
Minimum BOD_5 Removal	percent	50
BOD_5 Remaining	lb/day	138
Secondary Cell No. 2		
Water Surface Area	acres	10.18
Maximum Liquid Depth	feet	6.0
Minimum Liquid Depth	feet	2.0
Effective Storage Volume	MG	9.6
BOD_5 Loading	lb/acre/day	14
Minimum BOD_5 Removal	percent	50
BOD_5 Remaining	lb/day	69
Tertiary Cell No. 3		
Water Surface Area	acres	19.6
Maximum Liquid Depth	feet	8.0
Minimum Liquid Depth	feet	2.0
Effective Storage Volume	MG	36.8
BOD_5 Loading	lb/acre/day	3.5
Minimum BOD_5 Removal	percent	50
BOD_5 Remaining	lb/day	35
Overall Facility Design		
Effective Storage Volume	MG	53.0
Detention	years	20

3. Remaining Treatment System Capacity

Due to the recent population growth, the wastewater ponds are expected to reach capacity much sooner than their design life of 2017. The remaining life of the existing lagoon system was projected based on current population projections, projected influent flows, and the current water levels in the ponds. The following calculation for a total containment pond was utilized from the SD DENR Recommended Design Criteria Manual:

$$A = I / WL$$

Where:

A	=	Estimated surface area in acres
I	=	Volume of in-flow in acre-feet
WL	=	Net water loss (evaporation + seepage – precipitation) in feet

The City's annual rainfall is approximately 24.62-inches, annual evaporation is estimated at 39-inches, and the seepage rate is estimated at 1/16 inch per day (22.81-inches per year). This results in a net loss of 37.19-inches (3.10-feet) per year.

The surface area used in the equation was based on the surface area at the HWL location. Therefore, the total surface area used for calculation purposes was 1,741,300-square feet (sf), or 39.97-acres. Based on the equation, the annual volume of inflow that can be removed each year is estimated at 120.71 acre-feet, or 5,258,123-cubic feet or (39.3 million gallons).

The City of Harrisburg completed lagoon monitoring reports in 2000, 2001, and 2003. The lagoon monitoring reports are provided in Tables C-1, C-2 and C-3 in Appendix C for reference. The City has also taken periodic measurements of the depth of each pond cell. A summary of these depths is provided in Table IV-7. As of December 2008, the first and second cells were at capacity; however the third cell had approximately 0.75 feet of capacity remaining.

Table IV-7: Historical Water Levels in the Evaporation Ponds

	Depth of Cell (ft)		
Date	**Cell 1**	**Cell 2**	**Cell 3**
February 8, 2000	2	0	0
July 3, 2000	1.67	0.5	0
January 2, 2001	2	0	0
July 3, 2001	2.5	1.5	0
January 2, 2002	2.5	0.08	0
January 7, 2003	2.33	0	0
July 1, 2003	2	1	0
Summer 2004	2	2	0
Summer 2005	2	0.5	0
December 6, 2006	3.5	3.5	1.5
July 11, 2008	5.67	5.5	6.67
July 18, 2008	6	5.5	6.583
August 20, 2008	5.5	5.33	7
September 2, 2008	6	5.167	6.67
September 9, 2008	6.33	5	6.583
September 19, 2008	5.67	5.75	6.5
October 14, 2008	6	6.33	6.33
October 31, 2008	5.33	5.5	7.5
November 21, 2008	5.4167	6	7.33
December 1, 2008	5.67	6.083	7.25

The following assumptions were used to determine the remaining capacity of the existing evaporation ponds:

- 39.3 million gallons can be removed through evaporation and seepage annually as calculated above.
- The third cell of the ponds had 4,790,035 gallons of remaining capacity (0.75 feet * 19.6 acres) as of December 1, 2008
- Recent influent flow data shown in Table IV-1 indicates that average daily flow is probably around 370,000 gallons per day.

Using this information, the ponds should have enough capacity for 119 days, or until March 29, 2009. Considering that evaporation during the winter months will be low, the lagoons are essentially full. The City will be applying for an emergency discharge permit this spring, and completion of the improvements to address Harrisburg's future wastewater needs is vital.

4. Existing Collection System

Harrisburg's existing collection system consists of mostly 8-inch diameter sanitary sewer piping. The portion in the original part of town was installed in 1974. Much of the system consists of new PVC piping installed in new developments between 2001 and 2007.

Harrisburg's wastewater collection system was evaluated in 2005 in the Water and Wastewater Infrastructure Facilities Plan Report. This report assessed the sanitary sewer infrastructure needed within the City's 2025 growth area and determined the preliminary size and location of sanitary sewer interceptors. The report did not evaluate future wastewater treatment options. Exhibit C-2 in Appendix C is from the Water and Wastewater Infrastructure Facilities Plan Report completed in 2006 and shows these proposed interceptors.

The City plans to conduct a survey of residences in 2009 to determine if storm water/sump pump drainage is being directed into the City's sanitary sewer system.

D. EFFLUENT LIMITATIONS

Currently, the City of Harrisburg does not have effluent limitations, since they cannot discharge from their evaporation ponds. Preliminary correspondence with the SD DENR has indicated that a future wastewater treatment system discharging into the Big Sioux River south of Ninemile Creek may have effluent limitations similar to those presented in Table IV-8.

Table IV-8: Preliminary Limits for Discharge to the Big Sioux River

Parameter	**30-Day Average**	**Maximum**	**Minimum**
Ammonia	1.0 mg/l	-----	-----
Nitrate	50 mg/l	-----	-----
Biochemical Oxygen Demand (BOD_5)	10 mg/l	-----	-----
Total Suspended Solids (TSS)	10 mg/l	-----	-----
pH	-----	9.0	6.5
Dissolved Oxygen (DO)	-----	-----	5.0 mg/l
Fecal Coliforms	400**	370 counts / 100 ml*	-----

* Daily Maximum from March 15 to November 15
** 30–Day Geometric Mean

The City also inquired about discharge to Ninemile Creek. Ninemile Creek flows into Lake Alvin, which is considered a protected water way, but since the proposed discharge location would be more than 5 miles from the lake inlet, a permit that includes nutrient removal may be an option. Preliminary discussions with the SD DENR have indicated that a future wastewater treatment system discharging into Ninemile Creek may have effluent limitations similar to those presented in Table IV-9. However, it should be noted that these are only estimates, since water quality modeling would be needed to determine the actual allowable levels.

Table IV-9: Preliminary Limits for Discharge to Ninemile Creek

Parameter	**30-Day Average**	**7-Day Average**	**Daily Maximum**
Five Day Biochemical Oxygen Demand (BOD_5), mg/L	30	45	N/A
Total Suspended Solids (TSS), mg/L	30	45	N/A
Fecal Coliform, no./100 mL (May 1 – September 30)	1,000	N/A	2,000
Ammonia-Nitrogen (as N), mg/L March 1 – October 31 November 1 – February 29	 1.0 2.0	N/A	 1.75 3.5
Dissolved Oxygen, mg/L	N/A	N/A	5.0 (Daily Minimum)
Total Phosphorus, mg/L	N/A	N/A	0.1
Total Residual Chlorine, mg/L (Applicable only if effluent is chlorinated)	N/A	N/A	0.019
pH	The pH of the discharge shall not be less than 6.0 standard units nor greater than 9.0 standard units in any sample.		

* There shall be no Acute Whole Effluent Toxicity in the discharge, as measured by the WET test.
** Percentage Removal Requirements (TSS and BOD_5 Limit): In addition to the concentration limit on TSS and BOD_5 indicated above, the arithmetic mean of the TSS and BOD_5 concentration for effluent samples collected in a period of 30 consecutive days shall not exceed 15 percent of the arithmetic mean of the concentration for influent samples collected at approximately the same times during the same period (85 percent removal).

E. FUTURE CONDITIONS

1. Population and Land Use Projections

The City of Harrisburg has experienced an explosive increase in population over the past eight years. Until this recent surge, the population in Harrisburg had remained fairly steady. Table IV-10 lists the historical population based on census data for the past 40-years.

Table IV-10: Historical Census Data for Harrisburg, SD

Year	Population
1960	313
1970	338
1980	558
1990	727
2000	958

In 1999, a number of developers began to show an interest in Harrisburg, and since then, the population has grown dramatically. Considerable population increases during a short time period make it difficult to accurately project the population of a community. Census information cannot be used since it does not reflect the recent population increase; however, building permit information can be used for population projections.

From 2003 to 2006, the City of Harrisburg experienced an average population increase close to 30% per year based on the number of building permits issued and an assumption of 3.04 people per household for single-family homes and 2.5 people per household for apartments. Fewer building permits have been requested during the past two years, but the City continues to see growth in the current economy.

Table IV-11 shows the number of annual building permits issued and the City's estimated population during the past eight years.

Table IV-11: Annual Building Permits and Estimated Population

Year	Building Permits Issued	Population*	Percent Increase
1960		313	
1970		338	
1980		558	
1990		727	
2000**	11	991	
2001	14	1,034	4.3%
2002	34	1,137	10.0%
2003	115	1,487	30.7%
2004	144	1,924	29.4%
2005	198	2,526	31.3%
2006	295	3,355	32.8%
2007	139	3,765	12.2%
2008	130	4,145	10.1%

*NOTE: Population has been projected for 2000 to 2008 using building permits, a density of 3.04 for single family housing, and 2.5 for apartment unit housing. This information indicates population of Harrisburg as of December 31, 2008.
**NOTE: Census data indicates the population as of April 1, 2000 was 958. The population is projected to have increased to 991 by the end of the year based on the number of building permits issued.

Much of the population increase is due to Harrisburg's proximity to the City of Sioux Falls, which has experienced a strong growth rate for the last several decades. It is important to keep in mind that this level of growth is highly dependent on the economy of the region, and changes to that economy would greatly impact population projections.

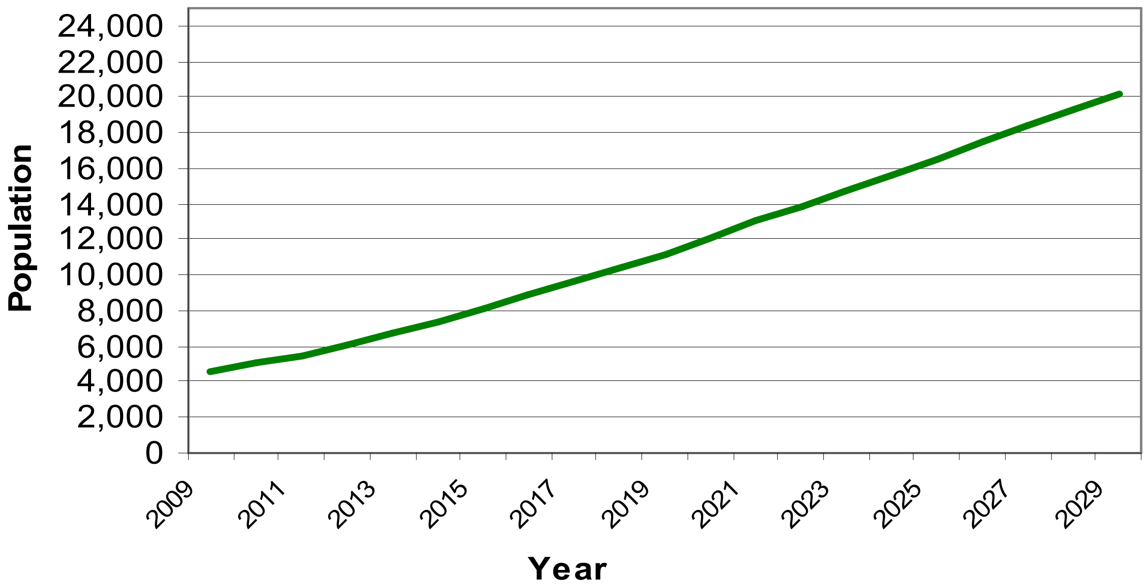
When compared to surrounding cities, Harrisburg's recent population increase has been quite high. During the past ten years, the nearby, slightly larger cities of Lennox, Tea, Brandon, and Hartford have had annual population increases between 4.6% and 12.15%.

The City of Harrisburg is not expected to maintain a population increase of over 10% for the next ten to twenty years. As the population increases, the annual percentage increase will decline. In addition, economic factors can greatly affect population increases. Therefore, Harrisburg's population is expected to increase 10% from 2009 to 2016, 8% from 2017 to 2021, 6% from 2022 to 2026 and 5% from 2027 to 2029.

Considering the recent increase in Harrisburg's population and its proximity to Sioux Falls, these projections over the next twenty years appears to be reasonable for design purposes. The projections indicate

Harrisburg's population in 2029 could reach 20,223. Figure IV-1 illustrates the population projection.

Figure IV-1: Population Projection for Harrisburg, SD



This differs from the population projection made during 2007, when the City first began to prepare this Facility Plan. At that time the lagoons were not projected to reach capacity until 2011. Therefore, population projections were completed for a 20-year planning period between 2011 and 2031. A two-phased approach was used for planning purposes. Because of this, the projected flows and loadings in the report will at times refer to years 2021 and 2031. For the projection completed in 2007, Harrisburg's population was expected to increase 12% from 2006 to 2010, 10% from 2011 to 2016, 8% from 2017 to 2021, and 6% from 2022 to 2031. Table IV-12 lists the projected population for Harrisburg initially completed during the 2007 phase of the Facility Plan, as well as the revised population projection completed in 2009.

Table IV-12: Annual Projected Population for Harrisburg, SD

Year	Projected Population 2007 Draft Facility Plan	Projected Population 2009 Facility Plan
2007	3,758	
2008	4,209	
2009	4,714	4,559
2010	5,280	5,015
2011	5,808	5,517
2012	6,389	6,068
2013	7,027	6,675
2014	7,730	7,342
2015	8,503	8,077
2016	9,353	8,884
2017	10,102	9,595
2018	10,910	10,363
2019	11,783	11,192
2020	12,725	12,087
2021	13,743	13,054
2022	14,568	13,837
2023	15,442	14,668
2024	16,368	15,548
2025	17,351	16,481
2026	18,392	17,469
2027	19,495	18,343
2028	20,665	19,260
2029	21,905	20,223
2030	23,219	
2031	24,612	

Assuming 2.5 homes per acre and 3 people per home, the expected increase from the 2008 population to the 2029 population will require approximately 2,090 acres of additional residential land. This land projection does not take into account commercial, institutional, governmental or industrial land needs. The City, in conjunction with the Southeast Council of Governments (SECOG), revised the City's Future Land Use Map as part of the Comprehensive Plan in early 2005. A copy of the Future Land Use Map is provided at the end of Appendix B.

2. Forecasts of Flows and Waste Loads

Typically, the planning period for a WWTP improvements project is 20 years. When this study began in 2007, projected design flows and loadings were established for two distinct, 10-year design periods given the uncertainty of the current population growth rate. The phased approach minimized risk and overall project cost associated with a 20-year planning period design based on highly uncertain population growth projections. The two design periods had time periods of 2011-2021 for Phase One, and 2021-2031 for Phase Two. The revised population projection also used a phased approach with the first planning period from 2009 to 2019, and the second from 2019 to 2029.

a. Hydraulic Load

Table IV-13 shows projected influent wastewater flows based on population projections for both the 2007 and 2009 population projections for selected years. Per capita flow values were estimated as very limited historical influent flow data was available. Per capita flow values are based on values found in engineering reference texts and are representative of flow for new sanitary sewer collection systems. The projected flows are assumed to include commercial flows. No significant industrial flows are anticipated for either design period.

Average dry weather (ADW) is the daily average flow when the groundwater is at or near normal and runoff is not occurring. Average wet weather (AWW) is the daily average flow for the wettest 30 consecutive days for mechanical plants. The maximum wet weather (MWW) is the total maximum flow received during any 24 hour period when groundwater is high and runoff is occurring. Peak hourly wet weather (PHWW) is the total maximum flow received during one hour when the groundwater is high, runoff is occurring, and the domestic, commercial and industrial flows are at their peak.

ADW flows were calculated assuming 75 gpcd. AWW flows were calculated assuming 100 gpcd. MWW flows were calculated by multiplying the AWW by an assumed peaking factor of two (2). PHWW flows were calculated by multiplying the AWW by a population based peaking factor as outlined in Ten States Standards. This varied from a factor of 3.4 in 2007 to a peaking factor of 2.8 in 2021 and 2.6 in 2031. Annual flow projections and the peaking factors for both the 2007 population projections and the revised 2009 projections is provided in Table D-1 in Appendix D.

Table IV-13: Projected Wastewater Influent Flows

Year	Design Condition							
	Based on 2007 Population Projection				Based on 2009 Population Projection			
	ADW, gpd	AWW, gpd	MWW, gpd	PHWW, gpd	ADW, gpd	AWW, gpd	MWW, gpd	PHWW, gpd
2007	281,854	375,805	751,610	1,261,754				
2008	315,676	420,901	841,803	1,394,632				
2009	353,557	471,410	942,819	1,540,851	341,933	455,910	911,821	1,496,258
2010	395,984	527,979	1,055,957	1,701,678	376,126	501,501	1,003,003	1,626,769
2011	435,582	580,777	1,161,553	1,849,257	413,739	551,652	1,103,303	1,768,135
2012	479,141	638,854	1,277,708	2,009,039	455,113	606,817	1,213,633	1,921,215
2013	527,055	702,740	1,405,479	2,181,986	500,624	667,498	1,334,997	2,086,931
2014	579,760	773,014	1,546,027	2,369,133	550,686	734,248	1,468,497	2,266,280
2015	637,736	850,315	1,700,630	2,571,596	605,755	807,673	1,615,346	2,460,332
2016	701,510	935,346	1,870,693	2,790,576	666,330	888,440	1,776,881	2,670,241
2017	757,631	1,010,174	2,020,348	2,980,333	719,637	959,516	1,919,031	2,852,157
2018	818,241	1,090,988	2,181,976	3,182,431	777,208	1,036,277	2,072,554	3,045,921
2019	883,700	1,178,267	2,356,534	3,397,647	839,384	1,119,179	2,238,358	3,252,280
2020	954,396	1,272,529	2,545,057	3,626,805	906,535	1,208,713	2,417,427	3,472,023
2021	1,030,748	1,374,331	2,748,662	3,870,781	979,058	1,305,410	2,610,821	3,705,993
2022	1,092,593	1,456,791	2,913,581	4,065,926	1,037,801	1,383,735	2,767,470	3,893,145
2023	1,158,149	1,544,198	3,088,396	4,270,527	1,100,069	1,466,759	2,933,518	4,089,374
2024	1,227,637	1,636,850	3,273,700	4,485,031	1,166,074	1,554,765	3,109,529	4,295,111
2025	1,301,296	1,735,061	3,470,122	4,709,910	1,236,038	1,648,051	3,296,101	4,510,805
2026	1,379,373	1,839,165	3,678,329	4,945,656	1,310,200	1,746,934	3,493,867	4,736,929
2027	1,462,136	1,949,515	3,899,029	5,192,786	1,375,710	1,834,280	3,668,561	4,934,653
2028	1,549,864	2,066,485	4,132,971	5,451,843	1,444,496	1,925,994	3,851,989	5,140,341
2029	1,642,856	2,190,474	4,380,949	5,723,396	1,516,721	2,022,294	4,044,588	5,354,310
2030	1,741,427	2,321,903	4,643,806	6,008,043				
2031	1,845,913	2,461,217	4,922,434	6,306,412				

b. Organic Load

Projected influent wastewater loadings are summarized in Table IV-14. Influent loading conditions for BOD_5, TSS, ammonia, and TKN were calculated using projected domestic populations and typical per capita loading rates. Commonly accepted values were used to determine the raw water BOD_5, TSS, NH_3-N, and TKN loadings due to the limited amount of historical data available. Per capita average loading rates for BOD_5, TSS, ammonia, and TKN were 0.20 ppd, 0.22 ppd, 0.025 ppd, and 0.038 ppd. Maximum values were calculated using the maximum to average ratio from the wastewater sampling as described above in Table IV-1. The projected loadings are assumed to include loadings from commercial flows. No significant industrial loadings are anticipated during the design period.

Table IV-14: Projected Wastewater Influent Loadings

Year	Based on 2007 Population Projection								Based on 2009 Population Projection							
	BOD		TSS		NH3		TKN		BOD		TSS		NH3		TKN	
	Average (ppd)	Max (ppd)	Average (ppd)	Max (ppd)	Average (ppd)	Max (ppd)	Average (ppd)	Max (ppd)	Average (ppd)	Max (ppd)	Average (ppd)	Max (ppd)	Average (ppd)	Max (ppd)	Average (ppd)	Max (ppd)
2007	752	997	827	1058	92	107	142	164								
2008	842	1117	926	1185	103	119	159	184								
2009	943	1251	1037	1327	116	134	178	206	912	1210	1003	1284	112	129	172	199
2010	1056	1401	1162	1487	130	150	199	230	1003	1330	1103	1412	123	142	189	219
2011	1162	1541	1278	1635	142	165	219	253	1103	1464	1214	1553	135	156	208	241
2012	1278	1695	1405	1799	157	181	241	279	1214	1610	1335	1709	149	172	229	265
2013	1405	1864	1546	1979	172	199	265	307	1335	1771	1468	1880	164	189	252	291
2014	1546	2051	1701	2177	190	219	292	337	1468	1948	1615	2068	180	208	277	320
2015	1701	2256	1871	2394	209	241	321	371	1615	2143	1777	2274	198	229	305	352
2016	1871	2481	2058	2634	229	265	353	408	1777	2357	1955	2502	218	252	335	388
2017	2020	2680	2222	2845	248	286	381	441	1919	2546	2111	2702	235	272	362	419
2018	2182	2894	2400	3072	268	309	412	476	2073	2749	2280	2918	254	294	391	452
2019	2357	3126	2592	3318	289	334	445	514	2238	2969	2462	3152	275	317	422	488
2020	2545	3376	2800	3583	312	361	480	555	2417	3207	2659	3404	297	343	456	527
2021	2749	3646	3024	3870	337	390	519	600	2611	3463	2872	3676	320	370	493	569
2022	2914	3865	3205	4102	357	413	550	636	2767	3671	3044	3897	339	392	522	604
2023	3088	4097	3397	4348	379	438	583	674	2934	3891	3227	4130	360	416	554	640
2024	3274	4343	3601	4609	402	464	618	714	3110	4125	3420	4378	381	441	587	678
2025	3470	4603	3817	4886	426	492	655	757	3296	4372	3626	4641	404	467	622	719
2026	3678	4879	4046	5179	451	522	694	802	3494	4635	3843	4919	429	495	659	762
2027	3899	5172	4289	5490	478	553	736	850	3669	4866	4035	5165	450	520	692	800
2028	4133	5482	4546	5819	507	586	780	901	3852	5110	4237	5424	472	546	727	840
2029	4381	5811	4819	6168	537	621	827	956	4045	5365	4449	5695	496	573	763	882
2030	4644	6160	5108	6538	570	658	876	1013								
2031	4922	6530	5415	6931	604	698	929	1074								

3. Flow Reduction

In the mid-1990's, Harrisburg completed an inventory within the City to determine if sump pumps were discharging into the City's sanitary sewer system. Excessive flows were noticed and the inventory identified several violators. These sump pump systems were modified to prevent discharge into the City sewer system.

Harrisburg intends to complete an inventory of homes again in 2009 to potentially reduce unnecessary flows into the sanitary sewer system.

V. DEVELOPMENT AND EVALUATION OF PRINCIPAL ALTERNATIVES

A. ALTERNATIVE EVALUATION

With the City's total containment ponds projected to be at capacity, Harrisburg realizes that it must begin moving forward with a plan for construction of a new wastewater treatment and disposal system.

1. No Action

One option that all municipalities have is the "No Action" alternative. Future population and flow projections indicate that the existing containment ponds will reach capacity in 2009. If the City does nothing, they will be forced to halt economic development to eliminate additional wastewater sources. For economic purposes, the City does not want to place a cessation on development.

If the future wastewater flows exceed the treatment plant's capacity, the City will either be forced to conduct planned emergency discharges or face overtopping of their existing evaporation ponds. Continued emergency discharges are not acceptable and unauthorized discharges would result in State and Federal fines. Overtopping their existing lagoon system will likely cause the existing collection system to back up and surcharge raw sewage into homeowner's basements. This outcome would be detrimental to the environment and residents of the City of Harrisburg.

As a result, this alternative is not desirable and will not be discussed further in the report.

2. Expansion of the City's Existing Evaporation Ponds

This alternative proposes to expand the City's existing total containment ponds to meet the future needs of the City. The SD DENR states that lagoons should be sited at least one-half (½) mile from the community, and one-fourth (¼) mile from a farm house or residence whenever possible. The high water line of ponds is required to be at least 50-feet from a residence. With new housing developments on the north, west, and east sides of the existing WWTP, expansion of the existing evaporation ponds would likely have to occur to the south. With the existing site, the recommended separation of one-half (½) mile from the community, and one-fourth (¼) mile from a farm house or residence could not be met. In addition, the land area required to accommodate the flows in 2029 for the 20-year design period is significant. Approximately 372.16 acres are required. Continuing to use the existing evaporation ponds would reduce the number of acres needed to 332.17. The calculation for the future evaporation ponds is provided in Table E-1 in Appendix E. At approximately $10,000 per acre, the land costs alone for this option would likely be $3,321,700, with additional construction costs for the ponds. It is expected that a pond this large would be difficult to site. Due to the required land area and siting constraints, this option is neither feasible nor desirable and will not be discussed further in this report.

3. Aerated Lagoons

Aerated lagoons are a good choice for smaller communities with basic treatment requirements (BOD_5 and TSS removal) to meet discharge limits. Aerated lagoons treat waste through waste conversion or uptake to biological organisms. Aerobic or heterotrophic organisms are targeted by maintaining an adequate level of dissolved oxygen. These systems are provided with air supply for two reasons: to maintain sufficient oxygen and to provide mixing to maintain the contents in suspension. Solids separation and recycle can also be incorporated if a higher rate of treatment is preferred to reduce the footprint, or to meet stricter effluent requirements.

In addition to lagoon modifications, Harrisburg's wastewater treatment system will require an upgrade to their influent piping, and the addition of phosphorus removal, ultraviolet disinfection, and discharge piping from cell three to Ninemile Creek. For both processes discussed herein to modify the existing lagoons, the following modifications were assumed:

a. Influent Piping Modifications

- Upsize the existing 12-inch sanitary sewer pipe from the intersection of Tiger Street and Prairie Street east to the railroad right-of-way with a 24-inch PVC sanitary sewer pipe.
- Replace the existing 12-inch VCP crossing the railroad right-of-way with four parallel 12-inch sanitary sewer lines to achieve the minimum cover required by the railroad. The new pipe will be installed by boring and jacking 20-inch casing pipe under the railroad right-of-way. Junction boxes will be required to transition from one 24-inch PVC sanitary sewer pipe to the four 12-inch pipes.
- Flow will be collected on the east side of the railroad right-of-way in a junction box to combine the flow from both sides of the railroad tracks. Sewage will be carried south from the junction box in a 27-inch PVC sanitary sewer pipe.
- Pipe will be increased to 30-inch immediately prior to primary treatment

b. Phosphorus Removal

Harrisburg will be given an effluent phosphorus limit if allowed to the discharge into Ninemile Creek because of their proximity to Lake Alvin. The phosphorus removal will require chemical addition of flocculants within the lagoon cells and ahead of the sand filters. Chemical pumps were assumed in the blower building and ahead of the sand filters.

A modular and compact sand filter was analyzed to remove phosphorus from the wastewater. The sand filters enable simple construction, quick installation, and low maintenance. The sand filter is ideal for installations with extreme site restrictions or as an add-on to existing treatment systems. The sand filters can be installed in an underground concrete tank and operate under a constant backwash mode, continuously cleaning the filter bed. This provides for a consistently high quality filtrate.

Advantages

- No flocculation zones are necessary
- No moving parts, thus minimal wear and tear and low O&M costs
- Low air consumption, no internal pumping; thus low energy costs
- Small footprint

Disadvantages

- Capitol cost of the system
- Chemical handling requirements
- Upgrade in staffing and operator grade for a more complicated treatment system

c. Ultraviolet Disinfection

Ultraviolet disinfection would be required after the sand filters prior to discharge to the effluent gravity sewer discharging to Ninemile Creek. The equipment proposed would be similar to the disinfection equipment described for the Harrisburg WWTP option later in this section.

d. Effluent Piping Modifications

- 30-inch effluent piping will be installed on the east side of the railroad right-of-way from the sand filter to the UV disinfection system
- 30-inch effluent piping will be installed from the UV disinfection system to an outfall at Ninemile Creek. The installation will require approximately 2,300 feet of 30-inch sanitary sewer pipe, 5 manholes, crossing 274th Street, and miscellaneous surface restoration

Nitrogen removal in cold climates is a common problem with aerated lagoons. The ammonia nitrogen (as N) limit will be required initially as stated above in the table of effluent limits. Lagoon covers have been used to maintain wastewater temperatures to allow nitrogen removal in cooler climates. Recent technology developed in the upper Midwest and tested in Canada shows nitrogen removal can be accomplished in colder temperatures using a submerged attached growth reactor. The two options considered in this Facility Plan include:

- Lemna Technologies LEMTEC Process with LPR
- Nelson Environmental OPTAER Process with SAGR

a. LEMTEC Process

The LEMTEC Biological Treatment Process (LBTP) is a covered lagoon system that consists of a complete mix zone, partial mix zone, quiescent/settling zone, and Lemna Polishing Reactor (LPR). Flow first enters the complete mix zone, next into the partial mix and quiescent zones, and lastly thru the LPR. Fine-bubble diffused aeration is provided in the complete mix and partial mix zones and in the LPR, in varying proportions.

Covered lagoon systems allow for design of higher rate aerated lagoon systems for advanced treatment of industrial and domestic wastewater. These processes use known information from aerated lagoon installations and combines this information with the benefits of a covered, insulated process. The covers retain heat allowing higher biological rates, especially for nitrification, throughout cold-temperature periods. The same kinetic principals that drive aerated lagoon design on an engineering/regulatory basis are used for the design of this process. The benefits are increased reaction rates and credit for eliminating the volume required for ice accumulation.

For Harrisburg, the proposed covered lagoon treatment system would reuse the existing earthen lagoon cells and would incorporate modifications to the aeration system, add insulated floating covers, add flow baffles inside the lagoon cells, and add a lagoon effluent polishing system for residual ammonia removal.

2. Headworks Treatment Process

Preliminary treatment is required ahead of the complete mix zone to ensure the capture of solids greater than ½-inch in size. Grit removal is also recommended to extend the life of downstream mechanical equipment and to reduce the volume of settleable material that cannot be re-suspended in the aeration cells of the lagoon.

3. Aerated Lagoon Improvements

Several LEMTEC system configurations were considered for Harrisburg, but the option selected for analysis for this Facility Plan was the option to use the existing lagoon cells at their current depths. Phase One allows Harrisburg to modify one cell immediately to get to the 10-year AWW design flow of 1.03 mgd (based on 2007 projections). Phase Two would involve modifications to the second lagoon cell to run in parallel to reach the 20-year AWW design flow of 1.84 mgd (based on 2007 projections).

The LEMTEC process divides the lagoon cell into a complete mix, partial mix and settling zone with custom baffles designed to minimize short-circuiting. For Phase One, based on the existing 10.2 acre ponds, the complete mix, partial mix, and settling cells will provide 5.0, 6.8, and

3.5 days of detention time, respectively. Further input from the SD DENR, in regards to the minimum detention time needed for complete and partial mix, would be needed to select a final layout for the LEMTEC system if it is the selected alternative.

The complete mix zone is designed for rapid BOD removal. Aeration and mixing are provided by a combination of fine bubble diffusers and floating mechanical mixers. Positive Displacement (PD) blowers are used to provide air to meet the complete mix zone aeration/oxygen demands. The zone is designed with at least 13.4 horsepower (hp)/million gallons (MG) or 0.12 standard cubic feet per minute (SCFM) per square foot of diffuser mixing energy to ensure suspension of solids. Additionally, ammonia can be removed in the complete mix zone by the heterotrophic bacteria present to support its nitrogen requirements for growth. The insulated cover minimizes heat loss from the complete mix zone and little sludge accumulation occurs within this zone.

The partial mix zone is designed with a lower level of aeration and mixing for optimal BOD removal. Aeration density in the partial mix zone is less than that provided in the complete mix zone. PD blowers are used to provide air to meet the partial mix aeration/oxygen demands. The partial mix zone is designed with 6.7 hp/MG and 0.01 SCFM/SF. The partial mix zone is also covered to reduce heat loss.

A permeable insulated floating cover is used to retain heat in the complete and partial mix zones for optimization of BOD and ammonia removal. The intent of the cover system is to maintain temperatures at 9 to 10 degrees Celsius. The cover system is a modular design of interconnecting panels and allows the operator to walk across the cover for access to diffusers and laterals with the system in operation.

The quiescent/settling zone is designed for total suspended solids (TSS) removal, algae prevention, and storage of sludge. The cover system prevents algae growth by eliminating sunlight below the cover and improving clarification. The cover prevents wind action from disturbing the water surface and thus allows the solids to settle. The insulation in the cover minimizes seasonal and diurnal temperature fluctuations, thus reducing stirring caused by thermal currents.

Solids storage is confined to the quiescent/settling zone. Given the long design detention time, the LBTP generates a minimal amount of solids. The anaerobic environment in the quiescent/settling zone promotes digestion of solids, thus reducing the sludge volume. Sludge removal from the settling zone is typically required every 8-10 years.

The LPR is the final process in the LBTP. The LPR is a fixed film reactor consisting of aerated, submerged, attached growth media modules. The media modules maintain a population of heterotrophic and nitrifying bacteria for final polishing of the lagoon effluent. The LPR is designed to meet the BOD and ammonia effluent limits. Aeration to the LPR is provided by the same PD blowers that supply aeration to the complete and partial mix zones.

4. Blower Building

Based on the LEMTEC layout selected for preliminary design, two (2) 25 horsepower PD blowers would be required for Phase One and three (3) would be required for Phase Two to meet the aeration/oxygen demand. In each case, one additional blower would be provided as a standby to meet redundancy requirements. All blowers are equipped with enclosures for sound attenuation and can be located outdoors. However, due to the close proximity to residential areas, a blower building was assumed.

5. Other Similar Installations

A similar LEMTEC system as described above has been approved by the Iowa Department of Natural Resources for the City of Villisca, Iowa. Additional redundancy requirements above those typical for aerated lagoon systems were required, including redundancy of the complete mix zone to provide 50% capacity with one complete mix zone out of service. As proposed during preliminary design, each train of the LEMTEC system designed for the Rock Valley, Iowa wastewater treatment system is designed to operate at 50% of the average design flow and loading.

6. Advantages and Disadvantages

Several of the advantages and disadvantages associated with this type of system are listed below:

a. Advantages:
   - Reuse of existing aerated lagoon infrastructure
   - Installation in existing lagoon without draining
   - Allows for ammonia removal (nitrification) year round
   - Sludge storage is accomplished within the system with removal required every 8 to 10 years

- Anaerobic digestion in the settling zone minimizes the amount of sludge produced
- Cover system helps minimize odors from process, maintain higher temperatures, and reduce algae
- LEMTEC system has received prior approval in the nearby state of Iowa

b. Disadvantages:
- High aeration equipment operating costs
- Preliminary treatment is required ahead of the complete mix zone
- Freezing potential in LPR

b. OPTAER Process with SAGR

The OPTAER treatment process is an aerated lagoon system that consists of a complete mix zone, partial mix zone, quiescent zone, and submerged attached growth reactor (SAGR). Flow first enters the complete mix zone, next into the partial mix and quiescent zones, and lastly thru the SAGR. Fine-bubble diffused aeration is provided in the complete mix and partial mix zones and medium/coarse bubble diffused aeration is provided in the SAGR.

For Harrisburg, the proposed OPTAER system would reuse the existing lagoons with several modifications. Cell #1 would be converted into a complete mix and partial mix zone by dividing the cell with an impermeable geomembrane baffle curtain. Existing Cells #2 and #3 would be converted into partial mix zones which would discharge into the SAGRs. Cell #3 would be divided into four (4) separate basins with berms. Cell #3 would be similar in size and shape to Cells #1 and #2. Cell #4a and #4b would be two (2) parallel SAGR cells for redundancy. Cell #5 would be a quiescent/settling zone. The total hydraulic detention time for the system is 33.4 days at 1.831 mgd.

Given the unique combination of the aerated lagoon and fixed-film reactor technologies, regulatory approval of the OPTAER process prior to design is anticipated. Additional redundancy requirements above those typical for aerated lagoon systems may be required. As proposed each train of the OPTAER system for Harrisburg is designed to operate at 50% of the average design flow and loading.

1. Headworks Treatment Process

Similar to the Lemna process, screening is recommended ahead of the lagoons to optimize performance and maintain equipment downstream. A screening building was assumed.

2. Aerated Lagoon Improvements

The complete mix zone is designed for accelerated BOD removal. The diffused aeration system in the complete mix zone is designed to provide oxygen for BOD removal and to ensure complete mixing of the contents, keeping the biomass is suspension. Supplemental mixing equipment (surface mixers) is not required in the complete mix zone. Little sludge accumulation occurs within the complete mix zone. Biomass would be carried from the complete mix cells to the partial mix zones. Some accumulation of solids is expected at the side embankments.

The partial mix zone is designed for long term BOD removal and aerobic digestion of settled biomass at the sludge/water interface. Aeration density in the partial mix zone is less than that provided in the complete mix zone. Due to the decrease in aeration density, convection cells are created between diffusers where solids (biomass) settle downward as water and air bubbles rise to the surface. The long detention time combined with the low sludge production in the partial mix zone promotes BOD removal and digestion.

MixAir Technologies (MAT) TA-22 fine bubble diffusers provide aeration to the waste water. The diffusers are made of a tubular micro-porous membrane that expands under higher air pressure to remove fouling from the pours. The diffusers are connected to a High Density Polyethylene (HDPE) header with marine grade rope. These diffusers have a design life of 10-years, slightly higher than the average 5-7 year lifespan for membrane diffusers.

The SAGR is designed to provide nitrification (ammonia removal) in cold weather climates. The SAGR consists of a gravel bed, horizontal influent distribution chamber, horizontal effluent collection chamber, and perforated-pipe lateral aeration system located in the gravel bed. The gravel bed is covered with a layer of peat or mulch to retain heat. Nitrifying bacteria grow on the surface of the gravel media. The aeration system, known as LINEAR, provides the required oxygen for ammonia removal. Using the SAGR, the OPTAER system claims to be capable of achieving 90% ammonia removal at temperatures as low as 4 to 5°C.

Diffused aeration in the SAGR LINEAR is provided via Low Density Polyethylene (LDPE) piping embedded in the gravel layer with air releases spaced at 9-inches on center. The diffuser locations are spaced according to the projected oxygen demand in the SAGR and perpendicular to the influent flow. The perpendicular orientation ensures that wastewater flow cannot channelize through the gravel layer.

3. Blower Building

PD blowers are used to provide air to meet the system's aeration/oxygen demands. Based on Harrisburg's influent design loading, the OPTAER MAT aeration system will have six (6) 150 hp positive displacement blowers, each capable of providing 2549 SCFM at a discharge pressure of 5.6 psi. The blowers will be capable of operating temporarily at 8.8 psi for diffuser cleaning at 156.7 bhp. Five (5) blowers will provide aeration to the complete mix and partial mix zones with one on standby. The standby blower would provide airflow to the SAGR LINEAR system at full build out. A blower building was assumed.

Air supply to the SAGR LINEAR system will be provided by one (1) 60 hp positive displacement blower. The blower will be capable of providing 1266 SCFM at a discharge pressure of 5.1 psi. The blower would be capable of providing 7.7 psi for maintenance purging at 44.6 bhp. At full build-out redundancy will be provided by the MAT aeration system standby blower.

All blowers will be provided with sound attenuating enclosures. Sound levels included with the OPTAER MAT enclosures will be 79 dB (A) while enclosures with the SAGR LINEAR will be at 73 dB(A).

2. Advantages and Disadvantages

Several of the advantages and disadvantages associated with this type of system are listed below:

a. Advantages:
   - Reuse of existing aerated lagoon infrastructure
   - Installation in existing lagoon without draining
   - Allows for ammonia removal (nitrification) during cold weather periods
   - No supplemental mixing equipment needed
   - No need for primary treatment
   - Sludge storage is accomplished within the system
   - Aerobic digestion in the partial mix zone minimizes the amount of sludge produced

b. Disadvantages:
   - High aeration equipment operating costs
   - Careful control of influent organic loading to SAGR required to minimize clogging potential

- Any removed gravel bed material must be treated because of attached biomass prior to ultimate disposal
- Freezing potential in SAGR
- No additional capacity in design beyond future design loadings
- No installations in the state of South Dakota or surrounding region

4. New Mechanical Wastewater Treatment Plant

A mechanical WWTP was also considered to meet Harrisburg's future wastewater treatment needs. In 2007, when this report was first drafted, it was assumed that the effluent from a WWTP had to be discharged to the Big Sioux River. It was thought that there was not enough separation to Lake Alvin to discharge to Ninemile Creek. The options discussed below describe the processes needed for a mechanical plant discharging to the Big Sioux River.

This option evaluates the gravity interceptor, force main, equalization basin, lift station, and mechanical WWTP required to convey wastewater from Harrisburg to the Big Sioux River. The WWTP is proposed adjacent to, or near the Big Sioux River to maximize the future area the WWTP would eventually serve via a gravity collection system. Large diameter gravity sanitary sewer piping is proposed from the current total containment ponds to a lift station located just east of the 2025 growth area along Ninemile Creek. Force main is proposed from the lift station to the WWTP. Exhibit E-1 in Appendix E provides a proposed layout for the gravity interceptor, lift station and force main. It also identifies several potential WWTP sites.

In addition, three treatment options were evaluated for the new mechanical WWTP, including:

- Sequencing Batch Reactor (SBR)
- Conventional Activated-Sludge
- Membrane Bioreactor (MBR)

As the proposed equipment for the mechanical WWTP is discussed, slight alterations may be required for each treatment option being evaluated.

a. Gravity Sanitary Sewer Piping

Gravity sanitary sewer flow is feasible from the City of Harrisburg to the west end of Lake Alvin. Force main is required from the west end of Lake Alvin to the proposed WWTP near the Big Sioux River due to the steep topography of the land. A gravity outfall is proposed from the WWTP to the river.

The cost to install large diameter sanitary sewer interceptors from the current evaporation ponds to Lake Alvin would be significant.

In addition, Lake Alvin is approximately two (2) miles outside the 2025 Growth Plan Area. Installing the large diameter gravity sanitary sewer piping required to serve basins so far outside the 2025 growth area is costly and premature at this time. As a result, gravity interceptors are proposed from the evaporation ponds to a lift station site located along Ninemile Creek just east of the 2025 Growth Plan area as shown in Exhibit E-1 in Appendix E. The gravity trunk sewer to the proposed lift station will consist of a network of 12-, 27-, 42-, and 48-inch diameter gravity sewer totaling approximately 12,600-feet.

b. Equalization Basin

An equalization basin is proposed at the lift station site to contain the difference between projected peak hour flows and maximum day flows. A safety factor of 2.0 will be used to size the basin because of the uncertainty of influent flows and to allow for additional storage capacity. The equalization basin will reduce the needed pumping capacity and the overall size of the lift station. Some equipment at the WWTP can also be reduced in size, since influent flows would not exceed maximum day projections. It also offers operational flexibility should the need arise to temporarily shut down the lift station for maintenance issues. The equalization basin would remain dry most of the time, and fill when flows exceed projected maximum day 2031 conditions. Design parameters for the equalization basin are provided in Table V-1.

Table V-1: Equalization Basin Design Parameters

Parameter	Value
Needed Volume (gallons)	1,385,000
Approx. Bottom Length (ft)	162
Approx. Bottom Width (ft)	102
Approx. Top Length (ft)	210
Approx. Top Width (ft)	150
Approx. Usable Depth (ft)	8
Approx. Total Depth (ft)	11
Top Area (ft^2)	31,500
Slope	3:1
Safety Factor	2
Number of Basins Required	2
Land Requirement (acres)	5.5

c. Lift Station

The lift station will be sized for two (2) pumps (one duty, one standby) with each pump capable of handling the 2021 Design year MWW flow. Once the 2021 Design year MWW flow has been reached, both pumps will be replaced with two (2) new pumps (one duty, one standby) with each pump capable of handling the 2031 MWW flow. The specific size, flow rate, and operating head condition will be evaluated during schematic design once a site is selected. Preliminary calculations indicate that the 2021 design year pumps would be sized for 1,900 gpm at 160 feet of total dynamic head (TDH), and the 2031 design year pumps would be sized for 3,400 gpm at 275 feet of TDH. Variable frequency drives (VFDs) will be used to match the pumping rate with the influent flow rate, reduce energy costs, extend motor life, reduce the required starting current, reduce maintenance costs, and to help prevent the wastewater from becoming septic.

A mechanical course screen would be located within the lift station to capture large solids and debris within the wastewater, and protect downstream pumps. A bypass channel adjacent to the influent channel(s) with a manual bar screen will be provided to divert flow around the mechanical screen should it need to be taken out of service. The screens would be located ahead of the pumps.

The wetwell will be sized to minimize holding time to reduce septic conditions from developing and according to SD DENR requirements. The use of a “self-cleaning” wetwell design will be investigated during schematic design to minimize maintenance and cleaning needs, eliminate odors, and reduce wetwell size. Odor control will be provided at the lift station site to reduce impacts to adjacent properties.

1. Wetwell/Drywell Layout versus Submersible Layout

The two main lift station layout options are: 1) Wetwell/drywell design, and 2) Submersible design.

The wetwell/drywell design would consist of separate wetwell and drywell vaults. The drywell vault would house the pumps and associated valves. A section and plan view of a wetwell/drywell design is provided in Exhibit E-2 and E-3 in Appendix E. Benefits of this design include:

- Ease of routine maintenance on pumps and valves and detection of small problems early before they become large problems
- Use of alternative pump drive configurations
- Smaller wetwell footprint required

Disadvantages include:

- Construction of two deep vault structures
- Large drywell footprint required for sufficient suction pipe length

The submersible design would consist of a separate wetwell and valve vaults. The wetwell and the valve vaults would house the pumps associated valves, respectively. A section and plan view of a submersible design is provided in Exhibit E-4 and E-5 in Appendix E. Benefits of this design include:

- Construction of only one deep vault structure
- Construction of shallow and smaller footprint valve vault structure

Disadvantages include:

- Requires pumps to be removed for routine maintenance
- Use of submersible-type pumps only
- Submersible pump dimensions may require larger wetwell footprint

The wetwell/drywell alternative was selected due to ease of maintenance and City familiarity.

d. Force Main

The force main from the proposed lift station to the new Harrisburg WWTP will consist of approximately 29,000-feet of 16-inch diameter pipe. Due to the anticipated high discharge pressure from the pumps, a portion of the force main may have to be high pressure ductile iron pipe (DIP) until the pressures drop to allow for the safe use of polyvinyl chloride pipe (PVC). Until the final alignment is selected, it is uncertain how much DIP will be required.

e. WWTP Preliminary Treatment

Wastewater flows from the lift station will be directed through the force main to the headworks building of the new WWTP. It is uncertain whether the strong population growth trend will continue over the design period, since it is very dependent on the economy of the region. Therefore, preliminary treatment systems will initially be sized for the 2021 Design year flows and loadings. After ten years, additional capacity can be added to accommodate the 2031 Design year flows and loadings. Exhibit E-6 in Appendix E shows the process flow diagram for preliminary treatment.

1. Influent Screening

While the lift station incorporates a mechanical bar screen, a fine screen is still needed in the preliminary treatment process to remove undesirable materials such as plastics and rags that pass through the bar screen. The fine screen also protects downstream equipment and improves the solids disposal process. Fine screening increases the amount of organic material that is removed with the screenings. A screenings washer/compactor can be used to remove the organic material, dewater, and compact the screenings prior to disposal. This can be accomplished using an ancillary screenings washer/compactor, or by a screen with an integral screening washer/compactor.

The 2021 design will incorporate one (1) mechanical fine screen with a capacity of at least 2.74 mgd to handle the 2021 design year MWW event. A second mechanical screen shall be added in 2021 to increase capacity to 4.90 mgd for the 2031 design year MWW event. Under lower flow conditions in each design period the screen(s) will be operated with longer cleaning cycle times. A bypass channel adjacent to the influent channel(s) with a manual bar screen will be provided to divert flow around the mechanical screen(s), with sufficient capacity to handle the MWW event with the mechanical screen(s) out of service. Clear openings between the bars on the manual screen will be 1-inch.

Screen selection depends on channel depth, amount of debris, desired capture rate, requirements of secondary treatment, cleanliness of screenings, dryness of screenings, and maintenance. A mechanical fine screen with openings of one-quarter (¼) inches or less will be used ahead of the conventional activated-sludge and SBR systems. A second mechanical fine screen with two-dimensional openings of one (1) to three (3) millimeters is required ahead of the MBR system in addition to the one-quarter (1/4) inch screen.

Flow into the headworks building will come from the influent lift station. Therefore, the influent channel will be relatively shallow. The rotary screw and rotary sieve screens are best suited for shallow channel applications. High capture efficiencies are possible with the use of perforated and wedge-wire screening elements within the rotary screw and rotary sieve screen. The drum screen is best suited to meet the pretreatment screening requirements ahead of an MBR system. The drum screen can be provided with mesh screening element to provide two-dimensional screening.

Based on cost, the rotary screw screen is the most economical alternative. These screens shall be further evaluated based on secondary treatment system recommendations, building layout and other building restrictions during schematic design.

The following three types of fine screens were evaluated:

1) Rotary screw screen
2) Rotary sieve screen
3) Drum screen

a. Rotary Screw Screen

The rotary screw screen is a self-cleaning, in-channel or tank-mounted screen that uses a cylindrical screen basket. An inclined rotating auger cleans the screen basket and collects and transport solids from the influent flow stream. Wastewater flows into the open end of the inclined screen basket where solids are retained. The solids form a mat on the surface of the screen basket, improving the influent solid capture rate. The auger rotates within the screen basket, and brushes on the auger flights remove solids from the screen basket surface. Cleaning is activated when a pre-set differential water level between the upstream and downstream sides of the screen is reached. Screenings are then conveyed upward through an inclined auger tube.

The screen can be provided with an integral screenings washer/compactor, where organics are removed. The screenings are dewatered and compacted in the auger tube. Screenings are discharged at the upper end of the auger tube into a container or bagger.

Benefits of the rotary screw screen include:

- Moderate solids capture rate
- Two-dimensional screening with use of perforated plate screening basket
- Low profile, minimal headroom required
- Minimum channel width required
- Low headloss due to low angle of inclination (35-degrees)
- Pivots out of channel for maintenance
- Integral screenings washer/compactor

Disadvantages include:

- Lower hydraulic throughput capacity than other screening options
- Cleaning brushes and wear bars in transport tube require annual maintenance
- Needs to develop solids mat for high capture rates
- Larger footprint (building area) needed due to low angle of inclination

The capital cost for one (1) screw screen for the 2021 design year with an integral screenings washer/compactor is $50,000.

b. Rotary Sieve Screen

The rotary sieve screen is a self-cleaning, in-channel or tank-mounted screen that uses a cylindrical screen basket, rotating rake arm, and an inclined auger to collect and transport solids from the influent flow stream. Wastewater flows into the open end of the inclined screen basket where solids are retained on the bars of the screen basket. The solids form a mat on the surface of the screen basket, which improves the influent solid capture rate. The rake arm rotates within the screen basket to remove solids from the screen basket when a pre-set differential water level between the upstream and downstream sides of the screen is reached. Solids are deposited in a screening hopper located at the screen's central axis. Screenings are then transported from the hopper through an inclined auger tube. The screen can be provided with an integral screenings washer/compactor to remove organics. The screenings are removed, dewatered, and compacted in the auger tube. Screenings are discharged at the upper end of the auger tube into a container or bagger.

Benefits of the rotary sieve screen include:

- High solids capture rate with wedge-wire screen basket design
- Low profile, minimal headroom required
- Low headloss due to low angle of inclination (35-degrees)
- Pivots out of channel for maintenance
- Integral screenings washer/compactor

Disadvantages include:

- Needs to develop solids mat for high capture rates
- Not capable of two-dimensional screening
- Screen basket size requires larger channel width
- Larger footprint (building area) needed due to low angle of inclination

The capital cost for one (1) sieve screen for the 2021 design year with an integral screenings washer/compactor is $100,000.

c. Drum Screen

Drum-type screens have a cylindrical screen surface that rotates in a flow channel. Drum screen construction varies depending whether the screen is fed internally or externally. For internally fed screens, flow enters the inside of the screen through one end of the cylinder and flows outward. Screenings are captured on the interior surface of the cylinder. For externally fed screens, flow is distributed over the top of the unit and passes through to the interior with the screenings collected on the exterior. Internally fed screens generally have a higher hydraulic capacity than externally fed screens.

Influent wastewater can gravity flow or be pumped to the inlet of the drum screen. After passing through the screen, the wastewater enters a collection trough where it drains by gravity.

A spray wash system is used to clean the surface of an internally fed drum screen. The screenings collect in the invert of the inclined drum and gravity flow out the screen. For externally fed screens, a combination spray wash and scraper bar remove debris from the screen surface.

Drum screens are not equipped with integral screenings washers/compactors. Screenings from both arrangements are transported via a conveyor to an ancillary washer/compactor where they are washed to remove organics and compacted for dewatering.

Benefits of the drum screen include:

- Very high solids capture rate
- Provides two-dimensional screening ahead of MBR system
- Flow can be pumped directly to unit
- Operates on a continuous basis

Disadvantages include:

- May require additional upstream screening (1/4" openings) to protect drum screen and minimize excessive fouling
- Relatively low throughput capacity for externally fed screens
- High organics capture rate and limited screenings dewatering capacity; ancillary screenings conveyor and washer/compactor needed
- Large footprint (building area) and headroom required
- Separate spray wash water system required

The capital cost for one (1) rotary drum screen and washer/compactor required for the MBR system 2021 design year peak flow capacity is $200,000.

2. Influent Flow Measurement and Sampling

Influent flow will be measured using a Parshall flume with the capacity to measure the MWW event. The flume will be located indoors, downstream of the fine screen. A 9-inch wide flume throat is required to measure the 2.74 mgd 2021 MWW flow and the 4.90 mgd 2031 MWW flow.

An automatic sampler will be used to collect a daily composite influent wastewater sample. It will consist of a pump to collect the sample and refrigerated sample storage. The pump can be programmed to take a sample at regular time intervals (time-paced sampling) or based on an influent flow signal from the influent flow meter (flow-paced sampling).

3. Grit Removal

Grit removal is used to remove fine particle inorganics from the waste stream. Removal of these materials reduces wear and maintenance on downstream process equipment such as pumps, tanks, etc. Grit not removed from the wastewater is transferred to downstream treatment processes and reduces the capacity of these processes/basins. Also, land application of solids containing inorganic grit material is not desireable. Design criteria for the grit removal process is 100% removal of particles 65 mesh or greater with a specific gravity of 2.65.

The design will incorporate one (1) grit system (basin and equipment) with a capacity of 2.74 mgd to handle the 2021 MWW event. A second grit system will be added in 2021 to increase capacity up to the 2031 MWW event.

Three types of grit removal systems investigated for this application are:

- Aerated-type
- Detritor-type
- Vortex-type

a. Aerated-Type

Aerated-type grit removal uses air to induce a vertical roll to the wastewater stream. The grit settles to the bottom and is removed with a screw conveyor, air-lift pump, flooded suction or self-priming recessed impeller grit pump, or a chain-and-bucket system. Pumping grit from the basin is the preferred method of grit removal. Pumping eliminates mechanical equipment inside the basin, reduces wear on mechanical parts, and lessens the need to dewater the basin for maintenance A hydrocyclone and classifier would be used to clean and dewater the grit.

Aerated grit removal may be necessary if septic conditions develop in the force main.

Long detention times are required for grit removal to ensure sufficient preaeration. Detention times are typically between 10 and 15 minutes for average flow conditions, and 3 to 5 minutes for PHWW flow conditions. Design criteria for aerated grit chambers includes adjustable air rates between 3 and 8 cubic feet per minute per foot of tank length.

The aerated grit basin layout consists of a square or rectanuglar tank with a sloped floor to either the center or one side for grit collection. Rectangular tanks have typical width-depth ratio and a length-width ratio of 1.5:1 and 4:1. The type of grit basin layout selected can also affect the type of grit removal mechanism used. The entrance and exit of the basin should be located 90-degrees respectively to each other, and separated as far as possible to prevent short-circuiting.

Benefits of the aerated-type grit removal include:

- Can be used for preaeration if influent wastewater septicity and odor issues exist

Disadvantages include:

- Increased mechanical equipment needs (air blowers, diffusers)
- Potential release of VOCs if present in influent wastewater
- Large basin footprint required

b. Detritor-Type

Detritor-style grit removal uses a square or rectangular basin and evenly distributes flow over it using a series of vanes or gates. This configuration achieves a 1 ft/s velocity and provides sufficient time for grit particles to settle to the bottom of the basin. Settled grit is raked to a sump using scrapers, buckets, plows, or rotating rake mechanisms. Grit is removed by a reciprocating rake mechanism or pump. The grit can then be washed and dewatered in a classifier.

Benefits of detritor-type grit removal include:

- Simple technology with minimal mechanical equipment

Disadvantages include:

- Low removal efficiency
- High organics carryover
- Large basin footprint required

c. Vortex-Type

Vortex-type grit removal induces a rotation into the incoming wastewater using the shape of the basin and a propeller/impeller. This rotational force causes the inorganic particles to be moved towards the outer wall of the chamber where they settle to the bottom of the tank. Settled grit is fluidized (by air or water) and removed by a pump. Multiple pump configurations are availabe, including air-lift, self-priming, and flooded-suction. The grit can then be washed and dewatered in a classifier.

Benefits of the vortex-type grit removal include:

- High grit capture
- Simple technology with minimal moving ` parts
- Multiple manufacturers
- Low headloss

Disadvantages include:

- Additional mechanical equipment required
- Deep basin layout
- Additional structure costs associated with flooded-suction pump option

f. Primary Treatment

Primary treatment is not proposed as part of first phase of the WWTP construction, which will treat flows up to the 2021 Design year. The preliminary and secondary treatment alternatives will be sized to accommodate the loadings for flows up to 2021 MWW. A process flow diagram for primary treatment is provided in Exhibit E-6 in Appendix E.

After ten years when 2021 MWW flows will be reached, additional WWTP capacity must be added to accommodate the 20-year design flow and loadings. In order to minimize the amount of additional capacity needed for the 20-year design loadings, primary clarification will be incorporated ahead of the secondary treatment process at this time. This will reduce the 20-year design loadings to the secondary treatment process and minimize the volume of additional secondary treatment needed for the 20-year design loadings.

Primary clarification reduces settleable solids and BOD_5 loading on downstream treatment processes. Typical solids and BOD_5 reductions are 65% and 30%. TKN is also typically reduced by 10%. Solids, BOD_5, and TKN loading reductions decrease the size of the secondary treatment process upgrades necessary for the 2031 design year loadings. Primary clarification also removes floating material (scum) minimizing operational problems in downstream processes.

Mechanically cleaned circular sedimentation tanks are used for primary clarification. In the circular tank, the flow pattern is radial and wastewater can be introduced in the center or around the periphery of the tank. Center-feed type clarifiers are most commonly used for primary treatment. Wastewater enters a circular feedwell designed to distribute the flow evenly in all directions. The feedwell diameter is typically between 15% and 20% of the total tank diameter. Energy-dissipating inlets (EDI) within the feedwell do not provide much benefit for primary clarification and are not typically used.

Solids are removed from the bottom of the tank by a rotating mechanism that rakes solids to a hopper located near the center of the tank. Scraper mechanisms can use a series of straight blades or spiral-curved blades supported by a truss to push solids to the center hopper. Spiral-curved blades operate at a higher rotational speed and remove solids faster than straight-blade scrapers, allowing for higher solids loading rates to the clarifiers. External pumps (air-diaphragm, rotary lobe, etc.) remove solids from the hopper to solids thickening and/or digestion processes.

A minimum of two tanks are recommended for redundancy. Influent flow is divided equally amongst multiple tanks using a flow splitter structure. Stop plates or slide gates will be used to isolate clarifiers from service for maintenance or low flow situations.

Primary clarification tanks are designed with a maximum surface loading rate (overflow rate) of 1,000 gallons per day per square foot at AWW flows and 1,500 gallons per day per square foot at MWW flows. Detention times are typically between 2.0 to 2.5 hours based on AWW flows.

Based on a design overflow rate of 1,500 gallons per day per square foot three (3) 40-foot diameter clarifiers are required for a 2031 MWW flow of 4.90 mgd. At reduced flow conditions, one or two clarifiers can be taken off-line to maintain the design overflow rate and detention time. Primary sludge will be removed using positive displacement-type pumps (one per clarifier). Primary sludge from each clarifier will be pumped to the digesters for co-digestion with thickened waste activated-sludge.

Scum is removed using a surface skimmer, located above the rake mechanism. The skimmer is supported off the rake mechanism. One or two skimmers can be used per clarifier. Scum is emptied into either a full-radius trough or scum box. A flushing device can be added to wash the scum from the trough or scum box. The scum is pumped to the digesters for treatment.

Based on sampling results, future phosphorus removal will be accomplished with chemical removal in the primary clarifiers. Chemically enhanced primary settling may result in increased removals in the primary clarifiers. Increasing removals in the primary clarifiers may possibly result in additional WWTP organic and hydraulic loadings capacity.

g. Secondary Treatment Alternatives

The secondary treatment process is the major process unit that dictates the quality of an effluent exiting a WWTF. The selection of the secondary treatment process will be affected by the following:

1. Identified stream classification and NPDES permit requirements
2. Site separation requirements and availability
3. Provide capacity for future projected flows and loadings

As described above, it is uncertain if the current population growth trend will remain constant over the design period. Therefore, secondary treatment systems will initially be sized for the 10-year design period (2011-2021) flows and loadings. After ten years, additional capacity can be added to accommodate the 20-year design flow and loadings. In order to minimize the amount of additional capacity needed for the 20-year design loadings, primary clarification will be incorporated ahead of the secondary treatment process. This will reduce the 20-year design loadings to the secondary treatment process. The reduced 20-year design loadings may require additional secondary treatment volume and aeration capacity.

1. Sequencing Batch Reactor (SBR)

A SBR is a secondary treatment process utilizing suspended growth micro-organisms to accomplished the intended treatment. The microbial functions are much the same as a conventional activated-sludge facility except that the aerate/mix/settle is accomplished in one tank instead of multiple tanks. In a typical SBR process, wastewater is treated in batches, with aeration being followed by a period of quiescent settling. The normal cycle is fill, react, settle, idle.

For batch processing, the operating volume is variable. The stages or cycles change according to influent flow variations. Cycle times can be adjusted for peak flows while maintaining designed effluent quality from the SBR system. A process flow diagram of the SBR secondary treatment process is provided in Exhibit E-7 in Appendix E.

ICEAS SBR System

In recent years, the industry has seen a progression toward continuous feed to the SBR reactor. This is advantageous for small plants, since all processes occur in one tank, and also for large plants since the potential for shock loadings to one cell is minimized. The continuous feed process has pre-determined aeration, settling, and decant cycles in a single basin, similar to a batch SBR, but without requiring bypass during settling and decant phases. A pre-react zone in each basin allows the system to handle flow and organic loading flucuations, and acts as a biological selector against the growth of filamentous organisms.

The ABJ™ Intermittent Cycle Extended Aeration System (ICEAS), is a continuous-fed SBR system that combines continuous flow activated-sludge technology with intermittent system operation. The ICEAS process incorporates continuous feed with pre-determined aeration, settling, and decant cycles in a single basin, similar to a true batch process SBR, but without requiring bypass during settling and decant phases. A pre-react zone in each basin allows the system to handle flucuations in flow and organic loading and acts as a biological selector against the growth of filamentous organisms.

Average monthly effluent quality from the ICEAS process for BOD_5, suspended solids, and ammonia-nitrogen would be 10 mg/L, 10 mg/L, and 1 mg/L, respectively. A fine bubble diffused aeration system will provide the required oxygen for BOD_5 and ammonia-nitrogen removal.

Design values for the ICEAS process, at both the 10-year and 20-year design condtions, are listed in Table V-2. At the 20-year design condition, primary clarification would be incorporated ahead of the ICEAS process. Primary clarification would reduce influent BOD_5, TSS, and TKN loadings by 30%, 65%, and 10%, respectively.

Table V-2: Secondary Treatment-ICEAS SBR Alternative Design Values

Parameter	**Unit**	**Design Year**	
		2021	**2031**
Number of Basins		2	4
Operating Volume, each	Gal	723,690	723,690
Operating Volume, total	Gal	1,447,380	2,894,760
Basin Width	Feet	43	43
Basin Length	Feet	125	125
Operating Depth	Feet	18	18
Average Flow	mgd	1.37	2.45
Peak Flow (MWW)	mgd	2.74	4.90
No. of Aeration Blowers		3	5

The basins would be constructed with common walls and operate in parallel. At average flow conditions, a 4-hour cycle with 120 minutes of aeration, 60 minutes of settling, and 60 minutes of decant, would be used. Cycle times would be automatically adjusted by the system at flow conditions above the average flows. For flows below the average flow, one or more of the basins could be removed from service. Table V-3 lists the manufacturer's recommended cycle times for the ICEAS process under various flow conditions.

Table V-3: ICEAS Process Cycle Times

Flow	Aeration	Settle	Decant	Total
Average Flow	120 min	60 min	60 min	4 hour
Greater Than Average Flow	90 min	45 min	45 min	3 hour

The ICEAS process requires aeration blowers and equipment to provide air to the basins. The system utilizes positive displacement type air blowers and fine bubble membrane disc aeration equipment.

A stainless steel effluent decant mechanism is provided in each basin to remove clarified effluent. The design of the decanter provides removal of clarified effluent without entraining settled sludge or removing floating material and scum. The operator can set the depth of the decanter by adjusting the limit switches on the mechanism.

Each basin will be provided with one waste sludge pump. The waste sludge pumps shall be of the submersible non-clog sewage type.

Secondary clarifier basins are not required with this alternative as the ICEAS process basins also act as the clarifiers during the settling and decant phases.

Peak flow treatment will be accomplished using the ICEAS process as described above. The construction of a peak flow clarifier will not be necessary with this alternative.

Several advantages and disadvantages associated with this alternative are listed below:

Advantages:

- Design incorporates a selector to prevent growth of filamentous organisms
- Continuous flow operation unlike conventional SBR
- Operational flexibility to optimize treatment efficiency
- Ability to handle fluctuation in flows and loads with minimal decrease in treatment efficiency
- Generates less waste activated-sludge than a conventional activated-sludge system
- Eliminates the need for secondary clarifiers and return sludge pumping facilities

Disadvantages:

- Proprietary technology
- May require greater degree of operator control than a conventional activated-sludge system
- Additional operating costs required for aeration equipment
- Scum handling may be required
- Moving parts on decanter may be subject to freezing or malfunction
- Digestion facilities are required to meet Environmental Protection Agency (EPA) 503 regulations for land application of biosolids

2. Conventional Activated-Sludge

The conventional activated-sludge process uses aeration tanks followed by final clarifiers to aerate, mix and settle the wastewater for further treatment. A process flow diagram of the conventional activated-sludge secondary treatment process is provided in Exhibit E-8 in Appendix E.

a. Aeration Tanks

In conventional activated-sludge aeration tanks maintain a population of biological organisms. The activated-sludge process uses a suspension of flocculent microorganisms composed of bacteria, fungi, protozoa, and rotifers to remove biologically degradable organic compounds (e.g. BOD_5) from the wastewater. The organisms are then settled in secondary clarifiers and returned to the aeration tank to provide the concentration of organisms targeted. Many different activated-sludge configurations can be used to accomplish treatment, including complete mix aeration and plug flow tapered aeration. Each configuration has its targeted application, advantages, and disadvantages. The activated-sludge configuration chosen for Harrisburg is plug flow tapered aeration. Aeration basins equipped with diffused aeration would be sized to handle the MWW design flow.

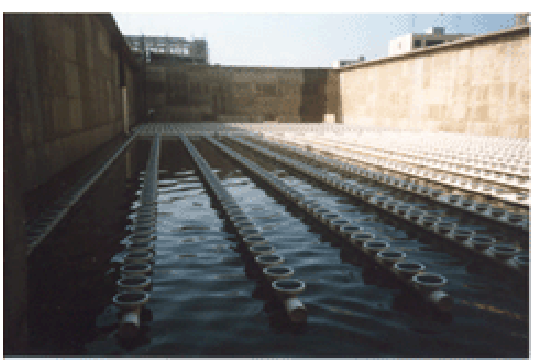
Plug flow activated sludge system

The plug flow tapered aeration activated-sludge process is one of the most commonly used biological processes for treatment of municipal wastewater. With plug flow, the aeration system is designed to match the oxygen demand along the length of the tank by tapering the aeration rates. Higher rates are applied at the beginning of the tank and decrease toward the end of the tank.

Aeration tanks will be constructed for the removal of carbonaceous BOD_5 and ammonia. Longer solids retention times (SRTs) are needed in the aeration tanks to establish the desired microorganisms to remove ammonia. SRT is based on the volume of aeration provided, and is the amount of time that a microorganism remains in the system to grow and thrive. The relative age corresponds to the level of treatment the organism can accomplish.

Microorganism growth is dependent on many factors including, temperature, pH, dissolved oxygen, etc. At warmer temperatures, organisms will grow faster than at lower temperatures. For example, an organism grown at 20 degrees Celsius (C) for 5 days may be able to accomplish the same level of treatment as an organism aged for 15 days at 10 degrees C. A 12-day SRT will be used at Harrisburg's WWTP to achieve nitrification at future design flows and loads for a design temperature of 10 degrees C. Assuming a 12-day SRT and a mixed liquor suspended solids (MLSS) concentration between 3,500 and 4,000 mg/l, approximately 1.2 million gallons of aeration capacity is required.

A selector design can be incorporated into the aeration basin design to reduce filamentous organism growth. Multiple selectors would be used at the influent end of each aeration tank to provide filamentous control and increase the settling properties of the activated-sludge. Baffles would be added to the first quarter of each basin to construct the selectors. Either an anoxic or aerobic selector will be used to provide well settling mixed liquor. Mixing would be provided for the anoxic selector, and air would be supplied for the aerobic selector. The details will be evaluated in the preliminary design phase.



Fine bubble membrane diffusers

Fine bubble membrane diffusers are recommended for the selector zones and main aeration zones of the tank due to high transfer efficiency and advances in technology allowing for longer service life.

Oxygen would be supplied to the aeration (Ox-1) portion of the tanks based on 1.1 lb oxygen/lb BOD_5 removed and 4.6 lb oxygen/lb TKN removed. The actual oxygen requirement (AOR), determined with projected future flows and loadings, is shown in Table V-4. Using an average alpha value of 0.5, DO of 2.0 mg/l, and an oxygen transfer efficiency of 25%, the air supply required for the future flows and loadings is shown in Table V-4. New positive displacement (PD) blowers would provide aeration. At the time of the 2021 upgrade, the reuse of the existing PD blowers would be evaluated. Water depth in the basin will be approximately 15 feet deep to allow for PD blowers. To provide for redundancy, two blowers will be sized to supply the required air demand with one additional blower for standby. The blowers will be housed in an enclosure or other structure. VFDs will be used to control the blowers based on oxygen needs to the system.

Aeration piping from the blowers to the basin will be either light wall steel or ductile iron pipe (DIP) outside the tank, and stainless steel within the tank.

An aeration flow splitter will be used to equally split flow to the aeration tanks. Stop plates or slide gates will be used to isolate tanks from service. The flow splitter will also receive the return sludge pumped from the secondary clarifiers.

For each mg/l of ammonia removed, approximately 7.1 mg/l of alkalinity are needed. Alkalinity in the plant influent is assumed to be sufficient based on sampling results.

b. Clarifiers

Clarifiers are required with activated-sludge process to settle the microorganisms from the mixed liquor exiting the aeration tanks. A portion of the settled mixed liquor is then returned back to the aeration tanks to maintain a targeted ratio. The sludge flow returned is termed return activated-sludge (RAS).

Secondary clarifier sizing is based on the solids loading rate (SLR) and overflow rate. Secondary clarifiers sizing for the future design conditions is shown in Table V-4. Since an equalization basin will be used at the lift station ahead of the WWTP influent, the secondary clarifiers will be sized to handle the MWW flow with the largest unit out of service while maintaining the surface overflow rate less than 1,200 gpd/sf.

Secondary Clarifier

The new secondary clarifiers would utilize an optimization package that incorporates center-feed technology and peripheral draw. The clarifier optimization package includes a center column, energy dissipating inlet (EDI), flocculating feed well (FFW), spiral scrapers, scum removal system, current baffling, and a sludge drum. The center column, EDI, and FFW are designed to minimize floc breakup and optimize settling performance. The current baffling is designed to minimize solids scouring during high flow periods. The spiral scrapers effectively and efficiently transport sludge to the sludge hopper for withdrawal.

A flow splitter will be used to divert MLSS equally to the secondary clarifiers. Stop plates or slide gates will be used to isolate clarifiers from service for maintenance or low flow situations.

A structure will be required to pump the sludge from the bottom of the secondary clarifiers to the influent aeration flow splitter. The RAS pumping facilities will have a recycle pumping capacity of up to 100% of the average return sludge flow. The design pumping rate will be approximately 625 gpm, firm capacity. The structure will be configured with sluice gates on the pipes from each clarifier sludge hopper. The sluice gates will modulate the proportioning of the sludge from each clarifier into the wetwell. The RAS pumps will pump from the wetwell back to the aeration tank flow splitter. Locations shall be provided for additional future RAS pumps. A waste activated-sludge (WAS) pump will remove solids from the system to a solids processing unit.

Design values for the conventional activated-sludge process, at both the 10-year and 20-year design condtions, are listed in Table V-4. For the 20-year design condition, primary clarification would be incorporated ahead of the activated-sludge process. Primary clarification would reduce influent BOD_5, TSS, and TKN loadings by 30%, 65%, and 10%, respectively.

Table V-4: Secondary Treatment-Conventional Activated-Sludge Alternative Design Values

			Design Year	
Parameter		**Unit**	**2021**	**2031**
Activated-Sludge System				
	Number of Basins		3	3
	Operating Volume, each	Gal	398,933	398,933
	Operating Volume, total	Gal	1,196,800	1,196,800
	Basin Width, each	Feet	40	40
	Basin Length	Feet	90	90
	Operating Depth	Feet	15	15
	Average Flow	mgd	1.37	2.45
	Peak Flow	mgd	2.74	4.90
	Solids Retention Time	days	12	12
	Design MLSS Concentration	mg/L	3,690	4,000
	No. of Aeration Blowers		3	5
	Actual Oxygen Requirement (AOR)	ppd	6,802	8,807
	Air Requirement	SCFM	3,000	3,900
Secondary Clarifiers				
	Number of Basins		2	3
	Basin Diameter	Feet	65	65
	Operating Depth	feet	12	12
	Average Flow	mgd	1.37	2.45
	Peak Flow	mgd	2.74	4.90
	Overflow Rate @ Peak Flow	Gpd/ft^2	581	632
	Solids Loading Rate @ Peak Flow	ppd/ft^2	29.2	35.2

Several advantages and disadvantages associated with the conventional activated-sludge alternative are listed below:

Advantages:

- Non-proprietary technology
- Thousands of installations with proven technology
- Well understood process that is simple to operate
- Process is flexible and will accommodate future expansion
- Risk of downtime spread to multiple process tanks

Disadvantages:

- Filamentous organism growth may occur without incorporating selector zones
- May encounter bulking and rising sludge in the secondary clarifiers

- Scum handling may be required in the aeration tank
- Additional operating costs required for RAS equipment
- Additional mechanical maintenance
- Separate solids treatment required
- Added capital costs to construct the separate structures required for treatment and clarification

3. Membrane Bioreactor (MBR)

The MBR system uses a combination of a suspended growth activated-sludge system and an immersed, low pressure ultrafiltration membrane system. The suspended growth activated-sludge system is a conventional activated-sludge system as described above. The ultrafiltration membrane system is located downstream of the activated-sludge system and eliminates the need for secondary clarifiers. Since sludge settling is not required, the activated-sludge process can be operated at MLSS concentrations between 10,000 to 15,000 mg/L. This is three to five times higher than concentrations in conventional activated-sludge systems. A smaller operating volume for the activated-sludge system is required due to the increased MLSS concentrations. A process flow diagram of the MBR secondary treatment process is provided in Exhibit E-9 in Appendix E.

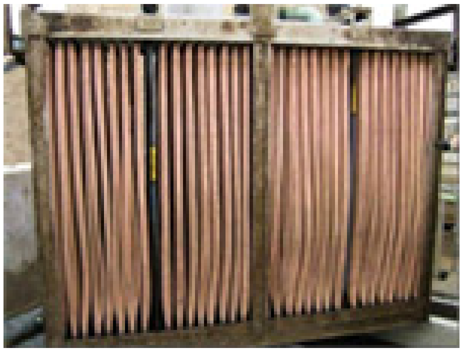
MBR system

A plug flow tapered aeration activated-sludge process is used for the removal of carbonaceous BOD_5 and ammonia. Aeration basins equipped with diffused aeration would be sized to handle the MWW design flow. Flows above the MWW design would be diverted and held in an

equalization basin located adjacent to the lift station pumping to the WWTP.

An aeration flow splitter will be used to equally split flow to the aeration tanks. Stop plates or slide gates will be used to isolate tanks from service. The flow splitter will also receive the return sludge pumped from the membrane basins.

The MBR system consists of bundled hollow-fiber membranes modules, with multiple modules per cassette. Each cassette is connected to a permeate header. A low-pressure vacuum is applied to the membrane system to draw permeate through the membrane, separating the MLSS from effluent water. Periodic cleaning of the membrane surface is provided by reversing the permeate flow and initiating a simultaneous air scour to backflush solids that have accumulated in the membrane pores. Chemical cleaning can also be used to restore membrane permeability if necessary.



MBR membrane cassette

The MBR system cannot tolerate rapid changes in flow. Any flow conditions above the maximum daily flow must be equalized prior to the membrane system. As a result, the size of the equalization basin ahead of the lift station discussed in Section V.A.4.b needs to be increased for the MBR option. The equalization basin would provide 14 days of storage at 2031 flows based on AWW conditions. This is a much larger equalization basin compared to the one proposed for pumping to a Harrisburg WWTP.

The equalization basin will lessen the peak flows, reduce the needed pumping capacity in the lift station, and reduce the overall size of the lift station, and reduces the needed WWTP capacity. It offers the WWTP operational flexibility should the need arise to reduce or temporarily eliminate flow from Harrisburg. The larger equalization basin also has the potential to create several problems. Odors will likely develop from storing the large amount of raw wastewater. Either surface aerators or a fine bubble aeration system will need to be installed in a portion of the basin to reduce odor problems.

Design parameters for the equalization basin are provided in Table V-5.

Table V-5: Equalization Basin Design Parameters for MBR WWTP

Parameter	**Value**
Needed Volume (gallons)	34,328,000
Approx. Bottom Length (ft)	652
Approx. Bottom Width (ft)	402
Approx. Top Length (ft)	700
Approx. Top Width (ft)	450
Approx. Usable Depth (ft)	8
Approx. Total Depth (ft)	11
Slope	3:1
Number of Basins Required	2
Land Requirement (acres)	30

Due to the allowable flux through the membranes, the peak firm capacity of the membrane module system is 1.37 mgd. The peak flow capacity of each membrane train is 0.685 mgd.

The membrane system effluent quality is far better than that achieved by conventional secondary clarification. The physical separation of the mixed liquor using the membranes is capable of achieving an effluent with BOD_5 and TSS concentrations of less than 3 mg/L.

Membrane cassettes are placed in either stainless steel or concrete tanks. Recirculation pumps are provided in each membrane module tank to return MLSS flow to the activated-sludge system. Sludge wasting is accomplished by diverting flow from the MLSS return line or wasting directly from the activated-sludge system.

Design values for the activated-sludge and membrane systems, at both the 10-year and 20-year design condtions, are listed in Table V-6. Once the 10-year design condition is reached, primary clarification would be incorporated ahead of the activated-sludge process. Primary clarification would reduce influent BOD_5, TSS, and TKN loadings by 30-, 65-, and 10-percent, respectively.

Table V-6: Secondary Treatment-Membrane Bioreactor Alternative Design Values

Parameter		Unit	Design Year	
			2021	2031
Activated-Sludge System				
	Number of Basins		2	3
	Operating Volume, each	Gal	265,963	266,039
	Operating Volume, total	Gal	531,925	798,116
	Basin Width	Feet	30	30
	Basin Length	Feet	80	80
	Operating Depth	Feet	15	15
	Average Flow	mgd	1.37	2.45
	Peak Flow	mgd	2.05	4.12
	Solids Retention Time	days	12	12
	Design MLSS Concentration	mg/L	8,000	6,670
	No. of Aeration Blowers		3 (75 hp)	5 (75 hp)
	Actual Oxygen Requirement (AOR)	ppd	6,802	8,807
	Air Requirement	SCFM	3,000	3,900
Membrane System				
	Number of Cassettes		12	24
	Membrane Operating Volume, total	Gal	65,371	130,742
	Average Flow	mgd	1.37	2.74
	Max Daily Flow	mgd	2.05	4.12

Several advantages and disadvantages associated with the MBR alternative are listed below:

Advantages:

- Produces a high quality effluent beneficial for reuse
- Eliminates need for secondary clarifiers
- Higher allowable MLSS concentration reduces required volume of activated-sludge system
- Process is flexible and will accommodate future expansion
- Individual membrane cassettes can be taken off line for maintenance and cleaning

Disadvantages:

- Produces a high quality effluent at a higher capital cost. Since Harrisburg does not intend to reuse the water at this time, it may be an unecessary expense.
- Proprietary technology
- Limited number of membrane manufacturers
- Limited U.S. installations
- Additional operating costs required for membrane permeate vacuum and backpulse cleaning equipment
- Separate solids treatment required
- Added capital costs for separate structures required for activated-sludge and membrane systems

h. Disinfection Alternatives

Three disinfection alternatives were developed and analyzed for the WWTP. The role of disinfection in wastewater treatment is to kill bacteria remaining after other treatment processes. The three alternatives investigated include a chlorine gas system, a liquid chlorine treatment process, and an ultraviolet disinfection system.

1. Chlorine Gas System

Chlorine gas is an effective disinfectant for wastewater treatment. However, this alternative poses many disadvantages including: operator and public safety issues, extensive equipment maintenance needs, and the need for dechlorination of wastewater effluent prior to discharge. The Clean Air Act Amendments regarding chlorine storage require a facility storing more than 2,500 lbs of chlorine to have a Risk Management Plan (RMP). The RMP must include procedures for informing public and emergency response agencies after accidental release. It also must include procedures for the use of emergency response equipment, including its inspection, testing, and maintenance. Finally, the RMP must document the first aid and emergency medical treatment necessary to treat accidental exposure to each regulated substance at the facility.

Harrisburg's WWTP would need more than 2,500 lbs of chlorine to accommodate its treatment capacity. Because of site separation issues from nearby residential areas and the potential hazards associated with this alternative it will not be given further consideration.

2. Liquid Chlorine Addition

Liquid chlorine, generated from sodium hypochlorite, can also be used for disinfection. Sodium hypochlorite is gaining popularity as a disinfectant, because it is less of a hazard compared to gas chlorination. Sodium hypochlorite is typically delivered in bulk or can be generated on-site. On-site systems are suited better for very small systems, because they can only generate a solution that is 0.8-percent sodium hypochlorite.

Sodium hypochlorite loses its disinfecting strength when stored in bulk, and subject to heat and light. A 17-percent solution stored at 80 degrees Fahrenheit (F) will lose 10-percent of its strength in 10 days, 20-percent in 25 days, and 30-percent in 43 days. Typical solution concentration delivered is approximately 12-percent. Chlorine dosages based on gas chlorination are used to determine sodium hypochlorite needs, since one (1) gallon of 12.5-percent sodium hypochlorite contains about 1.25 pounds of chlorine. Typically, a 30-day supply is stored on-site. Required storage volume is based on the design flow through the system. Storage of hypochlorite must be in sturdy, non-metallic containers (typically polyethylene construction) with secure tank tops, pressure relief valves, and overflow piping.

A large amount of on-site storage would be required to meet the 30-day supply for Harrisburg's WWTP. An on-site generation system would be recommended given the size of the system that would be required for the design flow.

Additional needs for the disinfection process include feed pumps, mixers (static or mechanical), and a contact basin sized to provide the necessary contact period. A structure to house the liquid hypochlorite equipment, storage containers, and related feed equipment would also need to be built.

A containment wall would need to be constructed in the storage building to contain spills from the storage tank. The containment shall provide 110-percent of the storage volume capacity of the chemical tank to allow freeboard. The containment wall shall be no higher than 3'-11" tall to avoid confined space entry procedures.

Chlorine is toxic to aquatic species and must be removed from the plant effluent before being discharged into the receiving water body. Dechlorination of liquid chlorine by sodium bisulfite is proposed and would be used to reduce the residual to zero. Again, 30-day storage capacity, feed pumps, and containment structures would be required for the dechlorination system.

The primary advantages of sodium hypochlorite include:

- Reduced potential health effects

Disadvantages of using sodium hypochlorite include:

- Increased chemical costs
- Additional structure needs
- High O&M costs relative to other disinfection systems
- Need for dechlorination
- Relatively short shelf life

Because of design flow, facilities and equipment requirements, operation and maintenance needs, site separation issues from nearby residential areas, and the potential hazards associated with this alternative, it will not be given further consideration.

3. Ultraviolet (UV) Disinfection

This alternative evaluates the use of ultraviolet radiation for disinfection of clarified, or MBR effluent. UV radiation does not inactivate microorganisms by chemical interaction. Instead, UV inactivates organisms with light absorption, which causes a photochemical reaction that alters the nucleic acids (DNA and RNA) essential for cell function. *Giardia* and *Cryptosporidium* are more sensitive to UV than bacteria, and viruses are more resistant than bacteria. UV radiation quickly dissipates into water to be absorbed or reflected off material within the water. The UV disinfection process produces negligible disinfection by-products.

Horizontal UV system in a concrete channel

UV dose is defined using IT (intensity and time) values similar to CT (concentration and time) values using chlorine disinfection. UV dose or IT is a product of UV light intensity and exposure time in seconds, stated in units of milliWatt second per square centimeter ($mW \cdot s/cm^2$) or milliJoule per square centimeter (mJ/cm^2).

Recent advances in UV technology have lead to more effective lamp designs and space saving configurations including low-pressure, medium-pressure, and pulsed UV irradiation in channel mounting and pipe mounting configurations. Recent research indicates that UV doses ranging from less than 10 mJ/cm^2 to as high as 40 mJ/cm^2 would be required to achieve 4-log inactivation of *Cryptosporidium*, *Giardia,* and viruses.

Advantages of UV disinfection include:

- No chlorine residual
- Non-toxic to aquatic species
- No chemical safety handling issues

Disadvantages of UV disinfection include:

- Maintenance needed for cleaning bulbs
- High capital cost
- Decreased effectiveness on effluents with high suspended solids concentrations
- Decreased effectiveness with iron salt chemical feed (P removal)
- Algae

The UV system would be located after the secondary treatment process prior to discharge from the WWTP. The UV system would be sized for either the MWW flow, the peak SBR decant rate, or the MBR effluent flow rate depending on the recommended secondary treatment alternative.

Vertical UV system in a concrete channel

Multiple system configurations are available to treat the projected peak flow. These include either a package system with UV modules in fabricated stainless steel channel, or a manufacturer supplied UV modules placed in a concrete channel provided by the Owner. The system layout will be further evaluated during schematic design, including horizontal or vertical UV bulb orientation. Controls and the power distribution center can be placed on a nearby slab-on–grade, or remotely in a separate building. The entire system can be placed indoors if desired. Additionally, automatic cleaning systems can be provided to minimize the amount of manual cleaning of the bulbs needed by the operator. Mechanical and chemical cleaning systems are available; however, the chemical cleaning system is proprietary and is available through only one UV system manufacturer.

i. Solids Disposal for Mechanical Options

Solids treatment systems will initially be sized for the 10-year design period (2011-2021) flows and loadings. After ten years, additional capacity can be added to accommodate the 20-year design flow and loadings. The flows and loadings during each of these design periods will correspond to the waste solids production from the primary and secondary treatment processes described previously.

Either aerobic or anaerobic digestion is an option for treatment of secondary treatment waste solids. Combined anaerobic digestion of primary and secondary sludge would be the best option if primary treatment was provided. Aerobic digestion of secondary sludge would be the best option for solids treatment if primary treatment is not provided.

1. Waste Activated-Sludge (WAS) Thickening

During the 2011-2021 design period, thickened WAS will not be needed ahead of aerobic digestion. During the 2021-2031 design period, thickened WAS will be blended with primary sludge prior to digestion.

Thickening of the WAS from the secondary treatment process will be needed to reduce the volume of the wasted sludge prior to digestion. This will also reduce the required digester volume. A WAS holding tank shall be provided ahead of the thickening process to allow for continuous wasting from the new secondary process. From the holding tank, the WAS will be pumped to the thickening process. A process flow diagram of the WAS thickening process is provided in Exhibit E-10 in Appendix E.

The thickening process should reduce the WAS volume of the secondary process waste stream from approximately 0.8-1.5% solids to 4-6% solids. During the 2021-2031 design period, the thickened WAS will combine with the primary sludge in a sludge blending tank. The resulting WAS and primary sludge mix is estimated to have a 4%-5% solids concentration. Since the solids will be thoroughly mixed in the digester, it is not critical to have an exact homogenous sludge blend.

Several technologies are available to thicken the sludge to meet the volume reduction goal. A tabulation of technologies and the typical thickened solids percentages expected with that technology is provided in Table V-7.

Table V-7: WAS Thickening Technologies

Technology	**Expected Thickened Solids Concentration**
Rotary Drum Thickener	5-8%
Gravity Thickener	2%
Dissolved Air Flotation	3-5%
Gravity Belt Thickener	5-7%
Centrifuge	<8%

Rotary Drum Thickener

Additional evaluation will be completed during preliminary design; but for this evaluation, a Rotary Drum Thickener (RDT) has been selected due to the following advantages:

- Technology can easily meet the 4%-6% solids goal
- Expected polymer use is small (12 lbs/dry ton)
- Cost for RDT is competitive with other technologies and between manufacturers
- Low energy use
- Easy to operate and provide normal maintenance with City staff
- Can be a redundant backup to dewatering unit used for digested sludge

Redundancy for the RDT is provided through the digested solids dewatering equipment, since the dewatering process will not be a 5-day/week operation. This process is described further in the next section of this report.

Thickener filtrate will be gradually returned to the aeration flow splitter or ahead of the primary clarifiers. The need for a filtrate holding tank and the design pumping capacity will be evaluated during schematic design.

2. Aerobic Digestion

Since primary treatment will not be provided for the 2011-2021 design period, aerobic digestion is recommended. Aerobic digestion will be used to treat the solids to meet requirements of the EPA 503 regulations. Aerobic digestion will produce a Class B land applicable product. A process flow diagram of the aerobic digestion process is provided in Exhibit E-11 in Appendix E.

The EPA 503 Regulations require that 60 days or 40 days of detention time be provided at 15 or 20 degrees C. Design temperature will be 15 degrees C. If the aerobic digesters are set up to operate in series, the EPA will allow a credit of 30% of the required detention time. The required detention time of the sludge prior to ultimate disposal will then be 42 days. Based on the projected WAS production from the secondary treatment process, two 50-foot diameter digesters operated in series are needed to meet the required detention time. Each digester will have an operating depth of 26 feet. The volume required for the each stage is 375,000 gallons.

Aeration to the aerobic digesters will be provided by 3 new blowers (2 duty, 1 standby) at 30 scfm/1000 ft^3. Each blower shall have a capacity of 1,550 SCFM, operating at approximately 11.5 psig. These blowers will be mounted either indoors or outdoors on a concrete pad with sound-reducing enclosures and will be VFD controlled. Diffusers in the aerobic digester will be stainless steel band-type coarse bubble diffusers.

3. Anaerobic Digestion

Anaerobic Digester

For the 2021-2031 design period, primary clarifiers will be added ahead of the secondary treatment process. Therefore, the aerobic digesters will be converted to anaerobic digesters to treat the combined primary and secondary sludge. The existing aerobic digesters will be converted to a 2-stage anaerobic system operated in series. The first stage will be retrofitted with a fixed cover and mixing system. The second stage will be retrofitted with a floating, gas holder cover. The minimum required solids retention time for high-rate digestion is 15 days to meet EPA 503 regulations for Class B sludge. The design operating temperature will be 35 degrees C. Feed sludge will be heated via a boiler and heat exchanger system. The boiler can be fueled by natural gas, biogas, or both. A gas handling system will be required for the biogas produced in the anaerobic digester. The boiler, heat exchanger, and gas handling system will be further evaluated during preliminary design. A process flow diagram of the anaerobic digestion process is provided in Exhibit E-12 in Appendix E.

Feed sludge concentrations of approximately 4%-5% are needed to reuse the existing digester volume without adding capacity for the increased sludge production due to the higher influent loading conditions. The increase in feed sludge concentration can be accomplished by increasing polymer dosage at the WAS thickening process.

j. Dewatering

Dewatering of digested sludge reduces the volume of sludge storage required before ultimate disposal. This process will be used with either aerobic or anaerobic digestion. As with sludge thickening, several technologies are available for dewatering, including centrifuges, belt filter presses, recessed plate presses, drying beds, and lagoons. Dewatering will be evaluated using a belt filter press (BFP) due to low capital costs, low energy requirements, and equipment availability.

A 2.0-meter width BFP is recommended. For the 2021 design year, the BFP will be sized to operate 2 days per week, six (6) hours per day with a projected sludge feed to the BFP of approximately 23,000 pounds per week. For the 2031 design year, the BFP runtime would be increased to operate 2 days per week, eight (8) hours per day with a projected sludge feed to the BFP of approximately 29,000 pounds per week. For aerobically digested sludge, the target dewatered solids content will be 14%-15%. For anaerobically digested sludge, the target dewatered solids content will be 18%-20%.

The BFP and polymer feed equipment shall be located in an enclosed structure. The BFP polymer feed system will be separate from the thickening polymer feed system. The BFP facility structure will consist of an enclosed pre-engineered metal or concrete block building on a concrete foundation and slab. The BFP and polymer feed system will be located in an enclosed section of the building.

The dewatered-sludge storage area should be covered to limit exposure to wet weather. A concrete basin storage area is proposed adjacent to the enclosed portion of the building with a canopy roof extended over the storage area. The dewatered area shall hold 180 days of dewatered-sludge. One end of the storage basin will remain open for access and sludge load-out.

k. Disposal Options

Solids disposal is necessary to remove the dewatered digested sludge from the facility. Multiple options exist for the disposal of dewatered digested sludge (biosolids).

Digested sewage sludge can be applied to nearby farmland, packaged and distributed to consumers as fertilizer, incinerated, or transferred to a landfill. Because of cost and the availability of farmland, land application of dewatered digested sludge is recommended for ultimate disposal.

1. Land Application

Land application involves the spreading, spraying, injecting, or incorporating biosolids onto or below the surface of the land to take advantage of its soil enhancing qualities. This process improves the structure of the soil and supplies nutrients to crops grown in the soil.

Land application procedures must follow requirements of the EPA 503 Regulations. The EPA regulates biosolids disposal on three factors: pollutants, pathogens, and attractiveness to vectors. Pollutants monitored in biosolids are harmful metals; pathogens include bacteria, viruses, and parasites; the biosolids attractiveness to vectors measures how rodents and flies are attracted to the dewatered-sludge.

Generally, the monitoring of biosolids quality is the responsibility of the producer (City). If the biosolids meet the EPA "Exceptional Quality" standards, the land-applier has no additional EPA requirements to meet. If the biosolids do not meet these standards, additional requirements are placed on the digested sludge and the application site to ensure health protection.

Additionally, a land application management plan must be developed, detailing biosolids pollutant concentrations, vector attraction reduction, and proposed application rates to meet contaminant levels outlined in the 503 Regulations. The land applier must notify the state permitting authority of the intent to apply biosolids to a particular site prior to land application. The management plan must be kept current and updated throughout the land application period.

Typical regional costs associated with the land application of biosolids sludge are shown in Table V-8. Costs include a per-gallon rate for application, a per-gallon rate for transportation of sludge, and a per-trip mobilization fee for travel and equipment costs.

Table V-8: Land Application Unit Costs

Parameter	**Cost**
Land Application	2 ¢/gal
Transportation Fee (up to 20 miles)	2.5 ¢/gal
Mobilization Charge	$1000 / trip

Biosolids contractors can also provide management services. With these services, a contractor will oversee the transportation, land application, paper work, soil testing, and record keeping of the biosolids in accordance with the requirements of the approved management plan.

Annual biosolids production for land application is estimated at 1,000,000 gallons. Harrisburg's WWTP will provide 180 days of digested, dewatered-sludge storage; therefore, land application will be required two times per year (spring and fall). Annual costs for land application based on the estimated biosolids production are shown in Table V-9:

Table V-9: Annual Biosolids Land Application Cost
(500,000 gal per application, twice per year)

Parameter	Cost	Cost/year
Land Application	$10,000	$20,000
Biosolids Transportation	$12,500	$25,000
Mobilization Fee	$1,000	$2,000
Management Services	$5,000	$5,000
Total		$52,000

These costs were used to develop the O&M costs for biosolids disposal. The present worth cost of direct land application of digested solids without thickening exceeds the total present worth cost of thickening plus land application. Therefore, thickening ahead of land application is recommended.

5. Outfall

Gravity outfall piping will be required to convey the treated effluent from the WWTP to the Big Sioux River. At this time, the location of the WWTP and outfall are unknown, so 2,000 feet of 30-inch outfall piping was assumed at a slope of 4%. Under these conditions, the outfall would have a capacity of 3,690 gpm. The size, length, and slope of the outfall will be finalized when a WWTP site is selected.

6. Regionalization

Several options for regionalization were considered including:

- Pumping wastewater to the City of Sioux Falls for treatment
- Building a larger WWTP than needed and selling excess capacity to the City of Sioux Falls or others
- Sioux Falls relocating the proposed WWTP on the south side of the City further south to accommodate Harrisburg
- Purchasing a portion of the proposed Sioux Falls WWTP located on the south side of the City
- Construction of a regional WWTP with the City of Tea

a. Pump to the City of Sioux Falls for Treatment

Harrisburg could pump their wastewater to the City of Sioux Falls for treatment. This would require Harrisburg to construct a small section of gravity sewer piping to a wet well and can-style lift-station at the south side of the existing evaporation ponds.

Initially, the wastewater would be pumped to Sioux Falls' Lift Station #240 located near 57th Street and the Big Sioux River.

This lift station would convey wastewater to Sioux Falls' current WWTP on the north side of the City.

The City of Sioux Falls plans to construct a new MBR WWTP in 2014 or 2016, directly across the river from Lift Station #240. At the time the new WWTP is constructed, flows from Harrisburg would be directed to the head of this WWTP. Exhibit E-13 in Appendix E provides a proposed layout for the gravity interceptor, lift station and force main. It also identifies Lift Station #240.

The Sioux Falls MBR plant will not tolerate rapid changes to influent flows. As a result, an equalization basin would be needed ahead of the lift station. In addition to lessening the peak flows sent to Sioux Falls for treatment, the equalization basin will also reduce the needed pumping capacity and the overall size of the lift station. Finally, it offers Sioux Falls operational flexibility should the need arise to reduce or temporarily eliminate flow from Harrisburg.

The Facility Plan evaluates two options for the equalization basin. The City greatly hoped to relocate the equalization ponds outside City limits and requested an option be considered to construct new ponds adjacent to a new lift station site outside City limits. This option proved to provide little savings since two lift stations were required. The first lift station would pump flow from the area of the current evaporation pond inlet to the new equalization basin. The second lift station would pump from the equalization basin to Lift Station #240.

As a result, a second option was considered that used the existing evaporation ponds to provide equalization and pretreatment. This option required only one lift station and no land acquisition.

The specific size, flow rate, and operating head conditions for the lift station pumps will be evaluated during schematic design once a force main route is finalized. Preliminary calculations indicate that the pumps would be sized for 75% of MWW, or 1,170 gpm at 145 feet total dynamic head (TDH) for the 2019 Design year, and 2,110 gpm at 325 feet TDH for the 2029 Design year.

Due to the high head conditions expected, the lift station may be designed as a duplex pump station, or be sized for two sets of pumps in parallel for a total of four (4) pumps. Both pump trains will be sized to handle 75% of the MWW flow independently from the other. Therefore, one (1) train will be for duty operation, while the other train will be used for standby operation. Each train will be alternated upon pump startup to decrease pump wear. Once 75% of the 2019 MWW flow is reached, the pumps will be replaced with new pumps capable of handling 75% of the 2029 MWW flow. It is anticipated that VFD's will be used to match the pumping rate with the influent flow rate, reduce energy costs, extend motor life, reduce the required starting current, reduce maintenance costs, and to help prevent the wastewater from becoming septic.

The wetwell will be sized to minimize pump start/stop cycles as per pump manufacturer recommendations and according to SD DENR requirements. The use of a “self-cleaning” wetwell design will be investigated during schematic design to minimize maintenance and cleaning needs, eliminate odors, and reduce wetwell size.

The wetwell/drywell and submersible design were considered for lift station layout. The wetwell/drywell configuration will be selected due to the ease of performing routine maintenance on pumps and valves. This layout also makes it easier for early detection of small problems, before they become large problems. Finally, it allows several pump drive configurations to be used and permits a smaller wetwell footprint.

Approximately 56,000-feet of 16-inch diameter force main would be required to transport from the Harrisburg’s proposed lift station to Lift Station #240 in Sioux Falls. Due to the anticipated high discharge pressure from the pumps, a portion of the force main may have to be high pressure DIP until the pressures drop to allow for the safe use of PVC.

At the time Sioux Falls constructs the MBR WWTP, the force main would need to be extended approximately 2,000 feet from Lift Station #240 to the WWTP on the other side of the river. The lift station pumps would need to be selected with the capability to address the additional head requirements. It would also require a river crossing and rock removal for installation of the force main. The capital costs for the work have been included in the cost of the force main, however several assumptions had to be made since the exact placement of the WWTP in not known at this time.

b. Building a Larger WWTP Than Needed and Selling Excess Capacity to the City of Sioux Falls

The option of Harrisburg building a larger WWTP than needed near the Big Sioux River and selling excess capacity to Sioux Falls was discussed with Sioux Falls City Staff on several occasions. Sioux Falls has stated that they are not interested in this option. In addition, it would require Harrisburg to take on additional debt upfront. They do not have the debt capacity for this option at this time. As a result, this option will not be evaluated further.

c. Sioux Falls Relocating Their Proposed WWTP on the South Side of the City Further South to Accommodate Harrisburg

This option was discussed with the City of Sioux Falls on several occasions. They have no intention at this time of relocating the plant further south. They have preliminary land options on property near Lift Station #240 and plan to convert the lift station to pump solids to the north plant for treatment. This will allow the south plant to treat only liquid waste and reduce their operational costs. If the WWTP was located further south, the City of Sioux Falls would have to construct another lift station and force main to transfer solids to Lift Station 240. They do not want these additional costs. As a result, this option will not be evaluated further.

d. Purchasing a Portion of the Proposed Sioux Falls WWTP Located on the South Side of the City
This option was also discussed with the City of Sioux Falls on several occasions. Sioux Falls would prefer to retain ownership of the entire WWTP instead of selling a treatment train to the City of Harrisburg. It is actually advantageous to Harrisburg not to purchase a portion of the plant to reduce their upfront capital costs. Instead, Sioux Falls funds the capital construction costs for their treatment needs and Harrisburg payments increase as their flows increase. As a result, this option will not be further evaluated.

e. Construction of a Regional WWTP With the City of Tea
A regional WWTP shared between the City of Tea and Harrisburg was discussed briefly with the City of Tea. Tea recently completed improvements to their lagoons, including adding aeration. These improvements provided them with several years of available capacity. They are also far enough from Lake Alvin to discharge into Ninemile Creek. As a result, they are not interested in a Regional WWTP with the City of Harrisburg; therefore, this option will not be evaluated further.

B. EVALUATION OF MONETARY COSTS

1. Total Capital Construction Cost

Project capital costs for Phase One, or capital costs for the first ten years of operation are shown in Table V-10. Project capital costs to increase capacity for Phase Two, or capital costs for years 11 to 20 are shown in Table V-11. A breakdown of the capital construction costs are provided in Appendix F.

Table V-10: Probable Capital Construction Cost Summary – Phase One

	Alternatives						
	Convert Existing Ponds to Aerated Lagoons with OPTAER Process and SAGR	Convert Existing Ponds to Aerated Lagoons with LEMNA Process	New Harrisburg WWTP			Pump to Sioux Falls: Can-Type LS and Use Existing Ponds for Equalization Storage	Pump to Sioux Falls: Can-Type LS and Construct New Lagoons Outside City Limits
Treatment Process			SBR	Conventional AS	MBR		
Gravity Sanitary Sewer Interceptor	$ 416,700	$ 416,700	$ 5,084,000	$ 5,084,000	$ 5,084,000	$ 416,700	$ 416,700
Lift Station	$ -	$ -	$ 1,488,800	$ 1,488,800	$ 1,488,800	$ 1,488,800	$ 2,389,500
Equalization Basin	$ -	$ -	$ 205,000	$ 205,000	$ 2,307,600	$ -	$ 2,307,600
Floating Aeration Units	$ -	$ -	$ -	$ -	$ -	$ 40,000	$ -
Force Main to Harrisburg WWTP	$ -	$ -	$ 2,369,000	$ 2,369,000	$ 2,369,000	$ -	$ -
Force Main to LS #240	$ -	$ -	$ -	$ -	$ -	$ 4,074,800	$ 4,074,800
Force Main from LS#240 to Future SF WWTP	$ -	$ -	$ -	$ -	$ -		
Preliminary Treatment	$ 200,000	$ 300,000	$ 776,800	$ 776,800	$ 985,800	$ 200,000	$ -
Primary Treatment	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Secondary Treatment	$ 3,510,000	$ 3,315,000	$ 2,204,350	$ 3,560,100	$ 4,151,800	$ -	$ -
Dual Stage Vertical Flow Gravity Sand Filters w/Alum Feed System	$ 1,550,000	$ 1,550,000				$ -	$ -
Blower/Chemical Feed Building	$ 150,000	$ 150,000				$ -	$ -
Disinfection Treatment	$ 300,000	$ 300,000	$ 260,500	$ 193,500	$ 193,500	$ -	$ -
Solids Digestion	$ -	$ -	$ 1,232,900	$ 1,232,900	$ 1,232,900	$ -	$ -
Solids Thickening/Dewatering	$ -	$ -	$ 1,489,300	$ 1,489,300	$ 1,489,300	$ -	$ -
Electrical/I&C	$ 450,000	$ 450,000				$ -	$ -
Control Building	$ -	$ -	$ 260,000	$ 260,000	$ 260,000	$ -	$ -
WWTP Sitework	$ 1,968,580	$ 789,580	$ 622,400	$ 751,300	$ 831,300	$ -	$ -
WWTP Outfall/Discharge Piping to Ninemile Creek/Discharge Piping to Wet Well	$ 533,720	$ 533,720	$ 715,600	$ 715,600	$ 715,600	$ 72,500	$ 72,500
Land Acquisition	$ -	$ -	$ 690,000	$ 690,000	$ 1,057,500	$ -	$ 525,000
Mobilization (8%)	$ 765,000	$ 654,000	$ 1,513,000	$ 1,636,000	$ 1,928,000	$ 547,500	$ 851,000
Subtotal Construction Costs	**$ 9,844,000**	**$ 8,459,000**	**$ 18,911,650**	**$ 20,452,300**	**$ 24,095,100**	**$ 6,841,000**	**$ 10,637,100**
Contingency (20%)	$ 1,968,800	$ 1,691,800	$ 3,782,400	$ 4,090,500	$ 4,819,100	$ 1,369,000	$ 2,127,500
Preliminary Opinion of Construction Costs	$ 11,812,800	$10,150,800	$ 22,694,050	$ 24,542,800	$ 28,914,200	$ 8,210,000	$ 12,764,600
Engineering, Legal, Construction Administration (20%)	$ 2,363,000	$ 2,031,000	$ 4,539,000	$ 4,909,000	$ 5,783,000	$ 1,642,000	$ 2,553,000
Total Engineer's Opinion of Probable Project Construction Cost	**$ 14,175,800**	**$12,181,800**	**$ 27,233,100**	**$ 29,451,800**	**$ 34,697,200**	**$ 9,853,000**	**$ 15,317,600**

Table V-11: Probable Capital Construction Cost Summary – Phase Two

Treatment Process	Alternatives						
	Convert Existing Ponds to Aerated Lagoons with OPTAER Process and SAGR	Convert Existing Ponds to Aerated Lagoons with LEMTEC Process	New Harrisburg WWTP			Pump to Sioux Falls: Can-Type LS and Use Existing Ponds for Equalization Storage	Pump to Sioux Falls: Can-Type LS and Construct New Lagoons Outside City Limits
			SBR	Conventional AS	MBR		
Gravity Sanitary Sewer Interceptor	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Influent Lift Station	$ -	$ -	$ 536,500	$ 536,500	$ 536,500	$ 824,500	$ 824,500
Equalization Basin	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Floating Aeration Units	$ -	$ -	$ -	$ -	$ -	$ 100,000	$ -
Force Main to Harrisburg WWTP	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Force Main to LS #240	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Force Main from LS#240 to Future SF WWTP	$ -	$ -	$ -	$ -	$ -	$ 537,300	$ 537,300
Preliminary Treatment	$ -	$ -	$ 381,500	$ 381,500	$ 592,500	$ -	$ -
Primary Treatment	$ -	$ -	$ 1,568,600	$ 1,568,600	$ 1,568,600	$ -	$ -
Secondary Treatment	$ -	$ 3,009,500	$ 1,903,200	$ 531,000	$ 4,184,900	$ -	$ -
Disinfection Treatment	$ -	$ -	$ -	$ 67,000	$ 67,000	$ -	$ -
Solids Digestion	$ -	$ -	$ 1,050,500	$ 1,050,500	$ 1,050,500	$ -	$ -
Solids Thickening/Dewatering	$ -	$ -	$ 373,100	$ 373,100	$ 373,100	$ -	$ -
Electrical/I&C		$ 40,000					
Control Building	$ -	$ -	$ -	$ -	$ -	$ -	$ -
WWTP Sitework	$ -	$ 100,000	$ 527,700	$ 397,200	$ 783,700	$ -	$ -
WWTP Outfall/Discharge Piping to Ninemile Creek/Discharge Piping to Wet Well	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Land Acquisition	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Mobilization (8%)	$ -	$ 320,000	$ 704,600	$ 545,000	$ 1,017,000	$ 91,600	$ 91,600
Subtotal Construction Costs	**$ -**	**$ 3,469,500**	**$ 7,045,700**	**$ 5,450,400**	**$ 10,173,800**	**$ 1,553,400**	**$ 1,453,400**
Contingency (20%)	$ -	$ 694,000	$ 1,410,000	$ 1,091,000	$ 2,035,000	$ 311,000	$ 291,000
Preliminary Opinion of Construction Costs	$ -	$ 4,163,500	$ 8,455,700	$ 6,541,400	$ 12,208,800	$ 1,864,400	$ 1,744,400
Engineering, Legal, Construction Administration (20%)	$ -	$ 833,000	$ 1,692,000	$ 1,309,000	$ 2,442,000	$ 373,000	$ 349,000
Total Engineer's Opinion of Probable Project Construction Cost	**$ -**	**$ 4,997,000**	**$ 10,148,000**	**$ 7,851,000**	**$ 14,651,000**	**$ 2,238,000**	**$ 2,094,000**

2. Operation and Maintenance Cost

In addition to capital costs, the City will incur additional operating expenses for the proposed treatment processes. For each process, these costs can be divided into energy, labor, repairs, and maintenance.

Energy costs would result primarily from the electrical cost of providing power for screening, grit removal, aeration, pumping, and disinfection. These annual costs were calculated assuming an average unit energy cost of $0.06 per kW-hr.

Additional labor will be required for daily operational and maintenance needs for the recommended treatment improvements. If Harrisburg constructs its own WWTP, an equivalent of one and a half (1 ½) additional full-time employees (FTE) will be required for the 2021 design year. An annual labor cost for the 2021 design year, including benefits, is estimated to be $90,000 per year. A total of two (2) FTEs are expected for the 2031 design year. These labor costs are included in the secondary treatment O&M costs. If Harrisburg elects to pump its wastewater to the City of Sioux Falls for treatment, it was assumed that approximately 260 hours of labor would be required annually to maintain the lift station, equalization basin and force main. If Harrisburg converts the ponds to aerated lagoons, the estimated staffing level is three-quarters of an employee for daily operational. The operator-in-charge will likely be required to hold a Class IV Operators Certificate. This change may require additional training and certification for plant staff.

Each of the mechanical and structural improvements would also require periodic repairs and maintenance to keep plant performance at an acceptable level. These annual costs were calculated using routine maintenance and repair frequencies and information provided by the equipment manufacturers.

Annual O&M costs for the first 10 years of operations are shown in Table V-12. Annual O&M costs for 2021 - 2031 are shown in Table V-13. O&M costs have been calculated from current costs assuming a 3.0 percent inflation rate and a 4.75 percent interest rate. While this interest rate is somewhat high for the current economic climate, it is assumed that interest rates will increase in subsequent years and that 4.75 percent is more typical of a normal economy. Detailed annual O&M cost breakdowns are included in Appendix G.

Harrisburg will incur monthly fees from the City of Sioux Falls if they pump to Sioux Falls for treatment. Sioux Falls has indicated that a current rate would be approximately $1.80/1,000 gallons of wastewater received, although this still needs to be negotiated. Sioux Falls has also indicated that the rate would increase approximately 3% annually. This information was used to prepare the present worth O&M cost for pumping to Sioux Falls. The calculation was based on 30 days of AWW flow and 335 days of ADW flow each year. The annual cost for pumping to Sioux Falls is provided in Appendix G.

Table V-12: Probable Project Annual Total O&M Costs[1] Summary – Phase One

Treatment Process	Alternatives						
	Convert Existing Ponds to Aerated Lagoons with OPTAER Process and SAGR	Convert Existing Ponds to Aerated Lagoons with LEMNA Process	New Harrisburg WWTP			Pump to Sioux Falls: Can-Type LS and Use Existing Ponds for Equalization Storage	Pump to Sioux Falls: Can-Type LS and Construct New Lagoons Outside City Limits
			SBR	Conventional AS	MBR		
Gravity Sanitary Sewer Interceptor	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Influent Lift Station	$ -	$ -	$ 52,600	$ 52,600	$ 52,600	$ 57,250	$ 57,250
Equalization Basin	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Force Main to WWTP	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Force Main to LS #240	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Force Main from LS#240 to Future SF WWTP	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Preliminary Treatment	$ 2,900	$ 2,900	$ 2,900	$ 2,900	$ 2,900	$ -	$ -
Primary Treatment	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Secondary Treatment	$ 267,000	$ 271,500	$ 116,000	$ 160,630	$ 164,150	$ -	$ -
Dual Stage Vertical Flow Gravity Sand Filters w/Alum Feed System	$ 24,200	$ 24,200	$ -	$ -	$ -	$ -	
Blower/Chemical Feed Building	Incl. w/Sec Treatment	Incl. w/Sec Treatment	$ -	$ -	$ -	$ -	
Disinfection Treatment	$ 12,400	$ 12,400	$ 21,500	$ 12,400	$ 12,400	$ -	$ -
Solids Digestion	$ -	$ -	$ 106,700	$ 106,700	$ 106,700	$ -	$ -
Solids Thickening/Dewatering	$ -	$ -	Incl w/Solids Digestion	Incl w/Solids Digestion	Incl w/Solids Digestion	$ -	$ -
Solids Disposal	Incl. w/Sec Treatment	Incl. w/Sec Treatment	$ 71,200	$ 71,200	$ 71,200	$ -	$ -
Sioux Falls Treatment of Wastewater	$ -	$ -	$ -	$ -	$ -	$ 348,802	$ 348,802
Total	**$ 306,500**	**$ 311,000**	**$ 370,900**	**$ 406,500**	**$ 410,000**	**$ 406,100**	**$ 406,100**

[1] O&M Costs includes energy, labor, and repair/replacement costs

Table V-13: Probable Project Annual Total O&M Costs[1] Summary – Phase Two

Treatment Process	Alternatives						
	Convert Existing Ponds to Aerated Lagoons with OPTAER Process and SAGR	Convert Existing Ponds to Aerated Lagoons with LEMNA Process	New Harrisburg WWTP: SBR	New Harrisburg WWTP: Conventional AS	New Harrisburg WWTP: MBR	Pump to Sioux Falls: Can-Type LS and Use Existing Ponds for Equalization Storage	Pump to Sioux Falls: Can-Type LS and Construct New Lagoons Outside City Limits
Gravity Sanitary Sewer Interceptor	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Influent Lift Station	$ -	$ -	$ 61,900	$ 61,900	$ 61,900	$ 75,900	$ 75,900
Equalization Basin	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Force Main to WWTP	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Force Main to LS #240	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Force Main from LS#240 to Future SF WWTP	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Preliminary Treatment	$ 8,900	$ 8,900	$ 8,900	$ 8,900	$ 8,900	$ -	$ -
Primary Treatment	$ -	$ -	$ 7,360	$ 7,360	$ 7,360	$ -	$ -
Secondary Treatment	$ 355,000	$ 432,200	$ 213,000	$ 293,700	$ 302,540	$ -	$ -
Dual Stage Vertical Flow Gravity Sand Filters w/Alum Feed System	$ 30,400	$ 30,400	$ -	$ -	$ -	$ -	
Blower/Chemical Feed Building	Incl. w/Sec Treatment	Incl. w/Sec Treatment	$ -	$ -	$ -	$ -	
Disinfection Treatment	$ 22,240	$ 22,240	$ 18,930	$ 22,240	$ 22,240	$ -	$ -
Solids Digestion	$ -	$ -	$ 17,550	$ 17,550	$ 17,550	$ -	$ -
Solids Thickening/Dewatering	NA	NA	Incl. w/Solids Digestion	Incl. w/Solids Digestion	Incl. w/Solids Digestion	NA	NA
Solids Disposal	Incl. w/Sec Treatment	Incl. w/Sec Treatment	$ 60,600	$ 60,600	$ 60,600	$ -	$ -
Sioux Falls Treatment of Wastewater	$ -	$ -	$ -	$ -	$ -	$ 605,317	$ 605,317
Total	**$ 416,600**	**$ 493,800**	**$ 388,300**	**$ 472,300**	**$ 481,100**	**$ 681,300**	**$ 681,300**

[1] O&M Costs includes energy, labor, and repair/replacement costs

3. Present Worth Analysis

The present worth of all costs, including Phase One and Phase Two capital construction costs, and O&M costs were calculated and are shown in Table V-14.

Table V-14: Probable Project Present Worth Costs Summary

	Alternatives						
			New Harrisburg WWTP				
Treatment Process	**Convert Existing Ponds to Aerated Lagoons with OPTAER Process and SAGR**	**Convert Existing Ponds to Aerated Lagoons with LEMNA Process**	**SBR**	**Conventional AS**	**MBR**	**Pump to Sioux Falls: Can-Type LS and Use Existing Ponds for Equalization Storage**	**Pump to Sioux Falls: Can-Type LS and Construct New Lagoons Outside City Limits**
Gravity Sanitary Sewer Interceptor							
Capital Present Worth	$ 416,700	$ 416,700	$ 5,084,000	$ 5,084,000	$ 5,084,000	$ 416,700	$ 416,700
O&M Present Worth	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Influent Lift Station							
Capital Present Worth	$ -	$ -	$ 2,025,300	$ 2,025,300	$ 2,025,300	$ 2,313,300	$ 3,214,000
O&M Present Worth	$ -	$ -	$ 925,200	$ 925,200	$ 925,200	$ 1,016,500	$ 1,016,500
Equalization Basin							
Capital Present Worth	$ -	$ -	$ 205,000	$ 205,000	$ 2,307,600	$ -	$ 2,307,600
O&M Present Worth	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Force Main to Harrisburg WWTP							
Capital Present Worth	$ -	$ -	$ 2,369,000	$ 2,369,000	$ 2,369,000	$ -	$ -
O&M Present Worth	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Force Main to LS #240							
Capital Present Worth	$ -	$ -	$ -	$ -	$ -	$ 4,074,800	$ 4,074,800
O&M Present Worth	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Force Main from LS #240 to SF WWTP							
Capital Present Worth	$ -	$ -	$ -	$ -	$ -	$ 1,074,600	$ 1,074,600
O&M Present Worth	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Sioux Falls Treatment of Wastewater							
Capital Present Worth	$ -	$ -	$ -	$ -	$ -	$ -	$ -
O&M Present Worth	$ -	$ -	$ -	$ -	$ -	$ 9,541,196	$ 9,541,196
Preliminary Treatment							
Capital Present Worth	$ 200,000	$ 300,000	$ 1,158,300	$ 1,158,300	$ 1,578,300	$ 200,000	$ -
O&M Present Worth	$ -	$ -	$ 101,300	$ 101,300	$ 101,300	$ -	$ -
Primary Treatment							
Capital Present Worth	$ -	$ -	$ 1,568,600	$ 1,568,600	$ 1,568,600	$ -	$ -
O&M Present Worth	$ -	$ -	$ 60,000	$ 60,000	$ 60,000	$ -	$ -
Secondary Treatment							
Capital Present Worth	$ 3,510,000	$ 6,324,500	$ 4,107,550	$ 4,091,100	$ 8,336,700	$ -	$ -
O&M Present Worth	$ 5,510,100	$ 6,037,800	$ 2,333,700	$ 3,605,500	$ 3,424,900	$ -	$ -
Filters w/Alum Feed System							
Capital Present Worth	$ 1,550,000	$ 1,550,000	$ -	$ -	$ -	$ -	$ -
O&M Present Worth	$ 315,000	$ 315,000	$ -	$ -	$ -	$ -	$ -
Blower/Chemical Feed Building							
Capital Present Worth	$ 150,000	$ 150,000	$ -	$ -	$ -	$ -	$ -
O&M Present Worth	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Disinfection Treatment							
Capital Present Worth	$ 300,000	$ 300,000	$ 260,500	$ 260,500	$ 260,500	$ -	$ -
O&M Present Worth	$ 289,900	$ 289,900	$ 289,900	$ 245,900	$ 245,900	$ -	$ -
Solids Digestion							
Capital Present Worth	$ -	$ -	$ 2,283,400	$ 2,283,400	$ 2,283,400	$ -	$ -
O&M Present Worth	$ -	$ -	$ 907,200	$ 907,200	$ 907,200	$ -	$ -
Solids Thickening/Dewatering							
Capital Present Worth	$ -	$ -	$ 1,862,400	$ 1,862,400	$ 1,862,400	$ -	$ -
O&M Present Worth	NA	NA	Incl. w/Solids Digestion	Incl. w/Solids Digestion	Incl. w/Solids Digestion	NA	NA
Solids Disposal							
Capital Present Worth	$ -	$ -	$ -	$ -	$ -	$ -	$ -
O&M Present Worth	$ -	$ -	$ 1,038,700	$ 1,038,700	$ 1,038,700	$ -	$ -
Electrical/I&C							
Capital Present Worth	$ 450,000	$ 450,000	$ -	$ -	$ -	$ -	$ -
O&M Present Worth	$ -	$ -	$ -	$ -	$ -	$ -	$ -

Table V-14: Probable Project Present Worth Costs Summary (cont.)

	Alternatives						
			New Harrisburg WWTP				
Treatment Process	**Convert Existing Ponds to Aerated Lagoons with OPTAER Process and SAGR**	**Convert Existing Ponds to Aerated Lagoons with LEMNA Process**	**SBR**	**Conventional AS**	**MBR**	**Pump to Sioux Falls: Can-Type LS and Use Existing Ponds for Equalization Storage**	**Pump to Sioux Falls: Can-Type LS and Construct New Lagoons Outside City Limits**
Control Building							
Capital Present Worth	$ -	$ -	$ 260,000	$ 260,000	$ 260,000	$ -	$ -
O&M Present Worth	$ -	$ -	$ -	$ -	$ -	$ -	$ -
WWTP Sitework							
Capital Present Worth	$ 1,968,580	$ 889,580	$ 1,150,100	$ 1,148,500	$ 1,615,000	$ -	$ -
O&M Present Worth	$ -	$ -	$ -	$ -	$ -	$ -	$ -
WWTP Outfall/Discharge Piping to Wet Well							
Capital Present Worth	$ 533,720	$ 533,720	$ 715,600	$ 715,600	$ 715,600	$ 72,500	$ 72,500
O&M Present Worth	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Land Acquisition							
Capital Present Worth	$ -	$ -	$ 690,000	$ 690,000	$ 1,057,500	$ -	$ 525,000
O&M Present Worth	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Mobilization							
Capital Present Worth	$ 765,000	$ 654,000	$ 1,513,000	$ 1,636,000	$ 1,928,000	$ 547,500	$ 851,000
O&M Present Worth	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Subtotal Construction Costs	**$ 9,844,000**	**$ 11,568,500**	**$ 25,252,750**	**$ 25,357,700**	**$ 33,251,900**	**$ 8,699,400**	**$ 12,536,200**
Contingency (20%)	$ 1,968,800	$ 2,313,700	$ 5,050,600	$ 5,071,600	$ 6,650,400	$ 1,739,900	$ 2,507,300
Preliminary Opinion of Construction Costs	$ 11,812,800	$ 13,882,200	$ 30,303,350	$ 30,429,300	$ 39,902,300	$ 10,439,300	$ 15,043,500
Engineering, Legal, Construction Administration (20%)	$ 2,363,000	$ 2,777,000	$ 6,061,000	$ 6,086,000	$ 7,981,000	$ 2,088,000	$ 3,009,000
Total Present Worth Probable Project Construction Cost	**$ 14,175,800**	**$ 16,659,200**	**$ 36,364,400**	**$ 36,515,300**	**$ 47,883,300**	**$ 12,527,300**	**$ 18,052,500**
O&M Present Worth	$ 6,115,000	$ 6,642,700	$ 5,656,000	$ 6,883,800	$ 6,703,200	$ 10,557,696	$ 10,557,696
Overall Present Worth Engineer's Opinion of Probable Cost	**$ 20,291,000**	**$ 23,302,000**	**$ 42,021,000**	**$ 43,400,000**	**$ 54,587,000**	**$ 23,085,000**	**$ 28,611,000**

C. DEMONSTRATION OF FINANCIAL CAPABILITY

The City of Harrisburg will work closely with the Southeast Council of Governments (SECOG) and Toby Morris of Northland Securities, Inc., the City's financial advisor, to develop a financing plan. The City would like to fund the proposed improvements with a combination of a State Revolving Fund (SRF) loan, grants, economic stimulus funds, federal appropriations, or other potential federal funding options.

A preliminary cash flow analysis and amortization table is included in Appendix H. This analysis considers a $9,853,000 project loan from the Clean Water SRF for pumping Harrisburg's wastewater to Sioux Falls for treatment and using the existing evaporation ponds for equalization and storage. Ideally, user fees would fund the sanitary department operating budget and project debt repayment. The City will be seeking Federal and State assistance to fund a portion of the project.

The analysis shows that the rate increases needed to repay the entire annual debt service would burden the residents of Harrisburg. The project cash flow analysis indicates the annual debt service to repay the Clean Water SRF loan at an interest rate of 3.00% would be $662,280. To fund the debt repayment and the additional O&M costs, rates would need to increase 45% each year for 2010, 2011, and 2012. In 2013, rates increases could be reduced to 3% annually to fund the debt. Projections were not made beyond 2015. This would result in rates increasing to $35.94 for the monthly customer charge and $8.50 per 1,000 gallons for the volume charge in 2015.

The City completed a phone survey in 2006 to determine their eligibility to qualify for lower to moderate income (LMI) status. The phone survey concluded that the average household income was too high to qualify for LMI. A copy of the survey and survey results are included in Appendix I.

D. CAPITAL FINANCING PLAN

The SRF Drinking Water Application has the complete Capital Financing Plan.

E. ENVIRONMENTAL EVALUATION

The City of Harrisburg recognizes the need to plan for the future wastewater needs of the community. The existing lagoons will reach capacity this year as a result of the recent growth. The City must develop and implement a new treatment alternative immediately to prevent environmental damage from an unpermitted overflow.

Each option considered was evaluated for its environmental impact.

1. Conversion of Evaporation Ponds to Aerated Lagoons

The conversion of the existing evaporation ponds to aerated lagoons with additional treatment for ammonia and phosphorus removal would not require taking land out of agricultural production.

Construction of the piping and blower building to provide aeration to the existing ponds, construction of the SAGR cells, and other minor construction on the site has several short-term impacts to the environment. Construction of these facilities would create dust and emissions common to this type of activity. Erosion control measures implemented during construction would minimize impact to the sites and adjacent properties.

The aerated ponds would likely have some odor problems. These odor problems would be less initially, and build over the years as loading to the first cell increased. This may cause concerns with nearby residences.

The most beneficial environmental impact of this alternative would be the safe and proper treatment of Harrisburg's wastewater. The City's evaporation ponds will soon reach capacity and this alternative provides for the safe treatment and discharge of the wastewater, protecting the environment. In addition, the preliminary discharge permit for this option indicates ammonia and phosphorus removal would be provided. This option provides the highest level of treatment prior to discharge.

2. Sequential Batch Reactor (SBR) WWTP

The construction of a new SBR WWTP would require taking land out of agricultural use for the lift station, equalization basin, and WWTP site. This would consist of approximately 46.0 acres.

Construction of the improvements has several short-term impacts to the environment. Construction of these facilities would create dust and emissions common to this type of activity. Erosion control measures implemented during construction would minimize impact to the sites and adjacent properties.

One common concern many property owners have regarding WWTP sites is the odor. New treatment technology and odor control equipment would be employed at the lift station and WWTP site to minimize odors. Buffer property will also be used to surround the WWTP site to minimize the impacts to adjacent property owners.

The most beneficial environmental impact of the new WWTP would be the safe and proper treatment of Harrisburg's wastewater. The City's evaporation ponds will soon reach capacity and the SBR WWTP alternative provides for the safe treatment and discharge of the wastewater, protecting the environment.

3. Conventional Activated-Sludge WWTP

The environmental impact for the conventional activated-sludge WWTP alternative would be similar to that of the SBR WWTP discussed above.

4. Membrane Bioreactor (MBR) WWTP

The selection of the MBR WWTP alternative would require taking land out of agricultural use for the lift station, equalization basin, and WWTP site. This would consist of approximately 70.5 acres. A large equalization basin is required at the lift station site to minimize the peak flows sent to the WWTP.

Construction of the improvements would have similar short-term impacts to the environment as the other treatment plant alternatives. Construction would create dust and emissions common to this type of activity. Erosion control measures implemented during construction would minimize impact to the sites and adjacent properties.

One common concern many property owners have regarding WWTP sites is the odor. New treatment technology and odor control equipment would be employed at the WWTP site to minimize odors. Buffer property will also be used to surround the WWTP site to minimize the impacts to adjacent property owners. The 15-acre equalization basin at the lift station site would likely have periodic odor problems due to the large amount of raw sewage being stored to reduce peak flows. This may cause concerns with nearby residences.

The most beneficial environmental impact of a new WWTP would be the safe and proper treatment of Harrisburg's wastewater. The City's evaporation ponds will soon reach capacity and the MBR WWTP alternative provides for the safe treatment and discharge of the wastewater, protecting the environment.

5. Pump Wastewater to the City of Sioux Falls for Treatment

Pumping wastewater to the City of Sioux Falls for Treatment and using the existing evaporation ponds for equalization and storage will require taking no land out of agricultural service.

Construction of the lift station, equalization, basin and force main has several short-term impacts to the environment. Construction of these facilities would create dust and emissions common to this type of activity. Erosion control measures implemented during construction would minimize impact to the sites and adjacent properties.

Using the evaporation ponds as the equalization basin will cause increasing odor problems over time. Costs to add floating surface aerators are included initially and for Phase Two to minimize the odor impacts. Odors in the force main discharge at Lift Station #240 may also cause concerns with nearby residences. Chemical treatment may be required to minimize odors.

The most beneficial environmental impact of sending Harrisburg's wastewater to Sioux Falls for treatment would be the safe and proper treatment of Harrisburg's wastewater. The City's evaporation ponds will soon reach capacity and this alternative provides for the safe treatment and discharge of the wastewater, protecting the environment.

F. COMPARISON OF ALTERNATIVES

Each of the alternatives was evaluated and compared to determine the best option for the City of Harrisburg. Each option incorporates a phased approach to minimize the initial costs to the City. The advantages and disadvantages of each option are summarized below.

1. Conversion of Evaporation Ponds to Aerated Lagoons

The main advantage of aerated lagoons is the reuse of the existing evaporation ponds. The equipment can be installed without draining the existing cells, which would allow for treatment during construction. The OPTER aerated lagoon system has the lowest present worth cost based on capital costs and O&M.

The major disadvantage of the aerated lagoon options is the lack of multiple installations in this region and the unknown amount of ammonia removal to be achieved. The proximity of the existing ponds to the City and residents could also cause issues with odor complaints over time.

The capital cost is further increased by the need for a headworks building for screening, a blower building, chemical addition for phosphorus removal, sand filters for phosphorus removal, and the addition of UV disinfection. The increased energy costs are a major disadvantage.

2. New Mechanical Wastewater Treatment Plant

A new mechanical WWTP would allow the City of Harrisburg to produce a high quality effluent with the potential for future chemical phosphorus removal. In addition, it would allow for Harrisburg to easily expand its capacity in the future. However, the upfront capital costs of the mechanical WWTP alternatives considered prevent them from being viable options. If mechanical WWTP were constructed, the SBR alternative has the lowest upfront capital and O&M costs.

3. Pump Wastewater to the City of Sioux Falls for Treatment

Pumping wastewater to Sioux Falls for treatment has several advantages. First, it has the lowest upfront capital cost of all the options considered. Second, it will not require the City to have the responsibility of meeting a challenging discharge permit that includes nitrogen and phosphorus removal. In addition, this option provides the fastest construction schedule for Harrisburg to alleviate the over loading of their existing evaporation ponds. It also promotes regionalization. Finally, the City of Harrisburg will not have to hire the additional labor needed to operate and maintain a WWTP.

In looking to the future, portions of this project can continue to be used beyond the 20 year planning period. The force main should last much longer than 20 years. Should the City grow such that the lift station would need to be abandoned, and a new one constructed further down stream to the south and east, a large portion of the force main could be reused.

This alternative, of course, also has disadvantages. First, the force main distance required to convey wastewater from Harrisburg to Lift Station #240, and eventually the new Sioux Falls WWTP is over 56,000 feet, or almost eleven miles. Depending on the selected route, the head conditions require placing the pumps in series. The pumping industry prefers these pumps not be operated in series due to the maintenance concerns that can develop. Solids settlement in the force main is also of great concern due to the time it would take to turn over the force main contents.

Odor may also be an issue with this alternative at the evaporation ponds, which will be used as an equalization basin, and at Lift Station #240 where the force main will discharge. Additional aeration will be needed at the equalization basin to control the odor problems. Chemical addition may be required in the force main to control odors. The odors will be the worst at Lift Station #240 initially, when flows are the lowest. Odors at the equalization basins will worsen over time. These odors may affect residents in the vicinity of either location.

G. VIEWS OF THE PUBLIC AND CONCERNED INTEREST GROUPS

A public hearing was held with proper notification during a regularly scheduled City Council meeting at 6:30 p.m. on April 13, 2009. No public comments were received. The Affidavit of Publication, sign-in sheet, and public hearing meeting minutes are provided in Appendix J.

The public hearing was originally scheduled for March 16, 2009, but was postponed to obtain further information on the proposed funding application. A few public comments were received at the March 16, 2009 meeting and these minutes are also included in Appendix J. Calculations were performed to determine that amount of land needed to discharge using spray irrigation to address one of the comments. In 2019, 1882 acres would be needed for spray irrigation at projected flows. In 2029, 3,400 acres would be needed for spray irrigation at projected flows. Because of the large land area required this option is not considered feasible.

VI. SELECTED PLAN, DESCRIPTION AND IMPLEMENTATION ARRANGEMENTS

A. JUSTIFICATION AND DESCRIPTION OF SELECTED PLAN

Based upon extensive study of the options and discussions with the City, HR Green recommends that Harrisburg proceed with pumping wastewater to Sioux Falls for treatment. The largest driver for the selection of this alternative was the low upfront capital cost compared to the other alternative. Funding this project has been a challenge for the City since it began evaluating treatment options in 2007. The City is hopeful that with economic stimulus funding and other grants they will be able to construct the required improvements.

B. DESIGN OF SELECTED PLAN

Design and construction of the selected plan is critical to providing wastewater treatment as soon as possible since Harrisburg's evaporation ponds are projected to reach capacity in the spring of 2009. It is a priority for Harrisburg to provide the safe treatment and discharge of its wastewater. The City realizes that several key tasks must be completed before design and construction can begin. Most importantly, the SD DENR must approve the Facility Plan and funding must be secured. In addition, the City needs City, Township and County approval for installation of the force main in the right-of-way. Finally, the City of Harrisburg and the City of Sioux Falls need to create a contract for the treatment of Harrisburg's wastewater. Harrisburg is prepared to work as quickly as possible with the City Engineer on the design of the proposed improvements and intends to bid the project yet this fall.

C. COST ESTIMATES FOR THE SELECTED PLAN

A summary of the Engineer's Opinion of Probable Capital Construction Cost for the recommended Phase One construction of a new lift station and force main to convey Harrisburg's wastewater to Sioux Falls for treatment is provided in Table VI-1.

Table VI-1: Engineer's Opinion of Probable Capital Construction Cost for the Recommended Construction of a New Lift Station and Force Main to Pump Harrisburg's Wastewater to Sioux Falls for Treatment

Treatment Process	Pump to Sioux Falls: Can-Type LS and Use Existing Ponds for Equalization Storage
Gravity Sanitary Sewer Interceptor	$ 416,700
Lift Station	$ 1,488,800
Floating Aeration Units	$ 40,000
Force Main to LS #240	$ 4,074,800
Preliminary Treatment	$ 200,000
WWTP Discharge Piping to Wet Well	$ 72,500
Mobilization (8%)	$ 547,500
Subtotal Construction Costs	**$ 6,841,000**
Contingency (20%)	$ 1,369,000
Preliminary Opinion of Construction Costs	**$ 8,210,000**
Engineering (8%)	$ 657,000
Construction Administration (8%)	$ 657,000
Legal Costs (4%)	$ 329,000
Total Engineer's Opinion of Probable Project Construction Cost	**$ 9,853,000**

D. USER RATE IMPACTS

The City of Harrisburg's current monthly sanitary sewer rates are $11.00 for the customer charge and $2.60 per 1,000 gallons for the usage charge. The City also charges a hook-up fee on all new construction building permits of $500.00.

As with any community, the City wishes to keep increases in user rates to a minimum. However, they realize that increases to rates and connection fees will be required to fund the proposed improvements. A 45% rate increase would be required each year in 2010, 2011, and 2012 to fund the $9.853 million proposed wastewater treatment system improvements. Three percent annual rate increases would be required in following years. Assuming these increases, the customer charge rate would increase to $35.94, and the usage charge would increase to $8.50/1,000 gallons in 2015. A rate increase this significant would be a burden to the City. Grant, economic stimulus, and federal appropriation funds will be requested to try and lower the City's portion of the project cost.

The City has increased rates annually on January 1st of each year for several years. An evaluation of the impact to user rates is included in the cash flow analysis in Appendix H.

E. ENVIRONMENTAL IMPACTS OF SELECTED PLAN

This report addresses several of the environmental impacts that will occur due to the pumping of Harrisburg's wastewater to Sioux Falls for treatment. Most significantly, this option provides for the safe and proper treatment of Harrisburg's wastewater for many years to come.

Construction of the lift station and force main will not remove agricultural land from service. Construction will create short-term dust and emissions typical with construction projects. Erosion control measures implemented during construction would minimize impact to the sites and adjacent properties.

Odors at the site of the equalization basin would be minimized with aeration and available treatment technology and odor control equipment could be used to reduce odors at Lift Station #240 if needed.

F. ARRANGEMENTS FOR IMPLEMENTATION

The City understands that a project of this magnitude will require significant planning and coordination between the City, State, and other funding agencies. Harrisburg is prepared to work together to provide for the future safe disposal of their community's wastewater. The schedule in Section VI.H identifies some of the major tasks to implement to project.

1. Intermunicipal Service Agreements
   Harrisburg will need to enter into an intermunicipal service agreement with Sioux Falls to treat their wastewater. The negotiation of this contract will begin later this spring. Harrisburg has been discussing this option with Sioux Falls for several years. No difficulties are anticipated in the agreement at this time.

2. Operation and Maintenance (O&M) Requirements
   This report identifies the O&M requirements of the proposed improvements. The City will need to prepare itself financially to fund the annual staff and equipment costs.

3. Pre-treatment Program
   Prior to a business or industry with a high strength waste establishing itself in Harrisburg, a pre-treatment program would be implemented.

G. LAND ACQUISITION

1. General Acquisition
   Land acquisition is not anticipated for the recommended option of pumping wastewater to Sioux Falls for treatment. The lift station will be constructed at the site of the existing evaporation ponds and the force main will be installed within the road right-of-way.

2. Acquisition Method
Land acquisition is not anticipated for this project.

3. Land Costs
For purposes of this report, land was assumed to cost $15,000 per acre. Other fees associated with purchasing, such as closing costs and legal costs the land were not included. Finally, costs for obtaining conditional land use permits from Lincoln County were not included in this report.

H. IMPLEMENTATION SCHEDULE

The City of Harrisburg is committed to constructing the lift station and force main to pump its wastewater to Sioux Falls for Treatment. The schedule described in Table VI-2 provides the tasks and dates for implementation of the project.

Table VI-2: Schedule for Implementation of Selected Alternative

Task	Start Date	Completion Date
Submit Revisions and Cost Updates for Placement on the State-Intended Use Plan (Include Draft Facility Plan)	March 2009	March 2009
Submit Facility Plan to City for Review	March 2009	March 2009
Complete Environmental Review	March 2009	April 2009
Public Hearing for Facility Plan	April 13, 2009	April 13, 2009
Submit Facility Plans to SD DENR, Apply for State Grant/Economic Stimulus/and CW-SRF Funding	April 2009	April 2009
Obtain Permits for Force Main Placement in City/Township/County/State Right-of-Way	April 2009	June 2009
State Grant/Economic Stimulus/and CW-SRF Funding Determined	May 2009	June 2009
Negotiate Contract and Treatment Rate with the City of Sioux Falls	May 2009	August 2009
Site Survey	May 2009	June 2009
Preliminary Design	May 2009	June 2009
Final Design	July 2009	September 2009
Advertise/Bid/Award/Notice to Proceed	October 2009	November 2009
Construction	December 2009	October 2010
Substantial Completion	October 2010	October 2010
Startup	October 2010	October 2010
Final Completion	November 2010	November 2010