October 2007

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

OCTOBER 2007

I hereby certify that this engineering document was prepared by me or under my direct personal supervision and that I am a duly licensed Professional Engineer under the laws of the State of South Dakota.

Date: ____________

____
TANYA L. MILLER, P.E.

License No. **8326**

My renewal date is **June 30, 2008**

Pages or sheets covered by this seal:
Entire report.

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TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

I. EXECUTIVE SUMMARY

A. PURPOSE

Harrisburg's existing evaporation ponds are projected to reach capacity in early 2011, and must be replaced to facilitate continued growth and economic development. Failure to expand the ponds or provide another means of wastewater treatment would result in an unauthorized discharge, potential environmental damage to the surrounding area, and State and Federal fines. This Facility Plan provides Harrisburg with a planning guide for the safe and economical treatment of the City's wastewater through 2031.

B. EXISTING FACILITIES

A 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, they are projected to be full in 2011.

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. The future population projection was made by evaluating the recent trends in building permits for new homes in Harrisburg. The population projection through year 2031 is shown in Table I-1.

Table I-1: Annual Projected Population for Harrisburg, SD

Year	Projected Population
2006	3,355
2007	3,758
2008	4,209
2009	4,714
2010	5,280
2011	5,808
2012	6,389
2013	7,027
2014	7,730
2015	8,503
2016	9,353
2017	10,102
2018	10,910
2019	11,783
2020	12,725
2021	13,743
2022	14,568
2023	15,442
2024	16,368
2025	17,351
2026	18,392
2027	19,495
2028	20,665
2029	21,905
2030	23,219
2031	24,612

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 flows for year 2021 and 2031 are summarized in Table I-2.

Table I-2: Projected Influent Flows

Condition	Design Year	
	2021	2031
ADW, mgd	1.03	1.84
AWW, mgd	1.37	2.45
MWW, mgd	2.74	4.90
PHWW, mgd	3.86	6.29

Influent loading conditions for BOD_5, TSS, ammonia, and Total Kjeldahl Nitrogen (TKN) are presented in Table I-3. They were calculated using projected domestic populations and typical per capita loading rates of 0.20 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 the report.

Table I-3: Projected Influent Loadings

Condition	Design Year	
	2021	2031
BOD_5 Average, ppd	2,738	4,904
BOD_5 Max, ppd	3,632	6,505
TSS Average, ppd	3,012	5,394
TSS Max, ppd	3,856	6,905
NH_3-N Average, ppd	336	602
NH_3-N Max, ppd	388	695
TKN Average, ppd	517	925
TKN Max, ppd	597	1,070

D. PROJECT OPTIONS

The Facility Plan evaluated the following five (5) options for Harrisburg's future wastewater treatment. For Options 4 and 5, a phased approach was used with the infrastructure needed for years 2011 to 2021 constructed first. Additional equipment and structures would be added in 2021 to provide treatment capacity through year 2031. The opinion of probable construction costs, and operation and maintenance (O&M) cost for the 2011 and 2021 phases, and overall probable project present worth costs are provided in Table I-4. The costs assume a 20% contingency due to the preliminary nature of the project. Engineering, construction administration, and legal fees are expected to be 20% of the overall project cost. All costs are in 2007 dollars.

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 discharging the effluent. It is expected that the 30-day average discharge limit for ammonia would be 1.0 mg/l. Aerated lagoons in cold climates, such as Harrisburg's, are not capable of this level of ammonia removal. Lagoon covers and other treatment options will not aid in meeting this low, year-round ammonia discharge limit. Since this treatment method cannot meet the estimated discharge permit, it will not be considered.

4. Option 4: New Mechanical Wastewater Treatment Plant

   Construction of a new mechanical WWTP would require discharge of the effluent. Harrisburg's nearest waterway, Ninemile Creek, discharges into Lake Alvin. Lake Alvin is a protected watershed and treated wastewater discharge is not allowed into its tributaries within ten (10) miles of the Lake. This forces Harrisburg to look at discharging into the Big Sioux River approximately five (5) miles east of the City.

   This option evaluates the gravity interceptor, force main, equalization basin, 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 located just east of the 2025 growth area along Ninemile Creek. 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 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

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 gravity sewer interceptor, equalization basin, lift station and approximately ten (10) 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 in 2012 or 2013, 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.

The Sioux Falls MBR plant cannot tolerate rapid changes to influent flows. As a result, a 15-acre equalization basin is proposed at the lift station 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 ponds will likely need to be aerated to reduce odors; however, it is expected that odor conditions would develop in the basins and affect residents in the area.

Table I-4: Summary of Opinion of Probable Construction and O&M Costs

Treatment Process	Alternatives			
	New Harrisburg WWTP			Pump to City of Sioux Falls
	SBR	Conventional AS	MBR	
2011 Capital Construction Costs	$ 29,739,000	$ 32,007,000	$ 37,367,000	$ 25,356,000
2021 Capital Construction Costs	$ 10,146,000	$ 7,849,000	$ 14,651,000	$ 1,320,000
2011-2021 Annual O&M Costs	$ 375,000	$ 411,000	$ 414,000	$ 521,000
2022-2031 Annual O&M Costs	$ 398,000	$ 482,000	$ 491,000	$ 806,000
Present Worth Project Costs	$ 45,661,000	$ 46,859,000	$ 58,841,000	$ 40,813,000

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 a new SBR WWTP. While pumping wastewater to Sioux Falls for treatment is less costly, it also raises several concerns regarding the operation of the lift station pumps, the force main distance, and the potential siting problems and odors at the equalization basin. The SBR WWTP is the next lowest cost alternative. The annual O&M costs for operating the SBR WWTP are less than the O&M costs for sending wastewater for Sioux Falls for treatment. As a result, it may be the lowest cost option when looking beyond the 20-year planning period used in this report. In addition, the design of the lift station pumps will be easier to operate and maintain compared with the Sioux Falls option.

The SBR WWTP alternative would allow for Harrisburg to easily expand its capacity in the future, and it has the potential to serve a larger area via gravity. It also would be able to produce a high quality effluent with the potential for future chemical phosphorus removal.

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 WWTP treats 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 WWTP. The current industries are considered "dry" industries, with their waste streams consisting mainly of sanitary flows from facility restrooms.

The City's first WWTP 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 flow measuring manhole and a series of three total containment ponds. Harrisburg is restricted from discharging treated wastewater to the protected Ninemile Creek, which flows into the protected Lake Alvin approximately three miles downstream. The current WWTP has a design average daily flow of 0.133 million gallons per day (mgd), max daily flow of 0.331 mgd, and an average biochemical oxygen demand (BOD_5) loading capacity of 275 pounds per day (ppd). Harrisburg's existing evaporation ponds are projected to reach capacity in early 2011.

Residential housing encompasses the current WWTP 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 designing an expansion and/or replacement of their existing WWTP that will meet proposed effluent limits, and current solids handling and disposal regulations. This facility plan addresses these needs based on projected loadings to the year 2031. This is equivalent to a 20-year planning period, from when improvements will be implemented or constructed in 2011.

The Facility Plan analyzes various treatment alternatives and develops opinions of probable cost for each alternative. Total capital costs, which include construction costs, engineering, administration and legal costs, have been developed for each alternative. The total present worth value incorporates capital costs, annual operation and maintenance (O&M) costs inflated over a 20-year period. All costs provided in this report are in 2007 dollars. Actual costs at the time they occur will have to be inflated from these estimates.

III. ENVIRONMENTAL REVIEW (to be finished when sites selected)

A. ENVIRONMENTAL INFORMATION

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B. HISTORICAL AND ARCHAEOLOGICAL SITES

Write Section Here

C. FLOODPLAINS AND WETLANDS

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D. AGRICULTURAL LANDS

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E. WILD AND SCENIC RIVERS

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F. FISH AND WILDLIFE PROTECTION

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G. WATER QUALITY AND QUANTITY

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H. DIRECT AND INDIRECT IMPACTS

Indirect impacts will occur throughout the City of Harrisburg as a result of the proposed improvements to construct t a WWTP. 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.

I. MITIGATING ADVERSE IMPACTS

Though we do not anticipate adverse impacts, we will contact the proper authorities if they arise.

IV. EXISTING AND FUTURE CONDITIONS

A. PROJECT NEED AND PLANNING AREA IDENTIFICATION

The existing zoning map is shown in 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 and west of the Industrial Park, and also 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 in 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 planned one-half mile to the west of the City on land that has not yet been annexed. 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, but Harrisburg's supervisory control and data acquisition (SCADA) system is not designed to store the data. As a result, Harrisburg has been collecting and recording periodic flow readings to determine current influent flows. The City may want to invest in equipment to begin monitoring and recording flows on a daily or more frequent basis to determine peak day and peak hour conditions.

2. Organic Load

Limited historical influent wastewater quality data was available for review. Thus, sampling and analysis was performed to obtain data on the current influent wastewater characteristics. City personnel and HRG staff completed the sampling. Two independent certified laboratories, South Dakota State University 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-mililiter (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-1 and IV-2. The City may want to consider periodic sampling and testing to verify the assumptions made from this limited data set.

Table IV-1: 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-2: 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

C. EVALUATION OF TREATMENT SYSTEMS

1. Existing Wastewater Treatment Plant

The existing WWTP site 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 also shows the residential areas directly north and east of the WWTP 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-3 and were obtained from the original construction plans.

Table IV-3: 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
Volume, MG	15.8	18.7	48.4

The City of Harrisburg operates their containment lagoons in series operation 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 where it is allowed to fill to the three (3)-foot level before opening the valve between the primary and secondary cells and allowing the water levels to equalize. Once the water levels have equalized, the valve between the primary and secondary cells is closed.

b. After the valve is closed, the City allows Primary Cell No. 1 to rise to approximately 3.5-feet. The City then opens the valve between Cells No. 2 and No. 3 until the water levels equalize. Once the water levels have equalized, the valve is closed between Cells No. 2 and No. 3, and the valve between Cells No. 1 and No. 2 is opened. Once these water levels have equalized, the valve between Cells No. 1 and No. 2 is closed.

c. The process is repeated at 0.5-foot increments until the water levels approach the high water level (HWL). Once the cells approach their HWL, transfers will need to 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 Plant 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-4.

Table IV-4: 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

Parameter	Unit	Value
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 Plant Capacity

During the summer of 2003, the first cell contained approximately 2-feet of liquid, the second cell 1-foot of liquid, and the third cell was dry. In the summer of 2004, the first two cells contained roughly 2-feet of liquid, and the third cell was dry. During the summer of 2005, the first cell contained approximately 2-feet of liquid, the second cell approximately 6-inches, and the third cell was dry. As of December 6, 2006, the first cell contained 3'-6" of liquid, the second cell contained 3'-6", and the third cell contained 1'-6" of liquid.

Due to the recent population growth, the wastewater ponds are expected to reach their capacity sooner than their design life of 2017. The remaining life of the existing lagoon system was projected based on current population projections. 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 0.06-inches 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 mid-depth location. Therefore, the total surface area used for calculation purposes is 1,647,562-square feet (sf), or 37.82-acres. Based on the equation, the volume of water evaporated per year is estimated at 5,106,412-cubic feet per year (38.196 million gallons). Once the yearly flow increases above this rate (104,650 gpd), there will be a net accumulation, and the lagoons will begin to fill up. Based on estimated billed water usage, the City of Harrisburg's average day flow rate likely surpassed 104,650 gpd in 2005.

The remaining capacity in the total containment ponds was calculated based on two flow rates. First, the population projections previously stated and the average day flow rate of 75 gallons per capita per day (gpcd) from the design of the total containment ponds was used. The calculation based on these assumptions showed that the lagoons will reach their terminal capacity sometime in the year 2008. Current water elevations in the lagoon indicate that they will not be full next year. Records kept by the City of Harrisburg document that the actual average day flow rate is less than 75 gpcd. The City of Harrisburg completed lagoon monitoring reports in 2000, 2001, and 2003, and these reports suggest that the average day flow rate varies from 48 to 62 gpcd. Therefore, an average flow rate of 54 gpcd was used and indicated that the capacity of the existing lagoons will be reached sometime in late 2010 or early 2011. The lagoon monitoring reports are provided in Tables C-1, C-2 and C-3 in Appendix C for reference. The lagoon capacity calculations are provided in Table C-4 in Appendix C.

4. Existing Collection System

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.

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 WWTP discharging into the Big Sioux River south of Ninemile Creek would have effluent limitations similar to those presented in Table IV-5.

Table IV-5: Preliminary Discharge Limits for the City of Harrisburg

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

E. FUTURE CONDITIONS

1. Population and Land Use Projections

The City of Harrisburg has experienced an explosive increase in population over the past six years. Until this recent surge, the population in Harrisburg had remained fairly steady. Table IV-6 lists the historical population based on census data for the past 40-years.

Table IV-6: 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. In recent years, the City of Harrisburg has experienced an average population increase of close to 30% per year based on the number of building permits issued and an assumption of three people per household.

Table IV-7 shows the number of annual building permits issued and the City's estimated population during the past six years. As of September 4, 2007, the City had issued 97 building permits.

Table IV-7: Annual Building Permits and Estimated Population

Year	Building Permits Issued	Population	Percent Increase
1999	12		
2000	11	991	
2001	14	1,034	4.3%
2002	34	1,137	10.0%
2003	115	1,487	30.7%
2004	144	1,925	29.4%
2005	198	2,527	31.3%
2006**	*295*	3,355	32.8%
2007***	*97*	3,648	8.7%

**NOTE: Use 31% increase trend shown from 2003 to 2005.*

***NOTE: 2006 projected population based on a density of 3.04 for single-family housing, and 2.5 for apartment unit housing.*
****Permits as of September 4, 2006.*

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 31% for the next 10 to 20 years. As the population increases, the annual percent increase will decline. In addition, economic factors can greatly affect population increases. Therefore, Harrisburg's population is expected to increase 12% from 2006 to 2010, 10% from 2011 to 2016, 8% from 2017 to 2021, and 6% from 2022 to 2031.

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 2031 could reach 24,520. Figure IV-3 shows the population projection and Table IV-8 lists the projected population of Harrisburg for years 2006 to 2031.

Figure IV-3: Population Projection for Harrisburg, SD

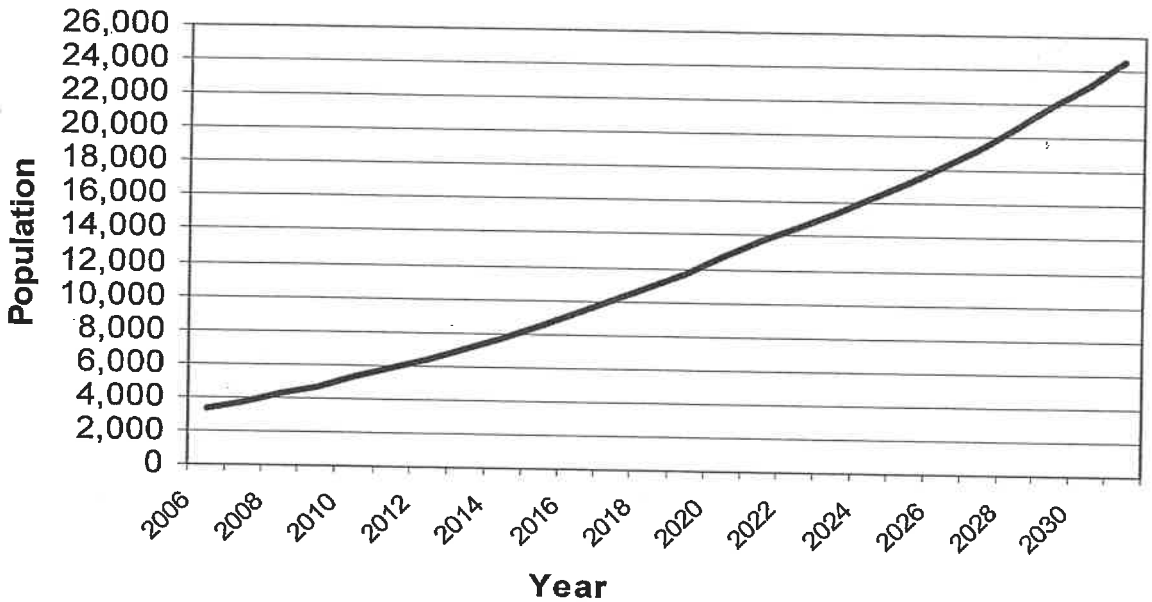


Table IV-8: Annual Projected Population for Harrisburg, SD

Year	Projected Population
2006	3,355
2007	3,758
2008	4,209
2009	4,714
2010	5,280
2011	5,808
2012	6,389
2013	7,027
2014	7,730
2015	8,503
2016	9,353
2017	10,102
2018	10,910
2019	11,783
2020	12,725
2021	13,743
2022	14,568
2023	15,442
2024	16,368
2025	17,351
2026	18,392
2027	19,495
2028	20,665
2029	21,905
2030	23,219
2031	24,612

Assuming 2.5 homes per acre and 3 people per home, the expected increased from the 2006 population to the 2031 population will require approximately 2,825 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 Wasteloads

Typically, the planning period for a WWTP improvements project is 20 years. In this study, projected design flows and loadings have been established for two distinct, 10-year design periods given the uncertainty of the current population growth rate. This phased approach minimizes risk and overall project cost associated with a 20-year planning period design based on highly uncertain population growth projections. The two design periods will be assumed to have time periods of 2011-2021 and 2021-2031.

a. Hydraulic Load

Projected influent flows are summarized in Table IV-9. Flows are based on projected populations for each design year. 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. A summary of annual flow projections and the peaking factors is provided in Table D-1 in Appendix D.

Table IV-9: Projected Influent Flows

Condition	Design Year	
	2021	2031
ADW, mgd	1.03	1.84
AWW, mgd	1.37	2.45
MWW, mgd	2.74	4.90
PHWW, mgd	3.86	6.29

b. Organic Load

Projected influent loadings are summarized in Table IV-10. Influent loading conditions for BOD_5, TSS, ammonia, and Total Kjeldahl Nitrogen (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 for either design period.

Table IV-10: Projected Influent Loadings

Condition	Design Year	
	2021	2031
BOD_5 Average, ppd	2,738	4,904
BOD_5 Max, ppd	3,632	6,505
TSS Average, ppd	3,012	5,394
TSS Max, ppd	3,856	6,905
NH_3-N Average, ppd	336	602
NH_3-N Max, ppd	388	695
TKN Average, ppd	517	925
TKN Max, ppd	597	1,070

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 could inventory homes again 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 full in the next three to four years, Harrisburg realizes that it must begin evaluating future options for wastewater treatment and disposal.

1. No Action

One option that all municipalities have is the "No Action" alternative. Future population projections indicate that the existing containment ponds will reach capacity in late 2010 or early 2011. 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 unauthorized discharges or overtop their existing evaporation ponds. 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 for the 20-year design period is significant. Approximately 451 acres are required in addition to the existing plant. The calculation for the future evaporation ponds is provided in Table E-1 in Appendix E. Due to the required land area and siting constraints, this option is neither feasible nor desirable and will not be discussed further in the 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.

Aerated lagoons can be used in warmer climates for year-round ammonia removal if lagoon temperatures can be maintained above 10 degrees C. Recently, lagoon covers are being used to maintain wastewater temperatures to allow aerated lagoons to be utilized in cooler climates. Other alternatives for additional nitrification are add-on processes, such as rotating biological contactors (RBCs), submerged gravel filter, and submerged fixed growth processes.

Design requirements are available for these types of systems, but are usually dictated by the regulating authority. Other requirements to consider are the location of groundwater, soil amenability for lining, and other siting requirements.

Advantages

- Solids treatment is generally not required. Periodic dredging and land application are sufficient
- Relatively low aeration requirements
- Low operator attention
- Low cost alternative depending on soil conditions, land prices, and effluent requirements

Disadvantages

- Low level of treatment without substantial design changes
- Long residence time in settling cells can increase the algal (TSS) content of the effluent, which may affect downstream disinfection
- Significant land requirement

The City of Harrisburg could convert their existing total containment ponds into aerated lagoons. It would require excavating each of the existing cells deeper to meet the minimum depth requirement of ten feet per the SD DENR, expanding the lagoons to create additional surface area, creating a quiescent settling cell, and installing surface aerators. The aerated ponds would require discharge. Harrisburg's nearest waterway, Ninemile Creek, discharges into Lake Alvin. Lake Alvin is a protected watershed and treated wastewater discharge is not allowed into its tributaries within 10 miles of the Lake. This forces Harrisburg to look at discharging into the Big Sioux River. Discussions with the SD DENR have indicated that the 30-day average discharge limit for ammonia would be 1.0 mg/l. Aerated lagoons in cold climates, such as Harrisburg's, are not capable of this level of ammonia removal. Lagoon covers and other treatment options will not aid in meeting this low, year-round ammonia discharge limit. Since this treatment method cannot meet the estimated discharge permit, it will not be considered.

4. New Mechanical Wastewater Treatment Plant

A mechanical WWTP will be needed to meet Harrisburg's estimated discharge permit. Ninemile Creek is located just south of Harrisburg, and flows by gravity to Lake Alvin and the Big Sioux River. However, the WWTP effluent would need to discharge into the Big Sioux River, since Lake Alvin is a protected waterway and no WWTP within ten miles can discharge into one of its tributaries.

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 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 2031 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 2031 maximum day 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 (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.

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 and sized to handle PHWW flows. The wastewater would be directed to either the pumps or an equalization basin after the screening process.

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:
   - Easy to perform routine maintenance on pumps and valves and detect small problems early before they become large problems
   - Several pump drive configurations can be used
   - 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 will be 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 will 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 headwords 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 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 chainand-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

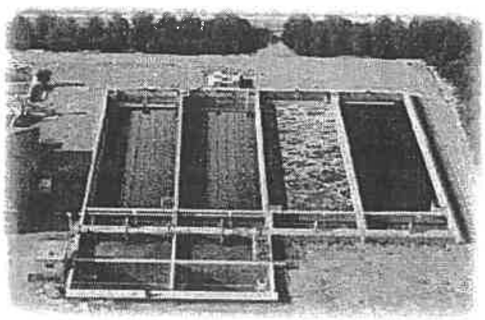
As described above, it is uncertain if the current population growth trend will remain constant over the design period. Therefore, secondary treatment systems will initially be sized for the 10-year design period (2011-2021) flows and loadings. After ten years, additional capacity can be added to accommodate 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.

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.



ICEAS SBR System

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:

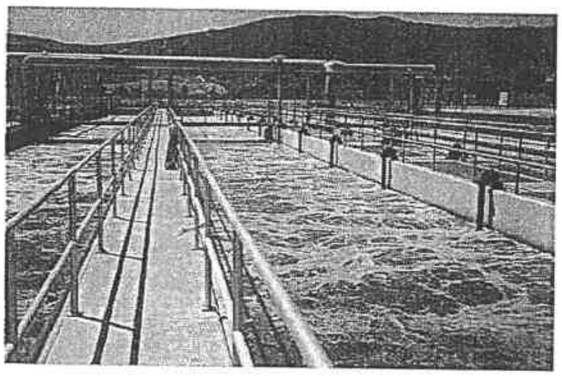
- Proprietary technology
- May require greater degree of operator control than a conventional activated-sludge system
- Additional operating costs required for aeration equipment
- Scum handling may be required
- Moving parts on decanter may be subject to freezing or malfunction
- Digestion facilities 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.



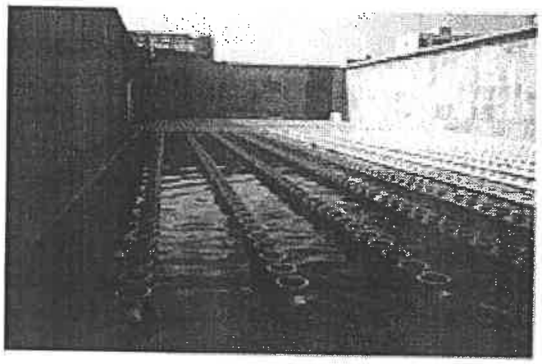
Plug flow activated sludge system

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

Parameter		**Unit**	**Design Year**	
			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 re listed below:

Advantages:

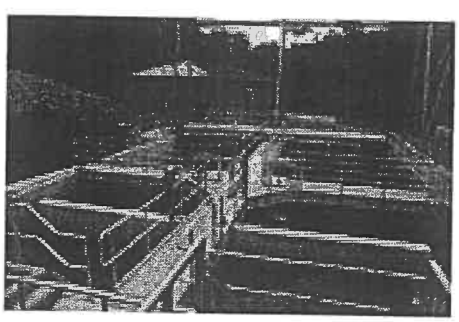
- Non-proprietary technology
- Thousands of installations with proven technology
- Well understood process that is simple to operate
- Process is flexible and will accommodate future expansion
- Risk of downtime spread to multiple process tanks

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)

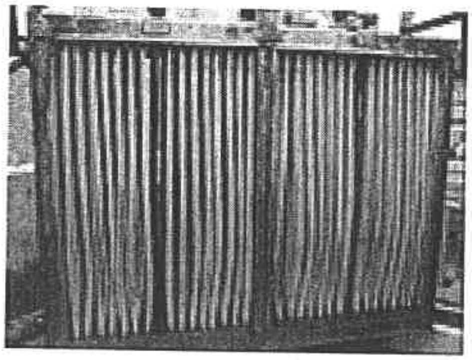
The MBR system uses a combination of a suspended growth activated-sludge system and an immersed, low pressure ultrafiltration membrane system. The suspended growth activated-sludge system is a conventional activated-sludge system as described above. The ultrafiltration membrane system is located downstream 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.



MBR membrane cassette

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.

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:

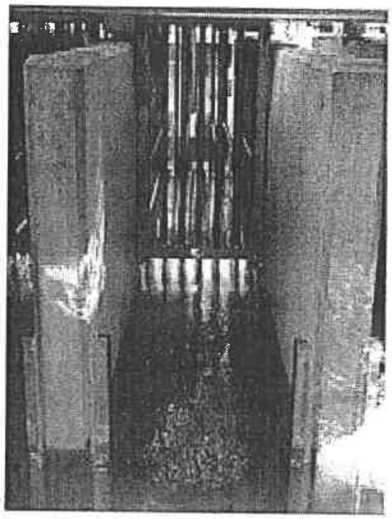
- Reduced potential health effects

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



Horizontal UV system in a concrete channel

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.

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 place 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.

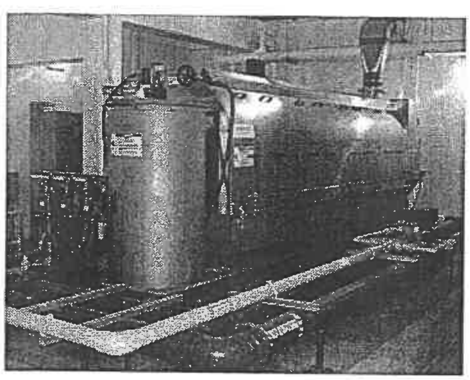
Thickening of the WAS from the secondary treatment process will be needed to reduce the volume of the wasted sludge prior to digestion. This will also reduce the required digester volume. A WAS holding tank shall be provided ahead of the thickening process to allow for continuous wasting from the new 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 the gravity sewer piping, equalization basin, and lift station infrastructure discussed in Sections V.A.4.a, b, and c of this report.

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 2012 or 2013, 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 similar to the one needed for Harrisburg's MBR WWTP is proposed at the lift station discussed in Section V.A.4.b. The equalization basin will provide 14 days of storage at 2031 flows based on AWW conditions.

The equalization basin will lessen the peak flows sent to Sioux Falls for treatment. It 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 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 1,425 gpm at 175 feet total dynamic head (TDH) for the 2021 Design year, and 2,550 gpm at 360 feet TDH for the 2031 Design year.

Due to the high head conditions expected, the lift station will 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 the 2021 MWW flow has been reached, all four (4) pumps will be replaced with four (4) new pumps (1 train duty operation and 1 train standby operation) with each train capable of handling the 2031 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 51,700-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 will have to be high pressure DIP until the pressures drop to allow for the safe use of PVC.

At the time Sioux Falls constructed 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 the 2021 design year are shown in Table V-10 and are in 2007 dollars. Project capital costs to increase capacity for the 2031 design year are shown in Table V-11 and are also in 2007 dollars. A breakdown of the capital construction costs are provided in Appendix F.

Table V-10: Probable Capital Construction Cost Summary – 2021 Design Year

Treatment Process	Alternatives			
	New Harrisburg WWTP			Pump to City of Sioux Falls
	SBR	**Conventional AS**	**MBR**	
Gravity Sanitary Sewer Interceptor	$ 5,084,100	$ 5,084,100	$ 5,084,100	$ 5,084,100
Influent Lift Station	$ 2,676,200	$ 2,676,200	$ 2,676,200	$ 3,280,400
Equalization Basin	$ 205,000	$ 205,000	$ 2,307,600	$ 2,307,600
Force Main to Harrisburg WWTP	$ 2,369,600	$ 2,369,600	$ 2,369,600	---
Force Main to LS #240	---	---	---	$ 4,181,000
Force Main from LS#240 to Future SF WWTP	---	---	---	$ 537,300
Preliminary Treatment	$ 776,800	$ 776,800	$ 985,800	---
Primary Treatment	---	---	---	---
Secondary Treatment	$ 2,204,350	$ 3,560,100	$ 4,151,800	---
Disinfection Treatment	$ 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	---
Control Building	$ 260,000	$ 260,000	$ 260,000	---
WWTP Sitework	$ 622,400	$ 751,300	$ 831,300	---
WWTP Outfall	$ 715,600	$ 715,600	$ 715,600	---
Land Acquisition	$ 690,000	$ 690,000	$ 1,057,500	$ 457,500
Mobilization	$ 2,065,000	$ 2,222,000	$ 2,594,000	$ 1,760,000
Subtotal Construction Costs	**$ 20,651,750**	**$ 22,226,400**	**$ 25,949,200**	**$ 17,607,900**
Contingency (20%)	$ 4,130,350	$ 4,445,280	$ 5,189,840	$ 3,521,580
Preliminary Opinion of Construction Costs	$ 24,782,100	$ 26,671,680	$ 31,139,040	$ 21,129,480
Engineering, Legal, Construction Administration (20%)	$ 4,956,420	$ 5,334,336	$ 6,227,808	$ 4,225,896
Total Engineer's Opinion of Probable Project Construction Cost	**$ 29,738,520**	**$ 32,006,016**	**$ 37,366,848**	**$ 25,355,376**

Table V-11: Probable Capital Construction Cost Summary – 2031 Design Year

Treatment Process	Alternatives			
	New Harrisburg WWTP			Pump to City of Sioux Falls
	SBR	Conventional AS	MBR	
Gravity Sanitary Sewer Interceptor	---	---	---	---
Influent Lift Station	$ 536,500	$ 536,500	$ 536,500	$ 824,500
Equalization Basin	---	---	---	---
Force Main to Harrisburg WWTP	---	---	---	---
Force Main to LS #240	---	---	---	---
Force Main from LS#240 to Future SF WWTP	---	---	---	---
Preliminary Treatment	$ 381,500	$ 381,500	$ 592,500	---
Primary Treatment	$ 1,568,600	$ 1,568,600	$ 1,568,600	---
Secondary Treatment	$ 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	---
Control Building	---	---	---	---
WWTP Sitework	$ 527,700	$ 397,200	$ 783,700	---
WWTP Outfall	---	---	---	---
Land Acquisition	---	---	---	---
Mobilization	$ 704,600	$ 545,000	$ 1,017,000	$ 91,600
Subtotal Construction Costs	**$ 7,045,700**	**$ 5,450,400**	**$ 10,173,800**	**$ 916,100**
Contingency (20%)	$ 1,409,140	$ 1,090,080	$ 2,034,760	$ 183,220
Preliminary Opinion of Construction Costs	$ 8,454,840	$ 6,540,480	$ 12,208,560	$ 1,099,320
Engineering, Legal, Construction Administration (20%)	$ 1,690,968	$ 1,308,096	$ 2,441,712	$ 219,864
Total Engineer's Opinion of Probable Project Construction Cost	**$ 10,145,808**	**$ 7,848,576**	**$ 14,650,272**	**$ 1,319,184**

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 $0.05 per kW-hr.

Additional labor will be required for daily operational and maintenance needs for the recommended treatment improvements. Overall labor costs are expected to increase as a result of WWTP improvements from the current labor effort and cost to maintain the facility. 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 (in 2007 dollars). 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.

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 in 2007 dollars. Annual O&M costs for 2021 - 2031 are shown in Table V-13 in 2007 dollars. O&M costs have been calculated from current costs assuming a 3.0 percent inflation rate and a 4.75 percent interest rate. 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 $1.93/1,000 gallons of wastewater received. Sioux Falls has been evaluating this rate and indicated that it will increase approximately 10% for the next four years, 6% the following year, and 3% annually for each of the following years. This information was used to prepare the present worth O&M cost for pumping to Sioux Falls. The calculation was based on six months of AWW flow and six months 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 – 2021 Design Year

	Alternatives			
	New Harrisburg WWTP			Pump to City of Sioux Falls
Treatment Process	SBR	Conventional AS	MBR	
Gravity Sanitary Sewer Interceptor	---	---	---	---
Influent Lift Station	$ 56,300	$ 56,300	$ 56,300	$ 63,000
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	---
Primary Treatment	---	---	---	---
Secondary Treatment	$ 116,000	$ 160,630	$ 164,150	---
Disinfection Treatment	$ 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	$ 71,200	$ 71,200	$ 71,200	---
Sioux Falls Treatment of Wastewater	---	---	---	$ 457,355
Total	**$ 374,600**	**$ 410,130**	**$ 413,650**	**$ 520,355**

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

Table V-13: Probable Project Annual Total O&M Costs[1] Summary – 2031 Design Year

Treatment Process	Alternatives			
	New Harrisburg WWTP			Pump to City of Sioux Falls
	SBR	Conventional AS	MBR	
Gravity Sanitary Sewer Interceptor	---	---	---	---
Influent Lift Station	$ 71,100	$ 71,100	$ 71,100	$ 93,500
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	---
Primary Treatment	$ 7,360	$ 7,360	$ 7,360	---
Secondary Treatment	$ 213,000	$ 293,700	$ 302,540	---
Disinfection Treatment	$ 18,930	$ 22,240	$ 22,240	---
Solids Digestion	$ 17,550	$ 17,550	$ 17,550	---
Solids Thickening/Dewatering	Incl. w/Solids Digestion	Incl. w/Solids Digestion	Incl. w/Solids Digestion	---
Solids Disposal	$ 60,600	$ 60,600	$ 60,600	---
Sioux Falls Treatment of Wastewater	---	---	---	$ 712,418
Total	**$ 397,440**	**$ 481,450**	**$ 490,290**	**$ 805,918**

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

3. Present Worth Analysis

The present worth of all costs, including 2021 and 2031 capital construction costs, and O&M costs were calculated using 2007 as the present year. Present worth costs are shown in Table V-14.

Table V-14: Probable Project Present Worth Costs Summary – 2007 Present Year

Treatment Process	Alternative			
	New Harrisburg WWTP			Pump to City of Sioux Falls
	SBR	Conventional AS	MBR	
Gravity Sanitary Sewer Interceptor				
Capital Present Worth	$ 5,084,100	$ 5,084,100	$ 5,084,100	$ 5,084,100
O&M Present Worth	$ -	$ -	$ -	$ -
Influent Lift Station				
Capital Present Worth	$ 3,212,700	$ 3,212,700	$ 3,212,700	$ 4,104,900
O&M Present Worth	$ 1,045,500	$ 1,045,500	$ 1,045,500	$ 1,270,400
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	$ 2,369,600	$ 2,369,600	$ 2,369,600	$ -
O&M Present Worth	$ -	$ -	$ -	$ -
Force Main to LS #240				
Capital Present Worth	$ -	$ -	$ -	$ 4,181,000
O&M Present Worth	$ -	$ -	$ -	$ -
Force Main from LS #240 to SF WWTP				
Capital Present Worth	$ -	$ -	$ -	$ 537,300
O&M Present Worth	$ -	$ -	$ -	$ -
Sioux Falls Treatment of Wastewater				
Capital Present Worth	$ -	$ -	$ -	$ -
O&M Present Worth	$ -	$ -	$ -	$ 12,867,500
Preliminary Treatment				
Capital Present Worth	$ 1,158,300	$ 1,158,300	$ 1,578,300	$ -
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	$ 4,107,550	$ 4,091,100	$ 8,336,700	$ -
O&M Present Worth	$ 2,333,700	$ 3,605,500	$ 3,424,900	$ -
Disinfection Treatment				
Capital Present Worth	$ 260,500	$ 260,500	$ 260,500	$ -
O&M Present Worth	$ 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	Incl. w/Solids Digestion	Incl. w/Solids Digestion	Incl. w/Solids Digestion	$ -
Solids Disposal				
Capital Present Worth	$ -	$ -	$ -	$ -
O&M Present Worth	$ 1,038,700	$ 1,038,700	$ 1,038,700	$ -
Control Building				
Capital Present Worth	$ 260,000	$ 260,000	$ 260,000	$ -
O&M Present Worth	$ -	$ -	$ -	$ -

Treatment Process	Alternative			
	New Harrisburg WWTP			Pump to City of Sioux Falls
	SBR	Conventional AS	MBR	
WWTP Sitework				
Capital Present Worth	$ 1,150,100	$1,148,500	$ 1,615,000	$ -
O&M Present Worth	$ -	$ -	$ -	$ -
WWTP Outfall				
Capital Present Worth	$ 715,600	$ 715,600	$ 715,600	$ -
O&M Present Worth	$ -	$ -	$ -	$ -
Land Acquisition				
Capital Present Worth	$ 690,000	$ 690,000	$ 1,057,500	$ 457,500
O&M Present Worth	$ -	$ -	$ -	$ -
Mobilization				
Capital Present Worth	$ 2,769,600	$ 2,767,000	$ 3,611,000	$ 1,851,600
O&M Present Worth	$ -	$ -	$ -	$ -
Subtotal Construction Costs	$ 27,697,450	$ 27,676,800	$ 36,123,000	$ 18,524,000
Contingency (20%)	$ 5,539,490	$ 5,535,360	$ 7,224,600	$ 3,704,800
Preliminary Opinion of Construction Costs	$ 33,236,940	$ 33,212,160	$ 43,347,600	$ 22,228,800
Engineering, Legal, Construction Administration (20%)	$ 6,647,388	$ 6,642,432	$ 8,669,520	$ 4,445,760
Total Present Worth Probable Project Construction Cost	$ 39,884,328	$ 39,854,592	$ 52,017,120	$ 26,674,560
O&M Present Worth	$ 5,776,300	$ 7,004,100	$ 6,823,500	$ 14,137,900
Overall Present Worth Engineer's Opinion of Probable Cost	**$ 45,660,628**	**$ 46,858,692**	**$ 58,840,620**	**$ 40,812,460**

C. DEMONSTRATION OF FINANCIAL CAPABILITY

The City of Harrisburg will work closely with the Southeast Council of Governments (SECOG) 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, federal appropriations, and other potential federal funding options.

A preliminary cash flow analysis and amortization table is included in Appendix H. This analysis considers a $29,525,404 project debt repayment from the Clean Water SRF for the SBR WWTP alternative. Ideally, user fees would fund the sanitary department operating budget and project debt repayment.

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 for the project of would be $2,030,724 to repay the Clean Water SRF loan at an interest rate of 3.25%. This would require rates to increase 30% for years 2008 to 2012, and 25% for year 2013, and 2% for year 2014 to 2015. Projections were not made beyond 2015. This would result in rates increasing to $48.05 for the monthly customer charge and $0.96 per 100 gallons for the volume charge in 2015. As a result, the City will be seeking Federal and State assistance to fund the project.

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 fill earlier than expected as a result of the recent growth. The City must developed and implement a new treatment alternative prior to the lagoons reaching capacity to prevent environmental damage from an overflow.

Each option considered was evaluated for its environmental impact.

1. Sequential Batch Reactor (SBR) WWTP

The construction of a new SBR WWTP would require taking directly 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 surrounding 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.

2. 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.

3. Membrane Bioreactor (MBR) WWTP

The selection of the MBR WWTP alternative would require taking directly 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 surrounding 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.

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

Selection of the alternative to pump wastewater to Sioux Falls for treatment would require taking 30.5 acres of agricultural land out of service for the lift station and equalization basin.

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.

The 15 acre equalization basin at the lift station site would likely have 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 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. Sequential Batch Reactor (SBR) WWTP

One advantage of the SBR WWTP alternative is the cost. While this alternative is not the lowest cost option of those considered; it is the lowest cost option of the three alternatives evaluated for Harrisburg to construct its own WWTP. The SBR WWTP alternative may be the lowest cost option when looking beyond the 20-year planning period used in this report. The annual O&M costs for operating the SBR WWTP are less than the O&M costs for sending wastewater for Sioux Falls for treatment. The SBR alternative would be the lowest cost option if the costs were extended further than 20 years. As a result, this option has the potential to be the lowest cost alternative.

In addition, the design of the lift station and the accompanying force main allows use of submersible pumps in parallel instead of series as required with the pumping to Sioux Falls option. The pumping industry prefers these pumps be operated in parallel due to the maintenance concerns that can develop with series operation.

The SBR WWTP alternative would allow for Harrisburg to easily expand its capacity in the future. Harrisburg has the potential to serve a larger area via gravity with the construction of it's own WWTP. The timing of this expansion would not need to be timed or coordinated with the City of Sioux Falls. Given the recent rapid growth of Harrisburg, and the level of uncertainty with future population projections, this would be advantageous.

The SBR WWTP would be able to produce a high quality effluent with the potential for future chemical phosphorus removal.

The main disadvantage of the SBR WWTP alternative is that is has a higher upfront capital cost than pumping to Sioux Falls for wastewater treatment.

2. Conventional Activated-Sludge WWTP

The conventional activated-sludge WWTP alternative advantages and disadvantages are very similar to those of the SBR WWTP alternative with one main exception: cost. The upfront capital and annual O&M costs are higher for the conventional activated-sludge WWTP alternative.

3. Membrane Bioreactor (MBR) WWTP

Advantages and disadvantages of the MBR WWTP alternative were considered and are summarized below.

One significant advantage and disadvantage of the MBR WWTP alternative is the high quality effluent the technology can produce. The MBR technology has the ability to produce a higher quality effluent than the other options considered. However, unless the City requires a high quality effluent for reuse or other needs, the cost of this option may outweigh the benefit. This option also has the potential for future phosphorus removal.

The greatest disadvantage of the MBR WWTP alternative is its capital construction cost. It is the highest of all the alternatives considered. In addition, the O&M costs for this alternative are high. Much of this is due to the upfront and replacement costs of the membrane modules.

Another disadvantage of the MBR option is the amount of equalization needed ahead of the WWTP. The 15-acre equalization basin that would be located at the lift station would frequently need to contain raw wastewater during summer periods to reduce influent flows. It is expected that odor conditions would develop in the basins and affect area residents.

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

Pumping wastewater to Sioux Falls for treatment has two main advantages. It is the lowest capital cost alternative of all the options considered. In addition, the City of Harrisburg is not directly responsible for the staffing and equipment needed to operate and maintain a WWTP.

This alternative also has several significant disadvantages.

First, the force main distance required to convey the wastewater from Harrisburg to Lift Station #240, and eventually the new Sioux Falls WWTP is approximately 53,678 feet, or slightly over ten miles. The head conditions require placing the submersible 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.

Similar to the MBR option, there are concerns over the amount of equalization needed ahead of the WWTP. The 15 acre equalization basin that would be located at the lift station would frequently need to contain raw wastewater during summer periods to reduce influent flows. It is expected that odor conditions would develop in the basins and affect residents in the area.

G. VIEWS OF THE PUBLIC AND CONCERNED INTEREST GROUPS

A public hearing was held with proper notification during a regularly scheduled City Council meeting on XXXXX. The public comments were............... The Affidavit of Publication and public hearing meeting minutes are provided in Appendix J.

VI. SELECTED PLAN, DESCRIPTION AND IMPLEMENTATION ARRANGEMENTS

A. JUSTIFICATION AND DESCRIPTION OF SELECTED PLAN

Based upon extensive study of the options, we recommend Harrisburg proceed with the SBR WWTP alternative. It is not the lowest cost alternative during the 20-year planning period. However, due to the concerns regarding the operation of the lift station pumps and the force main distance, and the potential odors at the equalization basin, we recommend not proceeding with the option to pump wastewater to Sioux Falls for treatment. As a result, the SBR WWTP is the next lowest cost alternative, and has the potential to be the lowest cost option if greater than a 20-year design life is considered.

B. DESIGN OF SELECTED PLAN

Design and construction of the selected plan is critical to replacing Harrisburg's evaporation ponds before they reach capacity. 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 of the WWTP can begin. Sites for the proposed improvements must be obtained, and Lincoln County will need to authorize Conditional Land Use permits. Several public meetings will be needed to educate the public on the need and impacts of the proposed improvements. SD DENR approval must also be given for the project and discharge permit. It is only once these tasks and others are completed that Harrisburg can begin to work with its City Engineer on the design of the proposed improvements.

C. COST ESTIMATES FOR THE SELECTED PLAN

A summary of the Engineer's Opinion of Probable 2021Capital Construction Cost for the recommended SBR WWTP alternative is provided in Table VI-1.

Table VI-1: Engineer's Opinion of Probable 2021Capital Construction Cost for the Recommended SBR WWTP Alternative

Treatment Process	SBR WWTP Alternative
Gravity Sanitary Sewer Interceptor	$ 5,084,100
Influent Lift Station	$ 2,676,200
Equalization Basin	$ 205,000
Force Main to Harrisburg WWTP	$ 2,369,600
Force Main to LS #240	---
Force Main from LS#240 to Future SF WWTP	---
Preliminary Treatment	$ 776,800
Primary Treatment	---
Secondary Treatment	$ 2,204,350
Disinfection Treatment	$ 260,500
Solids Digestion	$ 1,232,900
Solids Thickening/Dewatering	$ 1,489,300
Control Building	$ 260,000
WWTP Sitework	$ 622,400
WWTP Outfall	$ 715,600
Land Acquisition	$ 690,000
Mobilization	$ 2,065,000
Subtotal Construction Costs	**$ 20,651,750**
Contingency (20%)	$ 4,130,350
Preliminary Opinion of Construction Costs	$ 24,782,100
Engineering, Legal, Construction Administration (20%)	$ 4,956,420
Total Engineer's Opinion of Probable Project Construction Cost	**$ 29,738,520**

D. USER RATE IMPACTS

The City of Harrisburg's current monthly sanitary sewer rates are $10.00 for the customer charge and $2.00 per 1,000 gallons for the usage charge. The City also charges a hook-up fee on all new construction building permits of $250.00.

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 30% rate increase would be required annually for the next five years to fund the proposed improvements. A 25% rate increase would be needed the sixth year. The rate increases could be reduced to 2% to 3% annually after that. The City would not be able to maintain the required coverage ratio during some of the years even with these substantial rate increases. Assuming these increases, the rate for 2014 would be $46.41 for the customer charge and $9.30 per 1,000 gallons for the usage charge. This rate will be a burden to the residents of Harrisburg, and State or Federal assistance will be required to fund a portion of the project.

The City anticipates annual rate increases beginning January 1, 2008 to begin preparing for the improvements. 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 construction of the new SBR WWTP. Most significantly, the new WWTP provides for the safe and proper treatment of Harrisburg's wastewater for many years to come.

Construction of the SBR WWTP will remove approximately 46.0 acres of 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 lift station and WWTP site would be minimized with available treatment technology and odor control equipment. Buffer property will also be used surrounding the WWTP site to minimize the impacts to adjacent property owners.

The environmental review of the selected sites indicates..............

F. ARRANGEMENTS FOR IMPLEMENTATION

The City understands that a project of this magnitude will require significant planning and coordination between the City, State and 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
   Intermunicipal service agreements will not be required for the selected plan.

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
   At the time a business or industry with a high strength waste establishes itself in Harrisburg, a pre-treatment program would be implemented.

G. LAND ACQUISITION

The City of Harrisburg will need to purchase land to construct the lift station, equalization basin, and WWTP. Approximately, one half acre will be needed for the lift station site for pumping wastewater to Harrisburg's WWTP. For the SBR WWTP and conventional activated-sludge WWTP alternatives, a 5.5 acre site will be required adjacent to the lift station site for construction of an equalization basin. For the MBR WWTP and pumping to Sioux Falls alternatives, a 30 acre site will be needed adjacent to the lift station site for construction of the larger equalization basin. Approximately 40 acres will be needed for construction of Harrisburg's own WWTP, future expansion area, and buffer. Sanitary sewer interceptors, force mains, and outfall will be located in public right-of-way or easements. The City may incur some costs for crop reimbursement in areas where easements need to be obtained. This report does not include the cost of crop reimbursement.

1. General Acquisition
   The City intends to obtain land through negotiation with area landowners since several potential areas could be used for the WWTP site. Acquiring land through condemnation would be a last resort option.

2. Acquisition Method
   The City intends to purchase property outright from a landowner.

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 replacing its existing evaporation ponds before they reach capacity. The schedule described in Table VI-2 provides the tasks and dates for implementation of the SBR WWTP.

Table VI-2: Schedule for Implementation of Selected Alternative

Task	Start Date	Completion Date
Submit Facility Plan to City for Review	October 2007	November 2007
Discussions with SD DENR on Funding and Selected Alternative	November 2007	January 2008
Evaluate Funding Options	January 2008	March 2008
Select Potential Sites for Lift Station, Equalization Basin, and WWTP Site	January 2008	February 2008
Begin Preliminary Negotiations with Landowners	February 2008	April 2008
Begin Preliminary Discussions with Lincoln County	March 2008	May 2008
Obtain Options on Land for Selected Sites	May 2008	July 2008
Obtain Conditional Use Permits for Selected Sites from Lincoln County	July 2008	December 2008
Purchase Land	January 2009	February 2009
Complete Environmental Review	January 2009	March 2009
Public Hearing for Facility Plan	February 2009	February 2009
Finalize Facility Plan and Submit to SD DENR for Review	March 2009	March 2009
Apply for Clean Water Intended Use Plan	March 2009	April 2009
Apply for Federal Funding	March 2009	June 2009
Preliminary Design	April 2009	August 2009
Final Design	September 2009	January 2010
Advertise/Bid/Award/Notice to Proceed	February 2010	March 2010
Construction	April 2010	November 2011
Startup	December 2011	February 2012
Permit Compliance	March 2012	March 2012