SECTION 6

EXPANSION ALTERNATIVES

Exhibit KAM-1

6. EXPANSION ALTERNATIVES

6.1 GENERAL DISCUSSION

The Facility Planning Area for the Northern Moraine Wastewater Reclamation District is roughly 14 % developed. Section 2 describes the comprehensive plans for the surrounding communities to be served and predicts an ultimate build-out of the Facility Planning Area to be as high as 10.0 MGD.

Sections 3 and 4 have demonstrated that extension of the collection system to serve the communities is a practical solution. A second solution could include construction of remote treatment facilities. The Facility Planning Area and current treatment facility were constructed to provide for regionalization. As such, it was the original intent of the Water Quality Management Plan that only one treatment facility would be required to serve the Planning Area.

Regionalization provides both economic and non-economic benefits over satellite treatment facilities. The greatest benefit is simply due to economy of scale. All treatment facilities have the same basic physical, biological and chemical process requirements. The appropriate processes must be constructed at each facility and expanded as required. As a result, a regional treatment facility can typically be constructed at a lower price per gallon treated.

A second economic factor to be considered is the ability to phase construction. Smaller facilities are traditionally completed in few phases because their service area is limited. In comparison, a regional facility can be readily phased to accommodate the needs of growth as it occurs. The existing users should not be required to pay for expansion. As land within the Facility Planning Area is served, a connection fee is paid. The amount of the fee is applied to constructing additional capacity.

A third economic factor is the cost of operation of a regional facility. Fewer staff are required per million gallon treated, automation is more cost effective and supplies are bought at reduced bulk rates.

Some non-economic factors include:
- Effluent quality
- Limitation of impacts on surrounding land values
- Limitation of environmental impacts
- Retention of trained professional staff

The largest drawback to regionalization will be the cost of constructing an interceptor sewer system that serves development as required instead of forcing properties to develop in sequence as the system is constructed.

Exhibit KAM-1

6.1.1 Biological and Hydraulic Expansion

It is evident that the existing processes are meeting the expectations of the operational staff. The expansion improvements must provide continuity to maintain the reliability and simplicity of operation established in the first phase.

The long-range plan should include a process train that minimizes pumping and maintains reliability and continuous compliance with NPDES permit limits. The alternatives investigated should take into consideration construction and operational costs, expandability, phasing and future regulations.

6.1.2 Biosolids Stabilization

The biosolids or sludge produced in the treatment process must be stabilized prior to disposal. The District currently utilizes aerobic digestion to meet the Class B requirements for land application.

Some alternatives to be considered include methods for increasing the solids concentration within the aerobic digestion, expansion of the aerobic digesters or conversion to a process that will produce Class A biosolids.

The selected biosolids alternative should be a function of the District's long-term goals and the biological process that is employed within the facility.

6.1.3 Excess Flow Treatment

The USEPA and Illinois EPA are contemplating elimination of wet weather permit standards. Unlike most communities along the Fox River, the District is capable of providing secondary treatment to 100% of the influent flow even during wet weather periods.

The proposed improvements will maintain the strength of the treatment facilities with respect to the treatment capacity of infiltration and inflow. In doing so, the District will be able to avoid future expenses related to this potential regulatory change.

6.1.4 Nutrient Removal

The Illinois EPA will be implementing additional nutrient removal criteria within the next ten years. Two nutrients of concern are total nitrogen and phosphorus.

Total Nitrogen can be removed by biological means through a denitrification process. Provisions for the denitrification process will need to be incorporated into the design of the expanded oxidation ditch process. This enhancement will address concerns regarding changes in the regulatory requirements with respect to this nutrient.

Phosphorous removal can be accomplished through chemical precipitation or biologically by adding a step to the denitrification process. Each of these alternatives should be reviewed and compared with the District's long-range biosolids stabilization plans.

6.2 EXPANSION PHASING

The most economical approach to expanding a regional treatment facility will be to develop a phasing program that meets the Facility Planning Area's long-range needs, while being constructed in small enough increments to avoid unnecessary debt.

The potential build-out of the Facility Planning Area equates to 90,000 P.E. The existing facility is designed to accommodate 20,000 P.E. Some basic factors to consider are land requirements, treatment process configuration, and ease of expansion for existing unit processes. The most direct approach to addressing the issues is to provide a conceptual design for the ultimate plant, and then to break the improvements into logical phases. Other regional plants where this approach has not been employed have resulted in side stream or parallel processes that are difficult to manage and operate. The phasing should capitalize on the strengths of the existing treatment facility, by incorporating the oxidation ditch, final clarifiers and sludge dewatering facility.

The Phase I Expansion completed in 1998 set precedence for the future expansion of the facility. While that particular expansion-phasing plan only contemplated one additional phase increasing the capacity from 2.0 to 3.0 MGD, it was concluded that future phases should be designed to parallel the processes as much as practical. It was also agreed that the expansion to 9.0 MGD should be completed in a minimum of additional four phases. Therefore, the selected expansion-phasing program is as follows:

- **Phase I – 2.0 MGD - Completed 1998**
- **Phase II – 3.0 MGD**
- **Phase III = 4.5 MGD**
- **Phase IV – 6.0 MGD**
- **Phase V – 10.0 MGD**

As demonstrated in Section 5, the current loading to the treatment facility is between 70 and 80% of the design capacity. It is recommended that once the facility plan is approved that the District pursue the Phase II design to ensure adequate capacity is available for continued development. Phase III, IV and V will need to be considered once the facility reaches 75% of its design capacity of the preceding phase. The design parameters and anticipated discharge requirements for each phase are identified on the following pages.

6.2.1 Phase II Design Parameters

Basis of Design

Influent Flows:	
Design Average Flow, MGD	3.0
Peaking Factor	2.5
Peak Hourly Flow Rate, MGD	7.5
Design Loading: (use conservative values)	
BOD_5 @ 220 mg/l, lbs. / day	5,504
TSS @ 260 mg/l, lbs. / day	6,505
NH3-N @ 35 mg/l, lb / day x 8.34 lb/gal	878
P @ 10 mg/l, lb / day	250
Design Effluent Parameters:	
BOD_5, 30 day avg., m g/l	20
TSS, 30 day avg., m g/l	25
NH_3-N, 30 day avg. (Apr - Oct), mg/l	1.5
NH_3-N, 30 day avg. (Nov - Mar), mg/l	3.0
TKN, mg/l	10.0
P, mg/l	1.5
pH, continuous range	6 - 9

6.2.2 Phase III Design Parameters

Basis of Design

Influent Flows:	
Design Average Flow, MGD	4.5
Peaking Factor	2.3
Peak Hourly Flow Rate, MGD	10.35
Design Loading: (use conservative values)	
BOD_5 @ 220 mg/l, lbs. / day	8,256
TSS @ 260 mg/l, lbs. / day	9,757
NH3-N @ 35 mg/l, lb / day x 8.34 lb/gal	1,317
P @ 10 mg/l, lb / day	375
Design Effluent Parameters:	
BOD_5, 30 day avg., m g/l	20
TSS, 30 day avg., m g/l	25
NH_3-N, 30 day avg. (Apr - Oct), mg/l	1.5
NH_3-N, 30 day avg. (Nov - Mar), mg/l	3.0
TKN, mg/l	10.0
P, mg/l	1.5
pH, continuous range	6 - 9

6.2.3 Phase IV Design Parameters

Influent Flows:	
Design Average Flow, MGD	6.0
Peaking Factor	2.2
Peak Hourly Flow Rate, MGD	13.2
Design Loading: (use conservative values)	
BOD_5 @ 220 mg/l, lbs. / day	11,010
TSS @ 260 mg/l, lbs. / day	13,010
NH3-N @ 35 mg/l, lb / day x 8.34 lb/gal	1,751
P @ 10 mg/l, lb / day	500
Design Effluent Parameters:	
BOD_5, 30 day avg., m g/l	10
TSS, 30 day avg., m g/l	12
NH_3-N, 30 day avg. (Apr - Oct), mg/l	1.5
NH_3-N, 30 day avg. (Nov - Mar), mg/l	3.0
TKN, mg/l	10.0
P, mg/l	1.5
pH, continuous range	6 – 9

6.2.4 Phase V Design Parameters

Influent Flows:	
Design Average Flow, MGD	10.0
Peaking Factor	2.0
Peak Hourly Flow Rate, MGD	20.0
Design Loading: (use conservative values)	
BOD_5 @ 220 mg/l, lbs. / day	18,348
TSS @ 260 mg/l, lbs. / day	21,684
NH3-N @ 35 mg/l, lb / day x 8.34 lb/gal	2,919
P @ 10 mg/l, lb / day	834
Design Effluent Parameters:	
BOD_5, 30 day avg., m g/l	10
TSS, 30 day avg., m g/l	12
NH_3-N, 30 day avg. (Apr - Oct), mg/l	1.5
NH_3-N, 30 day avg. (Nov - Mar), mg/l	3.0
TKN, mg/l	10.0
P, mg/l	1.5
pH, continuous range	6 - 9

6.3 HEADWORKS

The District's headworks has been expanded twice to date. The facility has a small screening channel that is fed from the 30" interceptor sewer. The screening channel is directly connected to the wet well, which is located underneath the channel. The screening structure is not adequately sized for the ultimate design flow. The current screens are only capable of treating 6.8 MGD verses the ultimate Peak Hourly Flow of 20.0 MGD. Similarly, the wet well is small compared to the ultimate peak hourly flow (12,500 gpm).

Consideration of expansion verses replacement was discussed with District staff. The existing screening and wet well structure could be expanded to the east. The staff is very pleased with the performance of the existing Lakeside mechanical fine screens and would prefer to maintain similar units if practical. The screened material is conveyed to ground level by shaftless conveyors. The staff would prefer to have the screens enclosed in a heated structure that enhances their access for maintenance and reduces winter operational issues. A second issue to be addressed is providing a continuous downstream head to increase screen performance. A third is to incorporate odor control on the headworks structure.

The existing control building includes a spacious dry well for the raw sewage pumps and allows for easy maintenance of the equipment. The dry well design is very functional and consideration for continued use should be incorporated into the expansion plans. However, it is important to note that the finish floor of the Control Building is only 739. When this structure was originally constructed the Record Flood Elevation was 736.5 according to the 1977 Plans. The 1998 plans indicate the 100-Year flood elevation to be 738.4. In short, this structure could potentially flood. Replacement of the raw sewage pumps should consider immersible motors to prevent damage to the pumps should flood waters exceed the 739 elevation.

The usable height of the wet well is limited to five feet from high to low water level, which equates to roughly 4,200 gallons. If the screening facility is expanded further east, the wet well could be expanded similarly.

The challenges for expansion include:

- Completing the renovation without removing the existing facility from service.
- Removal of the concrete fillet in wet well.
- Balancing the hydraulic split between future screens.
- Space limitations within the existing property.

Construction of a new facility would provide the District with greater flexibility to incorporate additional treatment alternatives. Options discussed included septic receiving, screening washing, and grit removal.

A large portion of the Facility Planning Area is currently served by septic tanks and private mechanical systems. The District has been approached on several occasions to treat septic waste. The District has elected not to accept the waste because it could upset the process without proper pre-treatment. If the septic receiving facility were constructed properly and the loads incorporated into the biological process design, the District could better serve the residents within the Facility Planning Area not currently served by sewers.

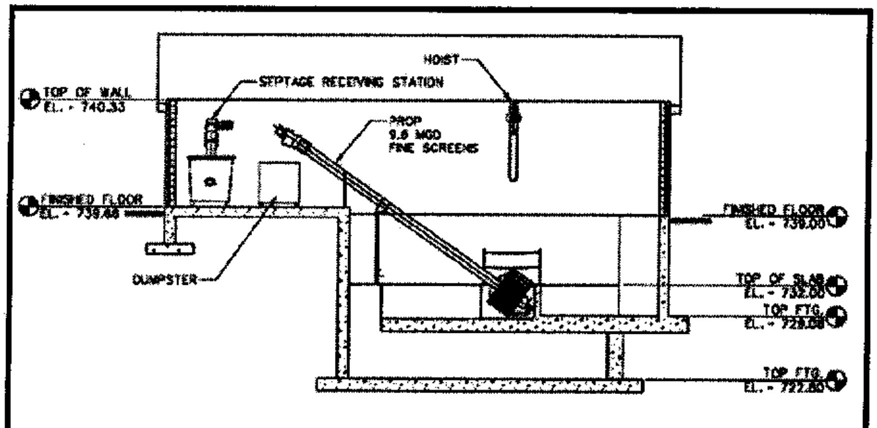
A septic receiving station would require convenient truck access. The preferred design would incorporate the septic receiving station into the headworks facility to minimize screenings handling and odor control. However, it is recognized that the receiving station could be constructed in a separate location and housed in its own building. This would require separate odor control, screening and grit facilities.

Consideration for grit removal was based on the staff's observations from cleaning sewers and wet wells. While the staff was not able to quantify the amount of grit received at the plant, it was noted that these cleaning operations often include high concentrations of sand. Through the discussions, it was agreed that as long as the process continued to begin with the oxidation ditch, the grit could be captured and removed as part of routine maintenance. However, grit removal would be required if future phases include primary clarifiers and anaerobic digesters.

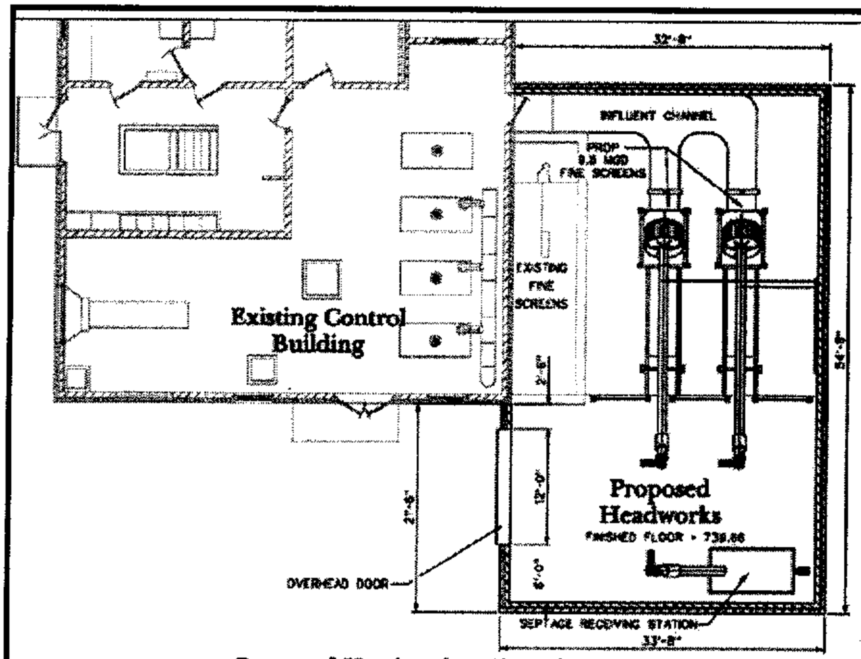
Grit removal can be accomplished either before or after raw sewage pumping. The options discussed included vortex and aerated grit chambers. In either case, this treatment process could be constructed in conjunction with the primary clarifiers if required.

The expansion of the existing headworks was the preferred alternative. It was determined that the structure should be expanded only once more to provide the most economical approach. The improvements will include construction of a new structure adjacent to the existing influent channel and wet well. The design of the structure allows it to be completed without any major shutdowns or extended by-pass pumping.

The proposed addition will be constructed at the same footing elevation and match the wall dimensions of the existing Control Building's foundation. Two new channels will be constructed for the Phase-V – 12.8 MGD screens. Each screen would be capable of treating in excess of 50% of the ultimate design flow. The existing channel will be maintained to allow for by-pass during times of maintenance. The screened material will be conveyed from the channel to the upper level. The selected equipment is capable of compacting, dewatering and bagging the material in dumpsters to reduce the odors traditionally associated with the screening. Also the upper level will be equipped with a 400 gallon per minute septage receiving station. The selected unit is also capable of washing, compacting and bagging the waste material.



Proposed Headworks - Section View



Proposed Headworks - Plan View

Extension of headworks and wet well will likely require the purchase of additional property to the east of the existing plant. Some of this property is may be within the flood plain. Prior to preliminary design, a wetland and topographic survey should be completed to better define construction issues.

The plan view and section (above) provides a conceptual layout of the proposed headworks. Expansion of the wet well will increase the volume from 4,200 gallons to roughly 18,000 gallons.

It is anticipated that the structural improvements would be incorporated into the Phase II Expansion. The existing screens will be maintained and one new Lakeside Model FS63 screen would be installed. The third channel will be utilized for an optional by-pass until Phase III when the second 12.8 MGD would be required. The existing screens will be replaced in Phase IV with an identical unit to Phase II & III. The septage-receiving unit would also be installed during Phase II.

6.3.1 Phase II – 3.0 MGD:

Design Parameters	
Number of channels	3
Peak Hourly Flow, MGD	7.5
Capacity existing screens, MGD	6.8
Capacity Phase II screen, MGD	9.6
Capacity by-pass channel, MGD	9.6
Gravity Influent	30"
Opening Space	¼"

6.3.2 Phase III – 4.5 MGD:

Design Parameters	
Number of channels	3
Peak Hourly Flow, MGD	10.35
Capacity existing channel, MGD	9.6
Capacity Phase II screen, MGD	9.6
Capacity Phase III screen, MGD	9.6
Gravity Influent	30"
Opening Space	¼"

6.3.3 Phase IV – 6.0 MGD:

Design Parameters	
Number of channels	3
Peak Hourly Flow, MGD	13.2
Capacity Phase II screen, MGD	9.6
Capacity Phase III screen, MGD	9.6
Capacity Phase IV screen, MGD	9.6
Gravity Influent	30"
Opening Space	¼"

Exhibit KAM-1

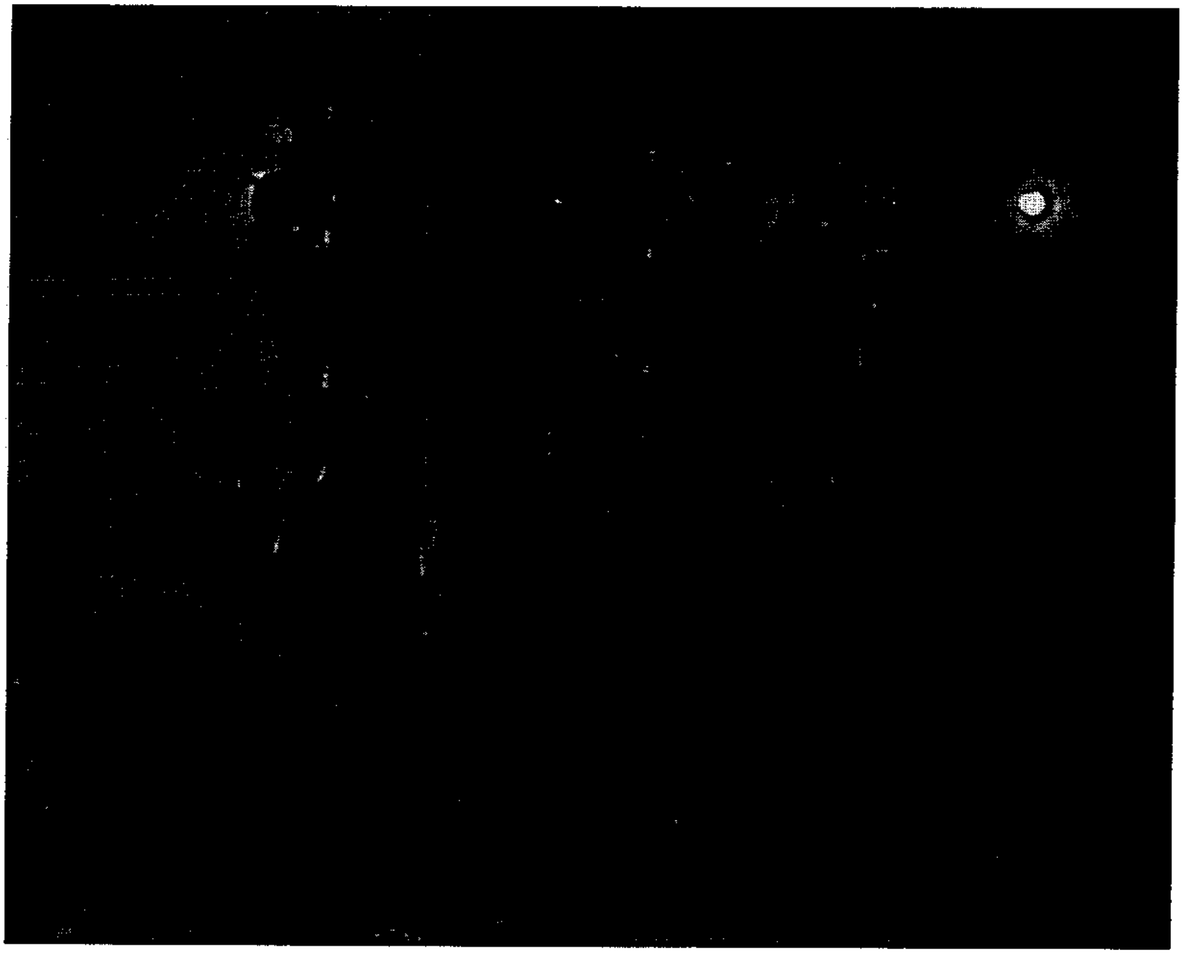
6.3.4 Phase V – 9.0 MGD:

<u>*Design Parameters*</u>	
Number of channels	3
Peak Hourly Flow, MGD	20
Capacity Phase II screen, MGD	12.8
Capacity Phase III screen, MGD	12.8
Capacity Phase IV screen, MGD	12.8
Gravity Influent	30"
Opening Space	¼"

Each phase complies with the minimum design requirements for providing continuous operation with the largest unit out of service.

6.4 RAW SEWAGE PUMP STATION

Expansion of the wet well will provide greater flexibility in operation for the pump station. The raw sewage pumps will continue to be controlled by variable frequency drives. As such the pumping system will be able flow pace and maintain a near-constant level in the wet well. The dry well has space for four pumps. The pumping system for each phase should possess the maximum range to limit start and stop operation. In addition, the pump station is rated with the largest pump out of service. Therefore, the range of flows must be covered by a three-pump system.



The pumping system installed during the Phase I expansion only provided enough capacity for 5.0 MGD and does not accommodate the Phase II requirements. The pumps will need to be replaced for this project. Due to the potential for flooding, it is recommended that proposed pumps include immersible motors.

Rather than replace all of the pumps, a more economical approach of only replacing two pumps in each phase will be attempted. As a result, the plant will be capable of flow pacing with the smaller pumps and operating one of the larger pumps in combination with two small pumps during peak flows. This approach will meet the redundancy requirements of being able to pump the Peak Hourly Flow with one of the largest pumps out of service.

6.4.1 Phase II – 3.0 MGD

Peak Hourly Flow, gpm	5,208
Number of existing units	2
Type	Flow Serve Centrifugal
Existing Pump Range (Each), gpm	580 - 1,160
Number of proposed units	2
Type	Wemco Hidrostal
Phase II Pump Range (Each), gpm	1,625 -3250
Force Main Dia.	18"
Velocity, ft/sec	7.31
Maximum Capacity of P.S.	5,570 gpm

6.4.2 Phase III – 4.5 MGD

Peak Hourly Flow, gpm	7,200
Number of Phase II units	2
Type	Wemco Hidrostal
Phase II Pump Range (Each), gpm	1,625 -3250
Number of Phase III units	2
Type	Wemco Hidrostal
Phase III Pump Range (Each), gpm	1,625 -3250
Force Main Dia.	18"
Maximum Capacity of P.S.	9,750 gpm

6.4.3 Phase IV – 6.0 MGD

Peak Hourly Flow, gpm	9,166
Number of Phase II units	2
Type	Wemco Hidrostal
Phase II Pump Range (Each), gpm	1,625 -3250
Number of Phase III units	2
Type	Wemco Hidrostal
Phase III Pump Range (Each), gpm	1,625 -3250
Force Main Dia.	18"
Maximum Capacity of P.S.	9,750 gpm

6.4.4 Phase V – 10.0 MGD

Peak Hourly Flow, gpm	13,888
Number of Phase III units	2
Type	Wemco Hidrostal
Phase III Pump Range (Each), gpm	1,625 -3250
Number of Phase V units	2
Type	Wemco Hidrostal
Phase IV Pump Range (Each), gpm	3,250 – 7,000
Force Main Dia.	24"
Maximum Capacity of P.S.	14,000 gpm

6.4.5 Description of Force Main Improvements

During the Phase I expansion, the raw sewage force main was only extended as an 18", but the original 12" main which fed the package treatment plants remains in service. Typically, force mains are sized to flow at velocities ranging from 2 feet per second to 9 feet per second. This range prevents the settling of solids, yet does not cause unnecessary headloss through the piping.

The Phase II minimum flow rate would be 580 gpm and maximum of 5,570 gpm. The velocity of the flow through the 12" main at 5,570 gpm will exceed 15 feet per second. The original design of the raw sewage pump station provided for 2-12" connections. It is recommended that both connections be used and the 12" force main within the yard be replaced with an 18" forcemain.

The Phase III minimum flow rate is expected to be 1,100 gpm, which would create a minimum velocity of 1.4 feet per second. The Maximum flow rate is proposed to be 7,200, which equates to slightly over 9 feet per second through the 18" main. Only minor improvements to provide for a diversion structure will be required.

The Phase IV minimum flow rate is expected to be 1,625 gpm, which would create a minimum velocity of 2 feet per second. The Maximum flow rate is proposed to be 9,750 that equates to slightly over 12 feet per second through the 18" main.

Phase V Improvements will result in a pumping capacity range from 1,625 to 14,000 gpm. The raw sewage in this phase will be diverted to grit removal and primary clarifiers. At this time the raw sewage force main should be replaced with a 24" main. The header piping in the dry well will also need to be modified to address these larger flow rates.

6.5 GRIT REMOVAL AND PRIMARY TREATMENT

The wastewater treatment facility process does not currently include grit separation or primary settling. Raw sewage is introduced directly to the biological process. The implementation and cost savings associated with these processes were discussed with District staff.

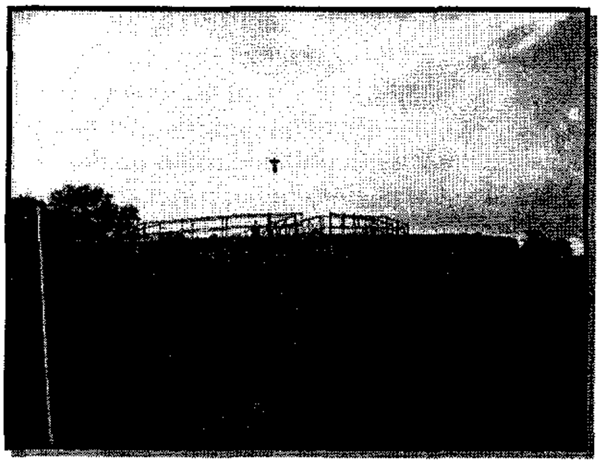
It was concluded that grit separation did not provide a direct cost benefit to the District, as implementation of the Phase II Improvements will allow for each channel of the oxidation ditches to be cleaned periodically. As a result, the capital and operational cost of constructing separate facilities for grit removal exceeded the twenty-year maintenance cost of cleaning the channels every two years.

It was recognized that primary settling would significantly reduce BOD_5 and TSS loading to the biological process. In addition, the reliability of chemical phosphorus removal was compared to the biological removal process. While the economic and operational benefits were recognized to be significant, the capital cost of constructing the new facilities and converting the digestion process to anaerobic digestion was determined to be impractical for the Phase II expansion. It was determined that these improvements were better suited for future phases.

Construction of the primary settling within Phase III would allow the plant to be expanded to 4.5 and the plant could continue to operate in single stage nitrification mode. While it is possible to aerobically digest primary sludge, the process it has a much greater potential for releasing odors than biologic sludges and would likely require construction of anaerobic digestion. However, if a second 2.25 MGD oxidation ditch is constructed the biological loading to the facility can be decreased to operate the facility as extended aeration. As a result, the aerobic digestion facility constructed in Phase II would have adequate capacity. Similarly, Phase IV loadings could be treated without primary clarifiers.

It was recommended by the operational staff that grit removal and primary clarifiers not be considered until the Phase V expansion.

6.6 BIOLOGICAL PROCESS



As explained in Section 5, the District employs a suspended growth biological process defined as single stage nitrification. The single stage nitrification process is a version of activated sludge that creates an environment to promote BOD_5 reduction and nitrification of ammonia (conversion to nitrate and nitrite) within the same process. The environment promotes the growth of aerobic microorganisms, which metabolize the influent BOD_5 and nutrients to energy and biomass.

The current process is completed within a two-ring oxidation ditch. The oxidation ditch design allows for the completely mixed basins to operate in series or parallel. Mechanical aerators are attached to the walls of the channels, which include a series of discs that rotate in the wastewater. The discs provide both mixing energy and the oxygen required by the microorganisms for aerobic respiration.

The Illinois EPA will soon require removal of phosphorus and total nitrogen from wastewater effluent as well. The oxidation ditch design possesses significant flexibility and can be adapted to incorporate biological phosphorus removal and the denitrification (conversion of nitrite & nitrate to nitrogen gas) by implementing slight operational modifications.

Denitrification is a process that occurs when the micro-organisms that have converted the ammonia to nitrate NO^{-3} and nitrite NO^{-2} are placed in an environment containing low dissolved oxygen levels. The aerobic microorganisms then recover the oxygen from the nitrate and nitrite for respiration. As a result, the nitrogen gas is released to the atmosphere.

Biological phosphorus removal is slightly more complex. The microorganisms that are selective for phosphorus survive in both aerobic and anaerobic conditions. Under aerobic condition, the organisms convert phosphorus to phosphate PO^4, but under anaerobic the phosphate is broken down to provide the required oxygen for respiration. Biological Phosphorus Removal requires development of a process where the microorganisms are repeatedly exposed to an anaerobic zone as part of the process. The three-channel design accommodates this requirement very well by operating the outer channel very close to an oxygen deficit.

The Phase II Expansion will include construction of the third channel as originally proposed in the 1998 Facility Plan Amendment. By providing additional process control, the single stage nitrification process can be upgraded to provide biological phosphorus removal and denitrification.

The upgraded process will require increased operator attention and flexibility. One of the key components will be the installation of a Supervisory Control and Data Acquisition (SCADA) system to monitor flow rates of raw sewage and return activated sludge in addition the monitoring dissolved oxygen levels in each of the channels. The staff will be required to closely monitor the key operational parameters such as the sludge age and feed to mass ratio, which are determined in the laboratory. The operators will also be given the capabilities to control the process through variable frequency drives on each of the aerator motors. This capability will allow the staff to vary the speed of the aerators as required to meet the oxygen demands of the biological process.

6.6.1 The Phase II Design Parameters

Design	Oxidation Ditch
Design Average Flow	3.0 MGD
Peak Hourly Flow	7.5 MGD
BOD_5	220 mg/l
BOD_5	5,505 lbs/day
TSS	260 mg/l
TSS	6,505 lbs/day
NH_3-N	35 mg/l
NH_3-N	875 lbs/day
Number of channels	3
Side Water Depth	14 feet
Channel Width Inside	20 feet
Channel Width Middle	20 feet
Channel Width Outside	20 feet
Detention Time	19 hours
Total Volume	2,357,509 gal
Solid Production	5,505 lbs/day
Solids Inventory	78,730lbs
Sludge Age	14.3 days
F/M Ratio	0.07
Oxygen Required	482 lbs/hr
Oxygen Supplied	900 lbs/hr

6.6.2 Future Phases

Phase III – 4.5 MGD will include construction of a second 2.3 MG oxidation ditch. The detention time within the process would be increased to 24 hours and it would qualify as extended aeration, as such, the aerobic digestion process will not need to be increased during this phase.

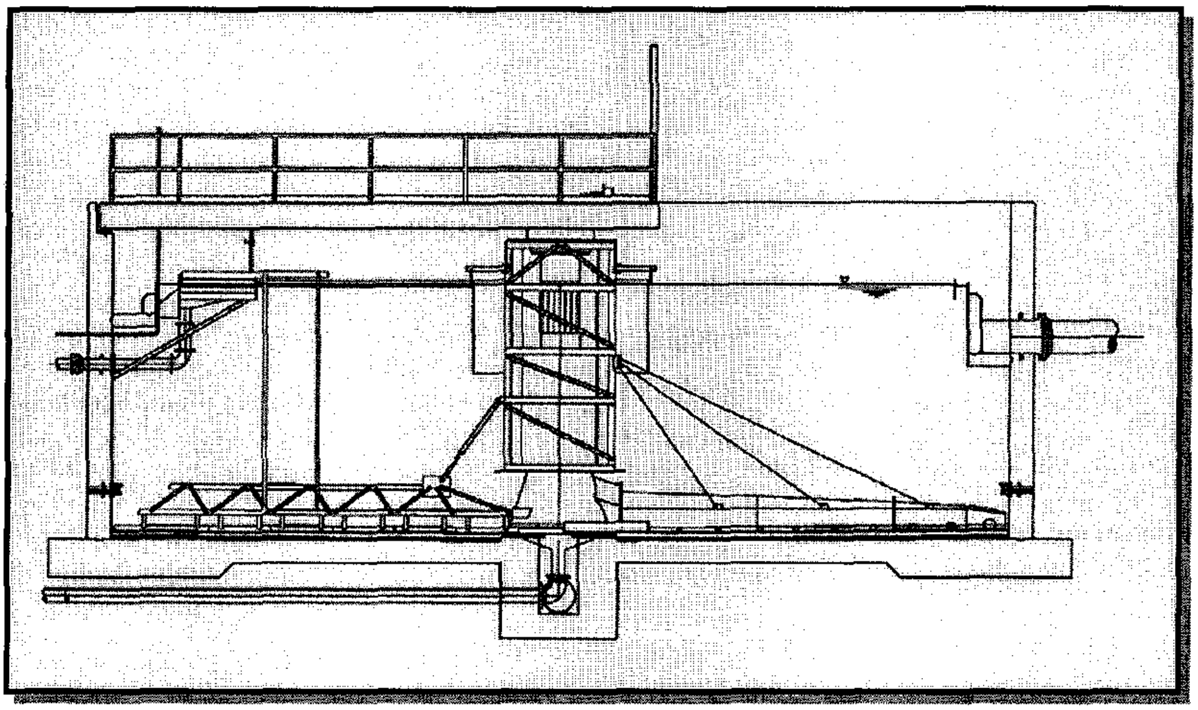
Phase IV – 6.0 MGD will increase organic and nutrient loading to duplicate the Phase III requirements. While the oxidation ditch will not need to be expanded to treat the additional load, the aerobic digesters will be upgraded to address the increased sludge quantity.

Phase V – 10.0 MGD will require construction of either a third oxidation ditch or primary clarifiers. While the final decision will be addressed in follow-up Facility plans, it is the current recommendation that primary clarifiers be installed in this phase. The primary clarifiers would reduce the biological and nutrient loading to the two oxidation ditches sufficiently to allow operations without providing additional volume.

6.7 FINAL CLARIFIERS

The performance of the existing clarifiers has been outstanding. The solids concentration in the final effluent is similar to that of a tertiary treatment facility and has continually met the NPDES permit standards. The Illinois EPA standard for surface overflow rates is 1,000 gallons per day per square foot. The existing 85-foot diameter final clarifiers have a surface overflow rate of 440 GPD/SF and under the Phase II conditions the surface-loading rate will be only be 660 GPD/SF.

The existing mechanisms were manufactured by USFilter/ Envirex. The To-Bro™ clarifier works on a hydraulic differential principal for sludge removal. The first phase design is large enough to accommodate Phase II flow with only minor pining changes. An additional Clarifier will be constructed in Phase III and two more in Phase IV.



The anticipated design loading for each phase is shown on the following page.

6.7.1 Phase II Design Conditions

Number of Units	2
Design	Hydraulic Differential
Clarifier Diameter	85ft
Design Average Flow (each)	1.5 MGD
Peak Hourly Flow (each)	3.75 MGD
Surface Loading Rate	660 gal/ day/ sq.ft.
Solids Loading Rate	22 lbs/ day/ sq.ft.
Weir Loading Rate	15,090 gal/ day/ ft.

6.7.2 Phase III Design Conditions

Number of Units	3
Design	Hydraulic Differential
Clarifier Diameter	85ft
Design Average Flow (each)	1.5 MGD
Peak Hourly Flow (each)	3.45 MGD
Surface Loading Rate	607 gal/ day/ sq.ft.

6.7.3 Phase IV Design Conditions

Number of Units	5
Design	Hydraulic Differential
Clarifier Diameter	85ft
Design Average Flow (each)	1.2 MGD
Peak Hourly Flow (each)	2.64 MGD
Surface Loading Rate	465 gal/ day/ sq.ft.

6.7.4 Phase V Design Conditions

Number of Units	6
Design	Hydraulic Differential
Clarifier Diameter	85ft
Design Average Flow (each)	1.67 MGD
Peak Hourly Flow (each)	3.33 MGD
Surface Loading Rate	587 gal/ day/ sq.ft.

6.8 DISINFECTION FACILITY

The chlorination facility was constructed during the Phase I Expansion 1998. The original design provided adequate capacity for a Peak Hourly Flow of 7.5 MGD.

Chlorine is a very strong oxidizing agent used for disinfection. The efficiency of the chlorine disinfection process is a function of chlorine concentration and detention time.

At the District, a Parshall flume upstream of the chlorine contact tank measures the final clarifier effluent. The chlorine dosage is flow paced to provide adequate disinfection without overdosing. The chlorine contact tank includes two parallel serpentine channels that provide 16 minutes detention time at 7.5 MGD. The serpentine pattern ensures that short-circuiting cannot occur which may result in inadequate detention time.

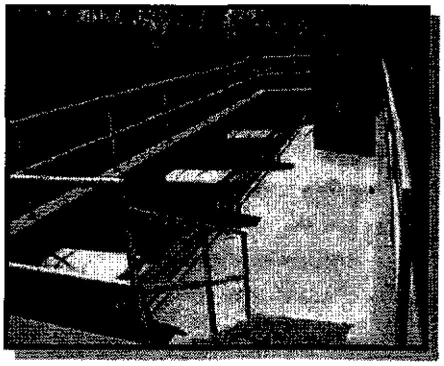
Depending on the detention time in the chlorine contact tank and the strength of the original concentration, chlorine residual is likely to be present prior to discharge. If the concentration of the chlorine in the effluent was significant, it may have a negative effect on native species within the receiving waters. The NPDES Permit limits the concentration of chlorine in the effluent to 0.05 mg/l to avoid these issues. The District neutralizes the chlorine residual by the addition of sulfur dioxide gas.

The existing facility meets the Illinois EPA requirements for the Phase II expansion. The design calculations are shown below.

<u>Chlorine Contact Tank:</u>

Number of Tanks	2
Design	12' x 56' x 8.5' SWD
Total Volume, cuft	11,425
Total Volume, gal.	85,460
Design Average Flow	3.0 MGD
Peak Hourly Flow	7.5 MGD
Detention Time @ Peak, Min.	16.4

Traditionally chlorine has been the preferred disinfection technology. However, concerns over disinfection by-products, post treatment requirements, public safety and chemical handling are some of the reasons most treatment facilities are converting from chlorination to ultraviolet disinfection.



Ultraviolet Disinfection is an environmentally friendly method of disinfecting wastewater. Microorganisms, including viruses, are inactivated when exposed to UV-C light in a controlled environment and dosage. The UV-C

Exhibit KAM-1

light with a frequency of 254 nano-meters causes a physical reaction with the organisms' DNA. This reaction prevents cell division and reproduction of potentially dangerous organisms and viruses.

While the chlorination system meets the design standards for Phase II, improvements will be required for Phase III, IV and V. The technology for ultra-violet disinfection equipment is rapidly improving. Therefore, selection of a particular design in this report is not practical. It is proposed that the District evaluate the current technology and install an ultraviolet disinfection system during Phase III that is expandable for each future phase.

6.9 RAS/WAS PUMP STATION

The existing RAS/WAS pump station consists of two constant speed 1,400 gpm pumps in a small pre-cast wet well. The pumps are controlled by floats and operate in an on/off cycle. Based on the expansion requirements, the existing pump station is not capable of meeting the future design conditions. The Phase II required return rate is 3.24 MGD or 2,250 gpm.

Calculate Volume of Return Activated Sludge Phase II

Assume RAS concentration is 7,500 mg/l
$(Q_{RAS} \times mg/l_{RAS}) + (Q_{In\ BOD} \times mg/l_{In\ BOD}) = (Q_{In\ BOD} + Q_{RAS}) \times mg/l_{MLSS}$
$(Q_{RAS} \times 7{,}500\ mg/l) + (3.0 \times 220\ mg/l) = (3.0 + Q_{RAS}) \times 4{,}000\ mg/l$
$7500\ Q_{RAS} + 660 = 12{,}000 + 4{,}000\ Q_{RAS}$
$3{,}500\ Q_{RAS} = 11{,}340$
$Q_{RAS} = 3.24$ MGD

It is recommended that a new pump station be constructed as part of the Phase II Expansion. The design of the pump station should accommodate future phases. Based on simply doubling the plant capacity in Phase IV requires a return rate of 6.5 MGD. The addition of primaries in Phase V will lower the return ratio and should be closer to 9.0 MGD or below. The estimated design conditions for the pump station are shown below.

<u>RAS Pumps</u>

Discharge elev. = 749.0
Centerline of pump = 726.0
Static Head = 23
Phase III Q_{ras} = 4,513 GPM
8" riser pipe @ 1,500 gpm = (45)/ 1000' x 150' of pipe = 7'
14" main @ 4,500 gpm = 22/ 1000' x 150' of pipe = 3'
TDH = 23 + 7 + 3 = 33 ft
Number of pumps = 3 @ 1,500 gpm @ 33 ft

The waste activated sludge from the biological process is expected to be equal to the incoming Biological Oxygen Demand due to the inclusion of biological phosphorus removal. As a result, Phase II will require wasting of roughly 5,505 lbs/day.

Calculate Volume of Waste Activated Sludge for Phase III

MOP 8 for Phosphorus removal estimates 1.0 sludge production factor
WAS = 1.0 x influent BOD_5
WAS = 1.0 x 5,505 = 5,505 lb/day
5,505 lbs/day / (8.34 lbs/gal x 7,500 ppm) = 88,000 gallons per day

The waste sludge volume in Phase III should not significantly increase because of the extended detention time in the biological process. The volumes in Phase IV will increase 11,010 lbs per day or 172,000 gallons per day.

In Phase V, if primary clarifiers and chemical phosphorus removal is included the wasting rate should be the same as Phase IV as the additional sludge is removed within the preceding process.

WAS Pumps

Discharge elev. = 749
Centerline of pump = 726.0
Static Head = 23
Phase IV Q_{was} = 250 GPM
6" riser pipe @ 250 gpm = (6.6)/ 1000' x 60' of pipe = 0.5'
6" main @ 250 gpm = 6.6/ 1000' x 500' of pipe = 3'
TDH = 23 + 0.5 + 3 = 26.5 ft
Number of pumps = 2 @ 250 gpm @ 26.5 ft

6.10 BIOSOLIDS STABILIZATION PROCESS

Prior to the Phase I Expansion, the sludge handling facility was utilized as a contact stabilization process. During the project, the tanks were converted to aerobic digesters and aerated sludge storage. Only the annular space is currently in use. The interior clarifiers have been abandoned.

The interior steel walls are reaching the end of their service life and are in need of replacement. The exterior walls of the tanks are constructed of concrete and are 78' feet in diameter.

Calculate the total digestion volume available:

> $(78 / 2)^2 \times \Pi$ = 4,780 square feet
> 4,780 sqft x 14 ft swd = 66900 cuft per tank
> 66900 cuft x 2 = 133,800 cuft of digester capacity
> 133,800 cuft / 4.5 cuft/ P.E. = 29,733 P.E.

The Illinois EPA requires sufficient digester capacity to provide 4.5 cuft / P.E. when operating single stage nitrification when provided in conjunction with separate thickening facilities. If separate thickening is not provided the volume should be increased by 25%. Additionally, the values are based on retaining the temperature in the digester at 60 degrees and a detention time of 27 days.

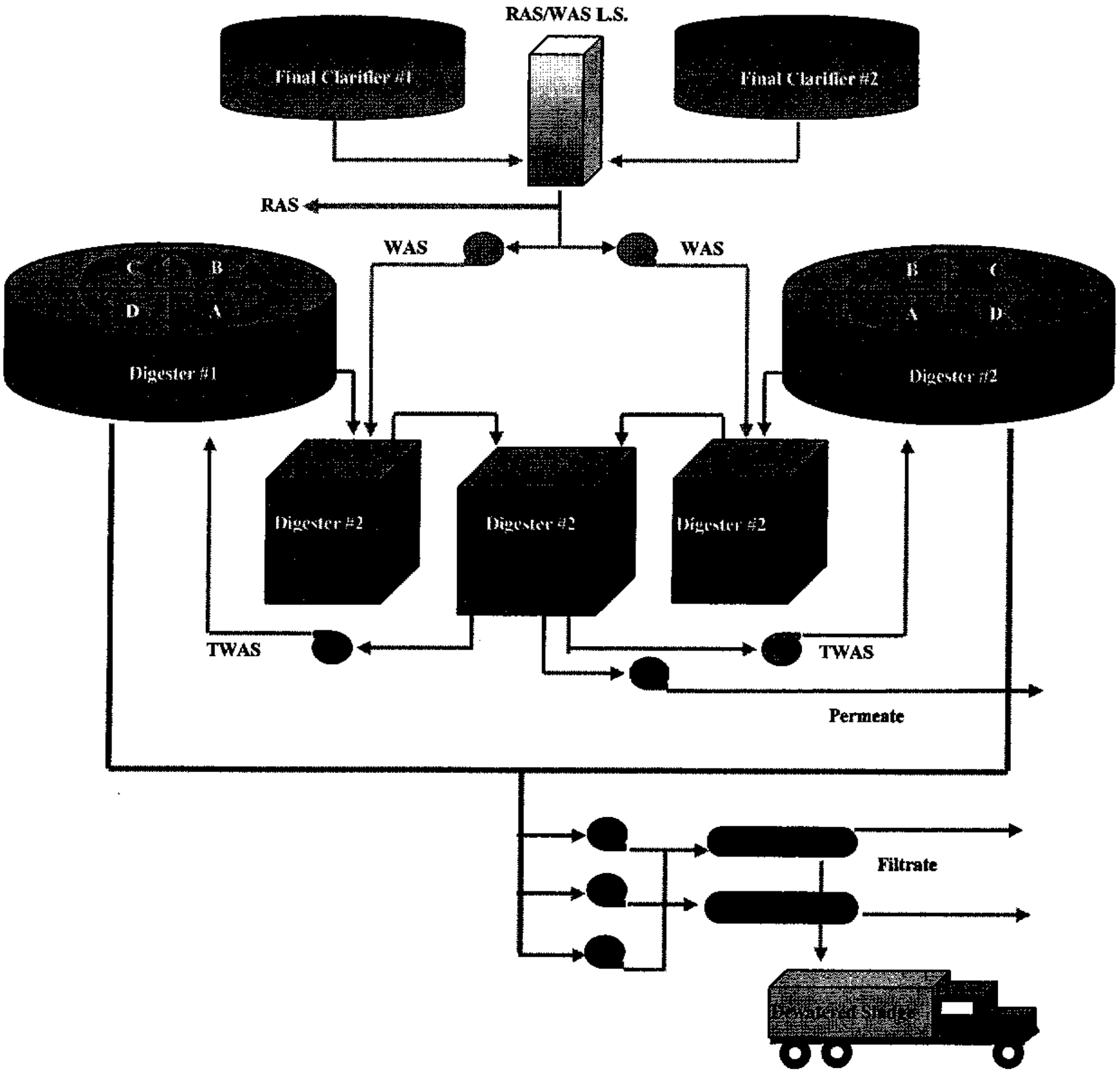
Options include:

- a gravity thickener, which will increase the solids to 2.5 to 3%
- a membrane thickening unit which will increase the solids concentration to 3 – 3.5%.

Gravity thickening results can vary and will typically not achieve the desired concentration. Membrane technology for the thickening application was investigated. The membrane thickening system is capable of thickening the digester solids to 4.0% solids, however the design calculations for the system only require 3.0% solids.

The following conceptual design includes construction of a membrane thickener complex and rehabilitation of the existing digesters. At Design Average Flow it is estimated that the biological process will produce 5,505 lb/day at 0.75% solids or 88,000 gal/day of Waste Activated Sludge. The WAS from the RAS/WAS pump station will be tributary to the two anoxic tanks of the proposed membrane thickening complex. The anoxic tanks have capacity for 25,000 gallons each.

The aerobic digestion process will consist of eleven basins including; two anoxic/ alkalinity recovery basin, a membrane thickening basin, and each existing digester (#1 and #2) will be divided into four compartments (A-D). The digesters #1 & #2 will operate in parallel while the cells within the digesters (A-D) will operate in series. The process will incorporate continuous recycle from digesters #1A and #2A to the anoxic and thickening basin. Cells B – D of each digester will isolate the digesting sludge to prevent recontamination.



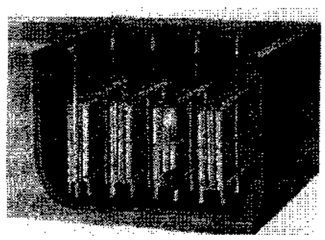
Increasing the solids concentration above 3.0% will allow heat from the exothermic reaction to be retained by the solids within the digesters. As the temperature within the digesters rises, the rate of volatile solids reduction changes. The rate of VSS reduction is a function of time and temperature. In anticipation of the variable conditions, the air design will incorporate capabilities to vary the air distribution to match the volatile solids destruction, taking place within each of the four cells. In addition, it is important to allow for alkalinity recovery. Therefore the design will incorporate an anoxic mixing zone for denitrification.

The proposed design includes placing the membrane system in a continuous loop with the first stage digester. This approach will provide in excess of 80 days detention time in the digesters by removing water as destruction occurs.

The temperature range within the aerobic digesters is expected to range from 20 °C to 35 °C. Based on the time –temperature curve provided in MOP 11, we can anticipate approximately 1,740 days- °C at 20 °C that equates to a range of to 51% reduction.

A proposed WAS pump controlled by a VFD will continuously feed the 0.75% WAS to the proposed anoxic mixing tanks. Simultaneously, approximately 400 gpm of 3.0% sludge from Digester #1A & #2A will be transferred to anoxic mixing tanks. The fresh sludge will provide the carbon source for denitrification and pH control within the digestion system. The anoxic mixing tank is sized to provide approximately two hours detention time.

To maintain a continuous flow from the anoxic tank to the membrane thickener and back to the digester, an air-lift pump will be installed within the system to provide recirculation. It was determined that the recirculation rate should be five times influent flow rate and therefore the pump should be capable of moving 500 gpm.



During our investigation, two membrane manufacturers were reviewed. It became evident that the membrane plate technology was much better suited for this application than hollow fiber strands. The membrane system will be designed for a flux rate of 5.1 gallons per day per square foot of membrane area based on the pilot plant and operational data from the manufacturer. Based on the influent flow and burn down anticipated within the digester, 13,760 square feet of membrane will be required which equates to 1,600 membrane plates.

Maintenance of the membrane system is limited to biological cleaning with 0.5% sodium hypochlorite solution once every four months. This cleaning process will only take roughly an hour and will require the addition of roughly 1,200 gallons of chemical.

As stated previously the system is expected to operate in a range between 20 °C and 35 °C. The lower value was based on operating data accumulated over the past three years at lower solids concentrations. It is expected that the volatile solids reduction in the first phase will be a minimum of 38% reduction. As a result, the aeration requirements for the first phase digester range are 2400 – 3,000 cfm.

The digested sludge from the first phase digester overflows to the second (B) and third (C) stage before entering the digested sludge holding tank (D). The aeration requirements for the subsequent stages are defined by the required mixing energy rather than oxygen demand, due to the efficiency of the first stage reactor.

6.11 SLUDGE DEWATERING AND STORAGE:

The Phase II improvements will include installation of the second belt filter press conveyor, polymer unit and progressive cavity pump. Each belt filter press has capacity to dewater 1,500 lbs / hour. Solids production from the digester complex equates to 15,750 gallons per day at 2.5% solids. This equates to a sludge feed rate of roughly 120 gpm. The operational time required the sludge with two belt filter presses would be roughly 7.6 hours per week.

The weekly sludge production equates to 351 cuft / day. The District currently contracts for land application of biosolids on agricultural property. It is recommended that the District provide 150 days winter storage if another means of disposal is not available. This equates to 1,960 cu. yds of stored material. The existing sludge drying beds will not be adequate to provide the required sludge storage. It is recommended that the District construct and 100 x 100 foot sludge storage barn as part of the Phase II Improvements.

6.12 PERMITTING

The permitting requirements for The Phase II project will include

- Submittal of the Facility Plan Amendment to Northeastern Illinois Planning Commission
- Submittal of the Facility Plan Amendment to the Illinois EPA Permit Section
- Request for an updated NPDES Permit
- Request for a Department of Natural Resources – Endangered Species Consultation
- Request for a Illinois Historical Protection Agency Consultation
- Army Corps of Engineer's sign-off for work in the Flood Plain
- Permit from DNR-Division of Water Resource for work in the Flood Plain
- Illinois EPA Construction Permit

Each of the permits will be acquired during the appropriate phase of the project.

6.13 PHASE II DESIGN SUMMARY

Influent Flows:

Design Average Flow	=	3.0	MGD
Design Maximum Flow	=	6.0	MGD
Peaking Factor	=	2.5	
Peak Hourly Flow Rate	=	7.5	MGD

Design Loading:

Average BOD_5 is 202 mg/l, but use 220 mg/l for design with septage receiving

Design BOD_5 = 3.0 MGD x 220 mg/l x 8.34 lb = 5,505 lb/day

Average TSS in 2002 was 212 mg/l, but use 260 mg/l for design with septage receiving

Design TSS = 3.0 MGD x 260 mg/l x 8.34 lb = 6,505 lb/day

NH3-N = 3.0 MGD x 35 mg/l x 8.34 lb/gal = 875 lb / day

Phosphorus – 3.0 MGD x 13 mg/l x 8.34 lb/gal = 250 lb / day

Design Effluent Parameters:

BOD_5, 30 day avg.	=	20	mg / l
TSS, 30 day avg.	=	25	mg / l
NH_3-N, 30 day avg. (Apr - Oct)	=	1.5	mg / l
NH_3-N, 30 day avg. (Nov - Mar)	=	3.0	mg / l
pH, continuous range	=	6 - 9	

Process Description:
The Wastewater Treatment Plant process shall include preliminary screening, raw sewage pumping, single stage nitrification, clarification, chlorine disinfection, aerobic digestion and sludge dewatering. The following is a listing of process components and their associated sizes.

<u>Headworks:</u>

Number of channels	3
Peak Hourly Flow, MGD	7.5
Capacity existing screens, MGD	6.8
Capacity Phase II screen, MGD	9.6
Capacity by-pass channel, MGD	9.6
Gravity Influent	30"
Opening Space	¼"

<u>Raw Sewage Pump Station:</u>

Peak Hourly Flow, gpm	5,208
Number of existing units	2
Type	Flow Serve Centrifugal
Existing Pump Range (Each), gpm	580 - 1,160
Number of proposed units	2
Type	Wemco Hidrostal
Phase II Pump Range (Each), gpm	1,625 -3250
Force Main Dia.	18"
Velocity, ft/sec	7.31
Maximum Capacity of P.S.	5,570 gpm

Single Stage Nitrification:	
Design	Oxidation Ditch
Design Average Flow	3.0 MGD
Peak Hourly Flow	7.5 MGD
BOD_5	220 mg/l
BOD_5	5,505 lbs/day
TSS	260 mg/l
TSS	6,505 lbs/day
NH_3-N	35 mg/l
NH_3-N	875 lbs/day
Number of channels	3
Side Water Depth	14 feet
Channel Width Inside	20 feet
Channel Width Middle	20 feet
Channel Width Outside	20 feet
Detention Time	19 hours
Total Volume	2,357,509 gal
Solid Production	5,505 lbs/day
Solids Inventory	78,730lbs
Sludge Age	14.3 days
F/M Ratio	0.07
Oxygen Required	482 lbs/hr
Oxygen Supplied	900 lbs/hr
MLSS Diversion Structure:	
Number of Units	1
Design Average Flow	6.4 MGD (DAF + RAS)
Peak Hourly Flow	10.7 MGD (PHF + RAS)
Influent MLSS	24"
Effluent MlSS	24"
Final Clarifier:	
Number of Units	2
Design	Hydraulic Differential
Clarifier Diameter	85ft
Design Average Flow (each)	1.5 MGD
Peak Hourly Flow (each)	3.75 MGD
Surface Loading Rate	660 gal/ day/ sq.ft.
Solids Loading Rate	22 lbs/ day/ sq.ft.
Weir Loading Rate	15,090 gal/ day/ ft.

Chlorine Contact Tank:	
Number of Tanks	2
Design	12' x 56' x 8.5' SWD
Total Volume, cuft	11,425
Total Volume, gal.	85,460
Design Average Flow	3.0 MGD
Peak Hourly Flow	7.5 MGD
Detention Time @ Peak, Min.	16.4
RAS WAS Pump Station:	
Design	Pre-Rotation
Number of Pumps	6
	4 - RAS
	2 - WAS
RAS Pump Capacity	1,500 gpm @ 33 ft (Each)
WAS Pump Capacity	250 gpm @ 26.5 ft (Each)
RAS Force Main Dia.	14"
WAS Force Main Dia.	6"
Biosolids Stabilization:	
Design	Aerobic Digestion
Waste Activated Sludge:	
WAS Production @ 0.75%	88,000 gpd
Continuous Sludge Thickening:	
Water from thickening WAS to 3%	66,000 gpd
Water from Phase 1 VSS Destruction	6,270 gpd
Total Water Removed	72,270 gpd
Membrane Flux Rate	5.1 gpd/ sq.ft.
Area per Plate	8.6 sq.ft
Number of Plates required	1,600

Phase 1 Aerobic Digestion:	
Volume of Anoxic Tank	27,000 cu.ft.
Volume of Thickener	13,500 cu.ft.
Volume of Digester #1A & #2	35,837 cu.ft.
Total Volume	76,338 cu.ft.
Detention Time @ 3%	29 days
VSS Reduction @ 20 °C	38%
Aeration Required @ 35 °C	3,000 scfm
Phase 2 Aerobic Digestion:	
Volume of Digester #2	27000 cu.ft.
Volume from Dig #1	2,102 cu.ft./day
Detention Time	17 days
VSS Reduction @ 20 °C	46%
Aeration Required @ 20 °C	1,075 scfm
Phase 3 Aerobic Digestion:	
Volume of Digester #3	2,700 cu.ft.
Volume from Dig #2	2,102 cu.ft./day
Detention Time	17 days
VSS Reduction @ 20 °C	50 %
Aeration Required @ 20 °C	1,075 scfm
Phase 4 Aerobic Digestion – Sludge Storage	
Volume of Digester #4	2,700 cu.ft.
Volume from Dig #3	2,102 cu.ft./day
Detention Time	17 days
VSS Reduction @ 20 °C	50%
Aeration Required @ 20 °C	1,075 scfm
Sludge Dewatering:	
Number of Pumps	2
Capacity	120 GPM
Number of BFP	2
Capacity	120 GPM @ 2.5%
Hours of Dewatering	7.6 hrs/ week/ unit

Item No.	Description	Quantity	Unit	Unit Price	July-03
GENERAL CONDITIONS					
	Dumpsters	13	Each	$400.00	$5,200.00
	Job Trailer	12	Mo.	$800.00	$9,600.00
	Eng Trailer	12	Mo.	$800.00	$9,600.00
	Supervision	12	Mo.	$12,000.00	$144,000.00
	Surveying	1	Each	[illegible]	[illegible]
	Record Drawing	1	Each	$5,000.00	$5,000.00
	Allowances				
	Bonds & Insurance	1	Each	$80,000.00	$80,000.00
	Overhead & Profit	1	Each	$800,000.00	$800,000.00
	TOTAL GENERAL CONDITIONS				[illegible]
SITEWORK					
	Top Soil Stripping & Stockpile	521	Cu. Yd.	[illegible]	[illegible]
	Top Soil Respread	521	Cu. Yd.	[illegible]	[illegible]
	Demo Sludge Drying Beds	1400	Cu. Yd.	[illegible]	[illegible]
	Sidewalk	9000	Sq. Ft.	[illegible]	[illegible]
	Retaining Wall	500	Sq. Ft.	[illegible]	[illegible]
	Curbs	500	Sq. Ft.	[illegible]	[illegible]
	Concrete Steps	[illegible]	Each	[illegible]	[illegible]
	Bit Surface 1-1/2"	131	Ton	[illegible]	[illegible]
	Bit Base 2-1/2"	[illegible]	Ton	[illegible]	[illegible]
	Bit Prime Coat	1800	Gal.	[illegible]	[illegible]
	10" Agg Base	[illegible]	Sq. Yds.	[illegible]	[illegible]
	Geotextile Fabric	[illegible]	Sq. Yd.	[illegible]	[illegible]
	Pavement Removal	[illegible]	Sq. Yd.	[illegible]	[illegible]
	Sidewalk Removal	500	Sq. Yd.	[illegible]	[illegible]
	Storm Sewer Removal	[illegible]	Lin. Ft.	[illegible]	[illegible]
	[illegible] Removal	[illegible]	Each	[illegible]	[illegible]
	Drainage Structure Removal	[illegible]	Each	[illegible]	[illegible]
	Tree Removal	50	In. Dia.	[illegible]	[illegible]
	Seeding	[illegible]	Acre	[illegible]	[illegible]
	Planting	66	Each	$400.00	$26,400.00
	Earthwork	20870	Cu. Yd.	[illegible]	[illegible]
	Sodding	[illegible]	Sq. Yd.	[illegible]	[illegible]
	Straw Bales	20	Each	[illegible]	[illegible]
	Silt Fence	2000	Lin. Ft.	[illegible]	[illegible]
	Rip Rap	360	Sq Yd	$15.00	$5,400.00

Item No.	Description	Quantity	Unit	Unit Price	July-03
[illegible]	Filter Fabric	100	Sq Yd	[illegible]	[illegible]
[illegible]	Fence Remove and Replace	400	Lin Ft	[illegible]	[illegible]
[illegible]	Chain Link Gate Removal	1	Each	[illegible]	[illegible]
[illegible]	Electrical Duct Bank Relocation	1100	Lin Ft	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	Each	[illegible]	[illegible]
[illegible]	Electrical Service	12	Lump Sum	[illegible]	[illegible]
[illegible]	Fiber Optic Cable	1000	Lin Ft	[illegible]	[illegible]
[illegible]	[illegible]	1	Lump Sum	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	18" Raw Sewage Force Main	500	Lin Ft	[illegible]	[illegible]
[illegible]	8" Water Main	2400	Lin Ft	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	8" RCP	[illegible]	Lin Ft	[illegible]	[illegible]
[illegible]	12" RCP	200	Lin Ft	[illegible]	[illegible]
[illegible]	36" RCP	[illegible]	Lin Ft	[illegible]	[illegible]
[illegible]	Inlet	15	Each	[illegible]	[illegible]
[illegible]	Catch Basin	1	Each	[illegible]	[illegible]
[illegible]	Special Structure	1	Each	[illegible]	[illegible]
[illegible]	12" FES	1	Each	[illegible]	[illegible]
[illegible]	36" FES	2	Each	[illegible]	[illegible]
[illegible]	Trench Backfill	187	Yd	[illegible]	[illegible]
[illegible]	Fittings	9200	Lb	[illegible]	[illegible]
	TOTAL SITEWORK				[illegible]