NASSAU COUNTY, NEW YORK

BASIC DESIGN DATA REPORT

FOR

CEDAR CREEK
WATER RECLAMATION FACILITIES

NOVEMBER
1975

CONSOER, TOWNSEND & ASSOCIATES
CONSULTING ENGINEERS
NEW YORK CHICAGO

Board of Supervisors Meeting Rm
5th floor
County Executive Bldg.

EDF

85-810285

12-26-85 RL

CONSOER, TOWNSEND & ASSOCIATES
CONSULTING ENGINEERS

360 E. GRAND AVENUE
CHICAGO, ILLINOIS 60611
(312) 337-6900

November 28, 1975

Mr. H. John Plock, Commissioner
Department of Public Works
Nassau County Executive Building
Mineola, New York 11501

Re: Cedar Creek Water Reclamation Facilities

Dear Mr. Plock:

In accordance with our Agreement for Engineering Services with the County of Nassau, dated October 28, 1974, we are pleased to submit to you this "Basic Design Data Report" which incorporates the changes in design that have been made since the completion of the Environmental Protection Agency Research Grant, entitled "Correlation of Advanced Wastewater Treatment And Ground Water Recharge." This Report was prepared in compliance with Division A of the Agreement and will serve as a companion document to the previously submitted plans and specifications.

As soon as you have had an opportunity to review this Report we will be pleased to meet with you and your staff to review all data and to clarify any matters in question.

Respectfully submitted,
CONSOER, TOWNSEND & ASSOCIATES

William J. Beckman

Wallace J. Beckman

Raymond J. Avendt

Raymond J. Avendt

David L. Herrin

David L. Herrin

WJB/RJA/DLH:jl

I. HISTORICAL INTRODUCTION

The increasing potable water requirements of Nassau and Suffolk Counties, Long Island, are being met by a limited ground water supply. Presently, the Counties are dependent upon this ground water as their sole source of potable water. Numerous studies have shown that the present water requirements are approaching the maximum allowable withdrawal rates that can be sustained without depleting the aquifer or noticeably lowering the water table. The disposal of treated wastewaters into the ocean, bays or Long Island Sound will accelerate the depletion of the fresh water supply due to non-replenshment of the aquifer and saline contamination. The magnitude of the hydrologic system deficiency for Nassau County alone has been projected to be 92 million gallons per day by 1990 and 177 million gallons per day by 2020.

Various water resource management alternatives have been investigated on Long Island in order to deal with this potentially critical water supply situation. One major area of interest is water reclamation and ground water recharge. The ground water quality would be maintained and in some instances enhanced by adequately designed and operated reclamation facilities, while the aquifer depletion would be curtailed by recharge.

Nassau County is currently in the middle of a water reclamation recharge project involving three phases over an approximate ten-year period.

Phase One of the project was the completion of an Environmental Protection Agency Research Grant entitled "*Correlation of Advanced Wastewater Treatment and Ground Water Recharge*." The objectives

accomplished during Phase One (July, 1972 to August, 1973) were:

1. The investigation of methods of recharging reclaimed water feasible on Long Island (injection wells, open basins, stream flow augmentation, etc.).

2. The establishment of water quality criteria for water reclamation-recharge projects based on applicable water quality standards, method of recharge, rechargeability of reclaimed water, existing ground water quality and public health considerations. The reclamation recharge water quality standards are equal to the U.S.P.H.S. Drinking Water Standards or the New York State Standards for Sources of Water Supply with respect to all bacteriological, physical and chemical characteristics. In the event that the reclaimed water quality does not meet the standards at any one of the seven quality control checkpoints, it will be automatically disposed of to the ocean.

3. To investigate the advanced wastewater treatment unit processes available and to determine the integrated water reclamation process scheme capable of producing the required water quality.

4. The preliminary design of a reclamation-recharge facility which would serve to demonstrate the performance and reliability of the system on a 4 million gallon per day basis for 3 to 5 years.

Phase Two of the project was initiated in October, 1974, following a favorable review of the Research Grant Report by both the U.S. E.P.A. and the New York Department of Environmental Conservation. Phase Two entails the preparation of detailed plans and specifications for the 5.5 mgd water reclamation-recharge facility

and its construction at the recently completed 45.0 mgd Cedar Creek Water Pollution Control Plant. (Formerly known as the Wantagh Water Pollution Control Plant) The purpose of this Basic Design Data Report is to present the design criteria for the proposed 5.5 mgd Cedar Creek Water Reclamation - Recharge Facility. The Basic Design Data Report will serve as a companion document to the contract plans and specifications, which will be completed in November, 1975. The design utilizes existing treatment units available at the Cedar Creek Plant wherever possible. Any additional treatment units have been designed on a permanent basis and made readily expandable for future large scale operations.

Construction of the facility could begin as early as July, 1976 and be completed in January, 1978.

Phase Three will be the Demonstration Project which will involve a long term assessment of the reliability economics and environmental response of a large scale water reclamation-recharge system will be optimized through surveillance of not only the reclaimed water quality but the stresses and deviation in the ground water quality and composition resultant from chemical, biochemical and physical reactions in the aquifer.

The results of the overall project are anticipated ~~to~~ provide a prudent and environmentally sound water resource management method of protecting and conserving ground water supplies not only for Nassau County but for application wherever critcal water supply situations exist.

II. WATER RECLAMATION FACILITY *

A. Wastewater Flows, Characteristics and Effluent Criteria

The utilization of a portion of the newly constructed Cedar Creek Water Pollution Control Plant for the Water Reclamation - Recharge Facility was a major objective of the project. The plant is an activated sludge type designed to treat an average flow of 45.0 million gallons per day (MGD). See Sheet D-1 of the Contract Drawings for a plan view of the plant. Based on schedules of construction of lateral sewers, house connections and incorporation of Freeport wastewaters with the Southwest Sewer District No.3 flows, the anticipated average wastewater flows may not exceed 28.0 MGD prior to 1983. Considering an average flow condition of 28.0 MGD, 5.5 MGD would be diverted from the effluent of the grit chamber and serve as the influent of the reclamation facilities described herein. The remainder of the flow (22.5 MGD) would receive secondary treatment utilizing one-half of the plant prior to ocean disposal.

Table No. II-1 has been included to show the anticipated raw wastewater characteristics for the water reclamation facility. The raw wastewater characteristics were developed based on the present wastewater characteristics at the Cedar Creek and Freeport Water Pollution Control Facilities. Where possible, the results of the United States Geological Survey grab sample analyses, which include metals, organics and pesticide concentrations, have been included in the tabulation.

* *All Contract Drawings referred to in this Chapter are contained in the Reclamation Plant Drawings.*

TABLE II-1

Plant Influent Wastewater Characteristics

Alkalinity	190	Chromium (hexavalent)	0.9	Sodium	na	Dimethonate	na	- Physical -	
ABS	10	Copper	0.2	Sulfate	70	Endrin	0.0	Color	na
Aluminum	0.7	Cyanide	0.04	TOC	155	Ethion	na	TON	na
Ammonia	22	Flouride		Uranyl Ion		Heptachlor	0.0	TDS	500
Arsenic	0.1	Hardness (as $CaCO_3$)	102.0	Zinc	0.7	Hep. Epoxide	0.0	Turbidity	250
Barium	0.2	Iron & Manganeese	2.0	- Microbiological -		Herbicides		Entrained Air	na
BOD_5	200	Lead	0.2	Coliforms	5.0×10^7/100ml	Lindane	na	Sus. Solids	200
Boron	0.3	Mercury	0.001	- Pesticides -		Methoxychlor	0.0	VSS	160
Cadmium	0.01	Nitrogen, Total	35	Aldrin	0.0	Org. PO_4^+ Carbonates	na	pH	7.4
Calcium		O_2 Consumed	na	Azodrin	na	PCB	0.2?		
CCE	8.0	Phenols	0.15	Chlordane	0.0	Toxaphene	0.0		
COD	350	Phosphorus	16	DDT	0.0	2-4-D Whole Water	0.4		
Chloride	165	Selenium	0.02	Dichlorous	na	2-4-5-T Whole Water (silvex)	0.0		
Chlorine Res. Free	na	Silver	0.01	Dieldrin	0.0				

All Values mg/l unless stated otherwise

na - not available

• concentration exceeds proposed standards

The anticipated water reclamation facility effluent quality should meet or approach the desired effluent quality shown in Table II-2. The effluent total nitrogen concentration from this facility should approximate the 2.0 mg/l level, and not exceed 3 mg/l; while phosphorus should not exceed 0.3 mg/l with 0.1 mg/l normally obtained.

The total dissolved solids and chloride content of the raw wastewater approaches the New York State standards of 500 mg/l and 250 mg/l, respectively. However, dissolved solids removal is not being incorporated in the design, since the effluent TDS concentration should be less than 400 mg/l, as at Bay Park.

B. Existing Cedar Creek Water Pollution Control Plant

The newly constructed Cedar Creek Water Pollution Control Plant is an activated sludge type with an average design flow of 45.0 MGD. The plant site is located immediately east of the right-of-way line of the proposed Wantagh-Oyster Bay Expressway. The site is about one mile south of Merrick Road and about four miles north of Jones Beach. The site contains 75 acres. The plant serves Sewage Disposal District No. 3 located in the southeast section of Nassau County.

The major process units incorporated into the plant include:

Influent Screen Chamber - The raw wastewater is conveyed to the plant through a 108 inch inside diameter sewer. The wastewater is passed through two (2) automatically cleaned screens or two (2) hand raked screens having one inch clear openings. Automatic heavy duty grinding units grind the screenings and return the

TABLE II-2

Effluent Quality Standards

Alkalinity (as $CaCO_3$)	na	Chromium (hex.)	0.05	Sulfate	250	Dimethonate	0.002	- Physical -	
ABS	0.5	Copper	0.2	TOC	3.0	Endrin	0.0002	Color	15 units
Aluminum	0.1	Flouride	1.5	Uranyl Ion	5.0	Ethion	0.02	TON	3 units
Ammonia	2.0	Iron & Manganese	less than 0.3	Zinc	0.3	Heptachlor	0.0001	TDS	500
Arsenic	0.05	Lead	0.05	- Microbilogical -		Hep. Epoxide	0.0001	Turbidity	0.5 JTU
Barium	1.0	Mercury	0.002	Coliforms-max	4/100ml	Herbicides	0.1	Entrained Air	none
BOD_5	2.0	Total Nitrogen	3.0		-Ave.1/100ml	Lindane	0.004	Sus.Solids	1.0
Boron	1.0	O_2 Consumed	2.0	- Pesticides -		Methoxychlor	0.035	VSS	na
Cadmium	0.01	Phenols	0.001	Aldrin	0.017	Org. $PO_4^{\ddagger}$ Organates	0.1	pH	6.5 - 8.5 Units
Calcium	less than Sat. Conc.	Hardness (as $CaCO_3$)	na	Azodrin	0.003	PCB	na		
CCE	0.2	Phosphorus	0.1	Chlordane	0.003	Toxaphene	0.005		
COD	na	Selenium	0.01	DDT	0.042	2-4-D Whole Water	0.1		
Chloride	250	Silver	0.05	Dichlorous	0.01	2-4-5-T Whole Water	0.01		
Chlorine Res. (free)	1.0	Sodium	less than 50% of Cat. or 29 mg/l	Dieldrin	0.017				

All values mg/l unless otherwise Stated

na - not available

material to the raw wastewater flow. The screen chamber contains signaling, warning and protective devices such as high level alarms, jam bars, manual start-stop controls and overload switches. Each of the four screen channels may be closed off in the front and rear and dewatered for repair or maintenance. An automatic sampling pump, mounted in the lower section of the chamber ahead of the screens, pumps a continuous representative sample of raw wastewater to a refrigerated sampler which is paced from the raw wastewater metering equipment.

Raw Wastewater Wet Well and Pumping Equipment - The screened raw wastewater flows by gravity to the divided wet well. Five (5) variable speed pumps have been installed providing a capacity of 1.2 MGD to 90.0 MGD, with 76.0 MGD stand-by capacity.

Grit Chambers - Sand and other heavy inorganic solids contained in the wastewater are removed in the two (2) aerated type degritting chambers. Each chamber is 28.0 feet wide, 70.0 feet long and has an average liquid depth of 14.5 feet, providing a volume of 28,400 cubic feet and 6.8 minutes detention time based on the design average flow. Air is introduced through swing type headers to diffuser units attached to a header located about 2 feet above the bottom of the chambers. 2,000 CFM of air capacity has been provided for use in the two aerated grit chambers. Settled grit is then washed and is disposed of on the plant site. The grit chambers are covered with a superstructure with all ventilation air deodorized prior to discharge to the atmosphere.

Primary Settling Tanks - The wastewater flows by gravity to six (6) primary settling tanks. Each tank is 41.5 feet wide, 160 feet long and has an average liquid depth of 10.0 feet. The

volume of the six tanks of 384,000 cubic feet (2,870,000 gallons) provides a detention period of 1.53 hours based on the average design flow of 45.0 MGD. Each tank is equipped with straight line longitudinal and cross collectors which scrape the settled solids to sludge hoppers at the influent ends of the tanks and skim floating materials to revolving type scum troughs located near the outlet end of the tanks. The sludge is withdrawn from the hoppers and pumped by torque flow type sludge pumps to either the digesters or the thickeners through glass lined cast iron pipe. Scum and other floating material flows by gravity to ejectors which are located in the primary tank gallery. The scum and floating material are ejected through a glass lined cast iron effluent line to the thickener or to the digesters.

The control equipment for the variable speed, torque flow type raw sludge pumps consists of sludge density measuring equipment and all required appurtenances which permit automatic withdrawal of the sludge from the sludge hoppers at a predetermined solids concentration.

Sludge sampling stations are located near the sludge pumping equipment to enable plant operating personnel to obtain grab or composite samples for checking or calibration of the density meter. Each settling tank is provided with one sludge pump. During periods of pump repairs the pump serving the adjacent tank can be used.

Effluent from the settling tanks flows over a series of adjustable troughs, having V-notched side plates, installed at the outlet end of the tanks. The total net length of weirs provided in each tank is about 500 linear feet. Based on a 45.0 MGD

average rate of flow, the tank overflow rate is approximately 1,170 gallons per square foot per day, with a weir overflow rate of 15,000 gallons per linear foot per day. A parshall flume is provided in the effluent channel of each primary settling tank for balancing of flows into each tank.

The expected BOD_5 and suspended solids removal based on the design flow were 30% and 45%, respectively. The entire primary tanks and related control facilities are covered with a building. All ventilation air is deodorized before being emitted to the atmosphere.

Aeration Tanks - The settled wastewater flows by gravity from the primary settling tanks to three (3) four-pass aeration tanks. Each pass is 25.0 feet wide, 285 feet long and has an average liquid depth of 15.0 feet. The total volume of the three tanks of 1,284,000 cubic feet (9,600,000 gallons) provides, when operated as a conventional activated sludge plant, a detention time of 5.14 hours based only on a settled wastewater flow of 45.0 MGD or 3.85 hours based on a settled wastewater flow of 45.0 MGD and 33% return activated sludge rate. Provisions have also been incorporated into the design for operation of the plant in accordance with the principles of step-aeration.

Air is introduced through diffuser units located about 2 feet above the bottom of the tanks. Air to the diffusers is carried through swing type headers. A spray water system for foam control has been provided in the aeration tanks as well as in the aerated channels.

The return activated sludge is withdrawn under controlled conditions from the final settling tanks. Rate of flow controllers

allow withdrawal of return activated sludge at equal rates from each final tank or the return sludge rates may be varied. The total return activated sludge flow is withdrawn into two chambers from which it is pumped by variable speed non-clog pumps into the first pass of each aeration tank in service. Rate of flow controllers permit the equalization or the varying of the rates of return activated sludge to each of the aeration tanks. Rate of flow of waste activated sludge to the sludge thickeners is controlled by flow regulating and metering equipment.

Two (2) variable speed and two (2) constant speed, 16,500 CFM, positive displacement type blowers and one (1) 5,000 CFM, positive displacement type blower with a variable speed drive are available, providing 71,000 CFM. All blowers are equipped with electric motor drives.

The mixed liquor effluent from the aeration tanks flows to the aerated mixed liquor channel from which two separate conduits carry the flow to the final settling tanks.

Final Settling Tanks - Rate of flow controllers distribute the flow equally to each of the six (6) final tanks. Each tank is 100.0 feet in diameter and has a side water depth of 12.0 feet, providing a total surface area of 47,100 square feet and a total volume of 565,200 cubic feet for the six tanks. Each tank is provided with a peripheral feeding raceway containing circular orifices located in the invert of the raceway. A center effluent trough having V-notched side weir plates mounted on the outside is provided. Total length of the weir plates in each tank is 120.0 feet.

Settled sludge is withdrawn from the bottom of the tanks through tubular type arms having rectangular cross-section, containing elliptical orifices of varying spacing and size, arranged to provide a uniform sludge withdrawal rate. The surface settling rates based on the average flow condition of 45.0 MGD is 1,000 gallons per square foot per day. The weir overflow rate based on the same flow condition is 39,600 gallons per linear foot per day.

Plant Effluent Screening Chamber - Effluent from the final settling tanks flows through two (2) traveling water screens, having 3/8 inch clear openings, which remove part of the solid material which has not settled in the final settling tanks. Screenings are pumped to the thickening tanks.

Chlorination Facilities - Chlorination facilities for effluent disinfection, influent odor control and prevention of sludge bulking are provided. Because of the length and diameter of the outfall sewer, a chlorine contact chamber was not required to provide adequate detention time. Chlorine feed equipment with a capacity of 8,000 pounds per day plus stand-by equipment have been provided.

Effluent Pumping - The elevations of the plant units were set so that maximum flows from the final settling tanks could be discharged by gravity through the outfall sewer at all times except during maximum ocean levels. Pumping capacity has been provided to discharge at a 90.0 MGD rate during maximum ocean levels, with stand-by capacity equal to the largest operating unit.

Sludge Disposal Facilities - Sludge thickening facilities have been provided to thicken waste activated and primary sludge (if required) in order to decrease required digester capacity. The

four (4) floatation type thickeners, each 20.0 feet wide, 75.0 feet long and 10.5 feet deep, were designed based on a net design loading rate of 10 pounds of waste activated sludge per square foot.

Primary sludge and thickened waste activated sludge are pumped to the digesters. Three (3) digesters have been provided, each having a 105.0 foot inside diameter with a 26.5 foot side water depth. On the basis of plant design criteria, two digesters used as primary units provide 14 days detention at average solids conditions. The third digester is used as a secondary digester. One (1) sludge storage tank of the same dimensions has been provided to store 18 days accumulation of digested sludge. All the sludge digesters and the storage tank are equipped with floating covers and external type heat exchangers. Digested sludge is pumped to the District No. 2 Bay Park Plant for ultimate disposal. The Bay Park plant is located approximately nine miles due west.

Power Generation Facilities - Engine driven generator units generate all the plant's required power. All equipment including pumps and low pressure air blowers are motor driven. All engines are of the high compression diesel cycle type capable of operation on a dual fuel of sewage gas and fuel oil, a dual fuel of natural gas and fuel oil or on diesel fuel oil only. Heat recovered from cooling and exhaust systems is used for space and sludge heating and domestic hot water heating. Supplemental heat requirements are furnished by boilers .

Process Control - Extensive computer facilities capable of direct digital control and complete data acquisition and logging have been incorporated into the design. The Process Instrumentation Diagrams which are included in the Contract Drawings illustrate the

control, instrumentation and metering devices available at the Cedar Creek Plant and their integration with the reclamation facilities.

C. Basic Design Data

1. General Discussion

The basic approach taken for design of the Cedar Creek 5.5 MGD water reclamation system was incorporation of the best available technology, combined with maximum utilization of the existing plant facilities to insure consistent production of the high quality effluent desired for recharge. To obtain this consistent effluent quality, sizing of the various unit operations has been based on reasonable, conservative design parameters taken from our experience and the literature. Sufficient operational flexibility is available in the proposed facility and modified existing facilities to enable optimization of operation and control. Additional criteria considered essential in the design of a facility demonstrating various advanced wastewater treatment process schemes were: continuous monitoring and alarm systems, an emergency power supply (already provided at the Cedar Creek Plant), short term retention of reclaimed waters, ability to have long term emergency disposal, stand-by equipment and multiple units.

The design of the reclamation facilities was also analyzed from a cost-effective viewpoint. Considering the 5.5 MGD scale, the treatment units necessary for the proposed facility could be utilized in a future full scale reclamation program for Nassau County. The utilization of these treatment units after the demon-

stration program would extend the service life of such equipment, thus further increasing the returns on the demonstration program expenditures. Wherever possible capital construction costs associated with equipment or materials that could not be incorporated into a full scale recharge facility for the Cedar Creek plant have been reduced.

The proposed water reclamation-ground water recharge facilities are shown schematically in Sheets F-1 and F-3 of the Contract Drawings. Appendix A entitled *Design Criteria of Reclamation Facilities*, should be consulted for basic design data and design calculations.

2. Chemically Aided Primary Sedimentation

As shown on Sheet F-1 of the Contract Drawings entitled *Flow Diagram Wastewater-Two Stage Secondary*, the influent to the reclamation facility will be 5.5 mgd of screened and degritted wastewater withdrawn under controlled conditions from the existing plant primary influent channel. The wastewaters will then be conveyed to a proposed rapid mix tank constructed immediately east of the existing primary channel. The rapid mix tank will be used for dispersal of the coagulant (lime). The automatic lime slaking and feeding equipment, a slurry storage tank, lime storage facilities and the required control and monitoring devices will be located in the Sludge Thickening Building. The rapid mix tank volume will be 4,190 gallons (560 cubic feet), providing a residence time of 1 minute based on the design average flow rate of 5.5 MGD. At peak flows, the volume will be 5690 gallons (760 cubic feet), providing a residence time of 1 minute based on 1½ times the average flow rate

and a side water depth of 9.5 feet. The chemical feed equipment will be capable of dosing up to 350 mg/l of lime. The pH of the rapid mix tank will be maintained between 9.5 and 11.0, as determined from operational experience. Lime storage capacity equal to 7 days of maximum dosage will be provided. Provisions have also been made for the introduction of ferric chloride into the rapid mix tank, if desired.

The effluent from the rapid mix tank will be discharged by gravity to a bulk-headed portion of the influent channel. The bulk-headed portion of the influent channel (85 feet long) will be divided between primary tanks 5 and 6. A stop gate will be provided to allow cross-flow when open. In each of the bulk-headed portions of the influent channel, horizontal shaft slow speed turbine mixers will be installed for flocculation. Each of the turbine mixers will consist of four (4) feet in diameter paddle assemblies. The detention time afforded by the whole flocculation chamber will be approximately 12.5 minutes, or 6.25 minutes when the stop gate is closed.

Provisions will be made for the introduction of an anionic polymer into the flocculation chambers when ferric chloride is being used.

The flocculated effluent will then flow to the head end of the No. 6 Primary Tank. Alternate process piping will be provided to allow for introduction of the coagulated wastewater into the head end of the No. 5 Primary Tank in the event of maintenance or repair operations on the No. 6 Primary Tank.

The lime precipitation in the primary clarifier and subsequent alum precipitation in the final clarifier has been provided in order to facilitate the high removals of phosphorus required. Lime precipitation will not only remove a significant portion of the phosphorus, but increased amounts of BOD_5, suspended solids and heavy metals. This will provide optimization and protection of the biological nitrification system.

One of the existing primary clarifiers(described previously in this section), having a surface area of 6,640 square feet, will provide a surface loading rate of 830 gallons per day per square foot at the design average flow rate of 5.5 mgd. The weir overflow rate will be 11,000 gallons per linear foot of weir per day. The detention period to be provided is approximately 2.1 hours.

The chemical sludge will be collected and pumped using the existing equipment to the digesters where it will be combined with the raw and waste activated sludge of the main plant. Provisions have been made for the recycling of raw sludge to the rapid mix tank for seeding purposes.

The degrees of treatment anticipated in the chemically aided primary treatment system are a 60 percent to 70 percent removal of suspended solids and a 45 percent to 55 percent removal of BOD_5, depending on the amount of coagulant employed.

Provisions have been made for the addition of CO_2 or acid to the primary effluent line for pH adjustment, if required.

If is anticipated that the chemically aided primary sedimentation will reduce the total phosphorus by approximately 75 percent.

Automatic sampling of the chemically aided primary effluent flow will be provided to monitor the process efficiency.

3. Secondary Treatment

Two Stage Secondary - The settled wastewater from the primary tank will flow by gravity to the existing aeration tanks for secondary treatment as shown on Sheet F-1 of the Contract Drawings, entitled *Two Stage Secondary Treatment*. The design allows for the operation of the secondary system as a two stage suspended growth system, similar to those investigated at Freeport and Contra Costa, with combined carbon oxidation - nitrification, intermediate clarification, anoxic denitrification, oxidation-stabilization and final clarification. The process piping required to convert the secondary to a three stage system similar to that used at Mannassas and Blue Plains, having, in addition to the above, an intermediate clarifier between the high rate activated sludge and the nitrification reactors is to be provided (refer to Sheet F-3 of the Contract Drawings). The ability to operate a separate sludge for nitrification would assure the desired effluent quality and hasten recovery in the event of plant upset. See Sheets F-2 and F-4 of the Contract Drawings for detailed modifications to the existing Cedar Creek return sludge system for two stage and three stage systems, respectively.

Both secondary treatment modes of operation are based on a constant 5.5 mgd flow and average wastewater constituents, with the exception of the aeration system for which peak BOD_5 and ammonia concentrations have been utilized. During the course of the proposed demonstration project the diurnal flow patterns experienced in the main plant will be maintained in order to investigate system stability under actual operating conditions.

As shown in Sheet G-1M of the Contract Drawings, the secondary treatment facilities of the existing Cedar Creek facility contain three (3) four-pass aeration tanks. The reclamation

facility proposed in this report will use portions of Aeration Tank Nos. 2 and 3.

As a two stage secondary system for BOD_5 and nitrogen removal, the settled wastewater from the primary tank will flow by gravity through the existing primary effluent metering equipment to the head of the first pass of Tank No. 3 of the existing secondary system. Utilizing the entire pass number one and pass number two, the volume provided will be approximately 180,000 cubic feet or 1,360,000 gallons, when operated with a 12.75 foot side water depth. The detention time will be 5.9 hours based on the average design rate of flow, 5.5 mgd. The combined carbon oxidation-nitrification facilities were designed based on a minimum wastewater temperature of 12° C, consistent with the data obtained from other treatment facilities in the immediate area. The operation will be conventional, with a mixed liquor suspended solids level of 3,180 mg/l, of which 85 percent are assumed volatile. Based on a BOD_5 concentration in the secondary influent of 100 mg/l, the calculated F/M ratio is 0.15 lb. BOD_5/day per lb MLVSS. The sludge age provided is in excess of 17 days. The existing air header contained in the Y-wall between passes 1 and 2 is adequate to provide for the increased oxygen demands of the combined carbon oxidation - nitrification system. Pertinent data regarding the BOD_5 and ammonia nitrogen oxidation rates are presented in the appendix.

The mixed liquor flow will be discharged through the existing number two pass effluent line to the final tank No. 5 influent channel. This flow will be isolated from the remainder of the plant flows by closing an existing aeration tank effluent channel butterfly gate.

One of the three final clarifiers, Final Tank No. 5, will be used, providing an overflow rate of 700 gallons per day per square foot. The solids loading on the clarifier will be 27 lbs. per day per square foot. Assuming a 1.0 percent underflow concentration, the average recycle rate will be 47 percent of the design rate of flow. Activated sludge removed from the clarifier will be returned to the aeration tank utilizing two proposed pumps, meters, and a new 16 inch diameter line. The effluent from the first stage will be mixed under controlled conditions with the return sludge from the denitrification final tank (No.7) and pumped through the existing return activated sludge pumps to the third pass of aeration tank No. 3, which will be operated as an anoxic suspended growth denitrification system. Denitrification and rising sludge in the clarifier are not anticipated due to the extremely low carbon concentrations available in the combined carbon oxidation-nitrification effluent.

The BOD_5, suspended solids and ammonia nitrogen removals for the combined carbon oxidation-nitrification system are 95 percent, 85 percent and 90 percent, respectively.

Existing Aeration Tank No. 3, passes three and four, will be used for denitrification and post aeration stabilization.

The denitrification and post aeration - stabilization tank volume is 92,240 cubic feet or 690,000 gallons, providing a detention time of 3.0 hours (2.0 hours denitrification and 1.0 hour post aeration - stabilization) based on the design average flow. Methanol will be added to the tank influent flow. The methanol

addition has been based on a ratio of 3.54 pounds of methanol per pound of nitrate-nitrogen applied. The methanol feeders will be automatically controlled from continuous nitrate and plant flow measurements.

A mixed liquor level of 4,200 mg/l having a volatile content of at least 64 percent will be required to assure complete denitrification at minimum temperatures. The reduced volatile content takes into account the addition of alum for additional reduction of the phosphorus content in the post aeration stabilization tank.

The mixed liquor from anaerobic denitrification will flow through a post aeration-stabilization zone to be provided in the last 2/3 of the fourth pass of Tank No. 3. The aeration capacity provided allows for a 50 percent methanol overdose. The purpose of the post aeration-stabilization zone is first, to remove any supersaturated nitrogen gas to avoid rising sludge problems in the final clarifier; second, to provide a mixing zone for the addition of alum; and third, to provide an additional aeration period for the removal of excess methanol. The return sludge rate will be 65 percent based on a 1.25 percent return sludge solids concentrations.

The mixed liquor will be conveyed to Final Tank No. 7 for final clarification. The overflow rate will be 700 gallons per day per square foot. The solids loading will be 36 pounds per day per square foot. Minimal production of biological sludge is anticipated; however, provisions have been made to waste sludge to the combined carbon oxidation-nitrification

system or to the digesters. Correspondingly, the ability to waste solids from the combined oxidation-nitrification unit to the denitrification unit is also available.

Three Stage Secondary - The alternate secondary sludge system would be a three stage system whose operation would be similar to the above except for the use of an intermediate clarifier between the high rate activated sludge unit and the nitrification unit (Refer to Sheet F-3 of the Contract Drawings). In this system, the settled wastewater from the primary tank will flow by gravity to the high rate activated sludge unit to be located in the first pass of existing Aeration Tank No. 3.

The mixed liquor from the activated sludge unit will be pumped to the High Rate Activated Sludge Final Tank, existing Tank No. 5, for clarification.

Activated sludge removed from this clarifier (Final Tank No. 5) will be returned to the aeration tank utilizing new return sludge pumps, meters, and a new 16 inch diameter line. The effluent from the final tank will be mixed under controlled conditions with the return sludge from the nitrification final tank (No. 7) and pumped through the existing return activated sludge pumps to passes number 3 and 4 of aeration tank No. 3, which will be operated as the nitrification facility for this alternate system.

The nitrification mixed liquor will be pumped from the nitrification unit to the nitrification final tank, existing Final Tank No. 7.

The nitrification effluent from this clarifier (Final Tank No. 7) will be mixed under controlled conditions with the return sludge from the denitrification final tank (No. 8) and then pumped to the head end of pass number 2 of Aeration Tank No. 3 for denitrification and post aeration-stabilization.

The effluent from the denitrification unit will be discharged through the existing aeration tank effluent channel to the denitrification final tank. This flow will be isolated from the remainder of the plant by closing an existing aeration tank effluent channel butterfly gate. The post aeration-stabilization will be accomplished through the use of the last 1/3 of the denitrification tank and channel swing diffusers.

The overall tank volume required for the post aeration stabilization process is 690,000 gallons (or 92,000 cubic feet). Pass 2 of aeration tank No. 3 provides 800,000 gallons (or 106,900 cubic feet). By operating the denitrification tank at side water depth of 12.93 feet, the detention time afforded by the first 2/3 of the tank will provide the 2.0 hours required for denitrification. Since post aeration - stabilization in the aeration tank effluent channel will be minimal, the process will begin in the last 1/3 of the denitrification tank through the use of the last three swing air diffusers in this tank.

Sheet F-4 of the Contract Drawings illustrates the return activated sludge system for the three stage secondary alternate.

4. Filtration and Carbon Adsorption

The filtration and carbon adsorption system design is based on the use of multiple individual units, the size of which would be incorporated directly in a future full-scale Nassau County recharge system, (Refer to Sheets G-9R and G-3R of the Contract Drawings). This approach was adopted because of the substantial degree of cost effectiveness it provides. However multiple units were provided so that downtime for maintenance will not have a large impact on available treatment capacity.

The clarified effluent from the secondary system will flow by gravity to the filter wet well where it will be pumped under controlled conditions to automatically backwashed, mixed media filters. Two gravity type filters will be provided with two additional filters on standby. Each filter will be 13 feet 4 inches wide and 38 feet 8 inches long, providing a surface area of 515 square feet. The average loading rate is 3.75 gallons per minute per square foot. Surface washing equipment will be provided. Backwash flow rates of 15 to 20 gpm per square foot have been provided. An average filter run of about 36 hours is anticipated, based on the type and degree of treatment to be applied in upstream processes and on the degree of operational control which is planned. However, the design recommendations make allowance for periods of operation that may be expected where filter runs will be reduced to under 24 hours, by providing additional reserve storage in the final clearwell. After passing through the filter media, the backwash water will be contained in a tank located in

the Tertiary Treatment building and will be pumped under controlled conditions back to the influent well. The storage tank will be 39.5 feet long by 36.9 feet wide. The tank volume will be 218,000 gallons (29,200 cubic feet), based on an approximate side water depth of 20 feet.

The carbon adsorption units will consist of two units in parallel. Two stand-by units will be provided. Each unit is 13 feet 4 inches wide and 38 feet 4 inches long, providing 511 square feet of surface area. The adsorption units will be operated as downflow packed bed units. At an average rate of 3.74 gpm per square foot, a contact time of 16 minutes is provided by two units in parallel. A possibility of utilizing three beds in parallel to extend the contact time to 24 minutes at the 2.49 gpm per square foot treatment rate has also been provided. See Table 3 for possible carbon adsorption unit arrangements and the corresponding detention times.

Based on an anticipated carbon exhaustion rate of 350 pounds per million gallons of wastewater processed,each unit would exhaust every 212 days. The regeneration furance proposed is such that one unit charge (110,700 pounds of carbon) can be regenerated every 2.375 days of continuous furnance operation. However, the operation will be staggered such that the carbon units are removed from service for regeneration sequentially. The furance will then operate continuously for (4 units x 2.375 day/unit) 9.5 days. The remaining days will be reserved as idle time for the furance before the next regeneration cycle. This provides an "oversize factor" in excess of 2 for the future ultimate facility.

TABLE II - 3

Carbon Adsorption Unit Arrangements

	Area (ft^2)	Rate (gpm/ft^2)	Contact Time Minutes	Depth (Ft.)
2 Beds	1022	3.74	16	8.0
3 Beds	1533	2.49	24	8.0
4 Beds	2044	1.87	32	8.0

Additional reserve will be provided by an inventory of 2 unit charges of carbon, one in each of the stand-by units. The makeup supply of virgin carbon will consist of 40,000 lbs. stored in a storage tank at the plant site.

Carbon regeneration rates vary with capacity of organic adsorption achieved. The 450 square foot hearth area proposed permits a carbon loading which is within the 100 pounds per 24 hours per square foot normally recommended during the 9.5 day operating period. The above is a general guideline, and has been compared with more specific manufacturer's recommendations. At the anticipated capacity of the carbon (0.5 pounds COD/pound carbon) a regeneration rate of 4 pounds of carbon per hour per square foot of hearth is indicated, or 43,200 pounds per 24 hours.

Lower capacities (pounds COD/pound carbon) tend to increase the regeneration rate (pounds/hour). Greater degrees of exhaustion of carbon will reduce regeneration rate, e.g., to about 2.5 pounds per hour where an exhaustion level of 1 pound COD per pound of carbon is achieved. Since this application is on polishing service, and relatively low and reasonable uniform feed COD concentrations can be expected, lower regeneration rates resulting from greater carbon exhaustion would be compensated for by an increase in the interval between required regenerations.

5. Disinfection and Storage

Although additional data are certainly warranted, present indications are that chlorination of secondary effluents can, with proper control of conditions, result in effective inactivation of virus. In the projected treatment system, certain additional treatment steps are proposed (coagulation, filtration,

lime treatment) which can be expected to increase virus reductions over those generally expected from secondary treatment alone. Most important, however, is that the quality of treated wastewater, which will be chlorinated, will be such to permit optimized effectiveness of chlorine. The wastewater will contain very low concentrations of suspended solids and compounds which react with chlorine (organics and ammonia). In addition, rapid mixing facilities will be available to optimize dispersal of chlorine. A 30 minute chlorine contact chamber is proposed, but this will be contained in a storage clear-well providing up to 9 hours further retention. Aeration tank number two, passes three and four, will be used for the chlorine contact chamber and effluent storage clearwell. The chlorine contact chamber will be covered to eliminate airborn contamination.

With a negligible chlorine demand expected in treated water, adequate residuals and long contact will provide an optimum environment for maximizing viricidal effects, at quite nominal chlorine dosages.

Chlorine would appear to be able to provide effective control of viruses and to maintain residuals desired in the distribution system. Chlorination is, therefore, the primary disinfection technique proposed. Dosage rates of from 1 to 8 mg/l, based on the average design rate of flow, are provided.

A final clearwell is provided following the reclamation facilities. This will serve as a pump suction chamber for service pumps to deliver treated water to the recharge site; it will provide additional chlorine contact time at the treatment site; and it will provide the necessary equalization to permit continuous delivery of 4 mgd to the injection wells, despite fluctuations in plant output due to backwashing of filters and carbon adsorbers.

A total volume for active control equal to 1.6 MG has been supplied to assure delivery of 4.0 mgd at all times.

Sheet 3 of the Contract Drawings shows the site plan of the Cedar Creek Plant and the location of the proposed structures.

D. Monitoring and Control

The concept of groundwater recharge of recovered municipal wastewaters requires that a high quality effluent be discharged from the reclamation plant at all times. Monitoring will be required at each of the unit processes in the reclamation plant to accomplish this goal.

In large scale systems, such as proposed (5.5 mgd), it is not feasible to store product water from the waste treatment facilities until all detailed chemical, bacteriological and virological tests have been completed. Therefore, it is necessary to utilize monitoring procedures which will facilitate real time operation and control of the reclamation facilities. Particular emphasis will be placed on monitoring requirements associated with health and safety and those constituents that adversely affect the recharge operations. The chlorine residual measurements will be supplemented by actual bacteriological and virological measurements. Such measurements, due to being time consuming, are not available to be effective for real time control of the treatment systems.

On-line monitoring can significantly improve the reliability in the quality of treated water. Monitoring equipment

has been reviewed and incorporated in the treatment system in order to (a) aid in controlling the unit operations so that remedial actions can be taken to improve the performance of the process, (b) to provide a real time indicator that will define the acceptabiliby of the treated water for recharge, and (c) to provide detailed data collection to assist in the process evaluation of the unit treatment operations. Table II-4 has been included to illustrate the anticipated water quality after each stage of the treatment process scheme.

TABLE II-4

ANTICIPATED WATER QUALITY THROUGH TREATMENT STAGES

Constituent	Raw Wastewater Concentration mg/l	Effluent Concentration From Indicated Process: Primary Clarifier (lime ppm.)	Carbon Oxidation + Nitrification	Denitrification + Post Aeration (Alum add.)	Filtration	Adsorption	Disinfection
BOD_5	200	80-100	5-15	4-8	1-5	0-2	
COD (Dichrom)	350	150-250	60-85	40-60	30-50	10-25	
TOC	130		10-30	10-20	10-15	5	
NH_3-N	22	18	0-2	0-1	0-1	0-1	
NO_2+NO_3-N			20	0-1	0-1	0-1	
Organic N	13	7	1-2	1-2	1-2	1-2	
Total N	35	25	21-24	2-3	2-3	2-3	
Total PO_4 as P	16	2-4	1-3	0.1-0.3	0.1-0.2	0.1-0.2	
Ortho PO_4 as P	10	1-2	1-3	0.1-0.3	0.1-0.2	0.1-0.2	
Turbidity	250 JTU			10-20 JTU	0.1-2 JTU	0.1-1 JTU	
S.S.	200	50-70		4-10	1-3	1-2	
	160						
TDS	500			350-450*			350-500*
Cl^-	60-75*			60-75*			70-80*
SO_4^-	70			70-90*			70-90
Phenols	.15						0.005
MBAS	10.0			0.3-1.5			0.03-0.3
CCE	8.0			1.6			0.05-0.5**
Coliform	5.0×10^7/100ml**						2/100 ml*

*Freeport TDS = 750 mg/l, Cl^-=300 mg/l.
All data reported as mg/l, except where indicated.

** Based on Tahoe.

TABLE II-4

ANTICIPATED WATER QUALITY THROUGH TREATMENT STAGES
(continued)

Constituent	Raw Wastewater Concentration mg/l	Effluent Concentration From Indicated Process: Primary Clarifier (lime ppm.)	Carbon Oxidation + Nitrification	Denitrification + Post Aeration (Alum add.)	Filtration	Carbon Adsorption	Disinfection
Al	.70			0.5-1	.02-.03	.01-.02	
Alkalinity ($CaCO_3$)	.19						
F	.5				.25-.5	.25-.5	
CR^{+6}			.01-.03		.91	.01	
Cr (hex)	0.9		.3-.5		.3-.5	.3-.5	
Cu	.16				.01	.01	
Fe+Mn	2.0		.2-.7		.07-.4	.07-.4	
As			.01		0	0	
Barium	0.2		.007				
Cadmium	0.01		.01				
Cyanide	0.04		.03-.07		.01	.01	
Boron	0.30		.8-1.2		.3-.5	.3-.5	
Pb	0.2		.06 -.08				
Hg	0.001		.0005				
Selenium	0.02		.005-.01				
Silver	0.01		.005		.2-.25	.2-.25	
Zinc	0.7		.2-.25				
pH	7.4						
Hardness ($CaCO_3$)	1.2						

Extensive instrumentation, monitoring and direct digital computer control capabilities were incorporated into the present Cedar Creek facilities. The Process Instrumentation Diagrams, Sheets I-1 through I-9 of the Contract Drawings for the Cedar Creek Water Reclamation Facilities, illustrate all monitoring and control intrumentation items included in the Reclamation Facilities.

Instrumentation to aid in controlling unit operations includes, as an example, on-line TOC monitoring of the activated carbon effluent. Increased organic carbon levels would indicate a loss of adsorption capacity and the need for regeneration. The instrumentation provided to evaluate final treated effluent acceptability for recharge includes as an example turbidity monitoring. High levels of turbidity would be reason for rejection of the water for recharge and the effluent would be by-passed to ocean disposal.

A significant amount of valuable data will be collected during the demonstration program. A framework will be developed prior to the actual implementation of the program for the collection, coding and storing of the data. Computer techniques will be included as an integral part of the program in order to make full use of the data collected. Application of computer methods tied into the recommended plant instrumentation will greatly facilitate the handling and storage of data and the analyses and control of the systems. Careful consideration should also be given to the method of evaluating each unit

operation, giving due consideration to the variables that affect each operation.

Additional software and memory core will be provided for the reclamation program as required.

III. TRANSMISSION MAIN

The proposed demonstration project involves the use of a transmission main system to convey reclaimed water to the recharge sites.

The routing which has been selected parallels the Wantagh State Parkway from the Cedar Creek Treatment Plant to the recharge site. This routing was selected because of the ease of pipe delivery, less interferences with private interests, and it results in a minimum of traffic disruption during construction due to the width of the existing right-of-way. The drawing entitled, *Location Plan*, included in the Contract Drawings for the Transmission Main illustrates this routing. The length of the transmission main required using this route is approximately 6.25 miles, or 33,000 linear feet.

The transmission main size was calculated based on head loss characteristics at an average rate of flow of 2,800 gallons per minute. These calculuations indicated that a 24 inch diameter force main would be adequate.

Based on the above investigations it is proposed that the transmission main be 24 inch diameter reinforced concrete cylinder pipe. The method of installation will be open trench type, with invert elevation just below the frost line for the Wantagh State Parkway.

IV. GROUNDWATER RECHARGE FACILITY*

A. Description of Meadowbrook Recharge Site

1. General Description

The Meadowbrook Recharge Site shown in Sheet 3 of the Contract Drawings is the roughly triangular-shaped piece of county owned land southeast of the intersection of Carman Avenue and Salisbury Park Drive in the Town of Hempstead. It includes approximately 35 acres of land which currently is occupied in part by the vegetable farm of the Nassau County Prison and in part by the structures and leaching basins of the Meadowbrook Sewage Treatment Plant.

Both the shallow-well and basin recharge studies will be conducted at the Meadowbrook site. The program proposed will recharge 4 mgd over a 3-year period; about 2.0 mgd are to be recharged through basins and about 2.0 mgd through wells. Five wells and 11 basins will be used. Their proposed locations are shown in Sheet G-1 of the Contract Drawings.

B. Basin Recharge System

1. General Discussion

The 11 basins proposed for recharge of reclaimed wastewater at the Meadowbrook site are shown in Sheet G-2 of the Contract Drawings. About 2.0 mgd will be recharged through these basins over a 3-year period. Seven of the basins will be constructed and four will be used as they presently exist.

The basins numbered from 1 through 7 in Sheet G-2 will be constructed as shallow recharge basins. The basins will

**All Contract Drawings referred to in this section are contained in the Recharge Site Drawings.*

be used for;(1) the determination of management practices that are most effective for optimizing recharge, (2) the evaluation of the clogging phenomena associated with the application of reclaimed wastewater, and (3) the evaluation of the potential of the unsaturated zone for improving water quality.

Basin 8 is an existing deep pit that, to date, has not been utilized for recharge purposes. Studies at the basin will be directed toward evaluating the effectiveness of deep pit recharge.

Basins 9 and 10 are shallow basins that are presently being used for the recharge of sewage effluent. Because the operation of the sewage treatment plant that these basins are servicing will be discontinued in the near future, the basins should be available for use in the recharge study. These two basins would be used primarily for the recharge of water in excess of that required for operating basins 1 through 8. As the study proceeds, data from basins 1 through 8 may indicate the desirability of running supplemental studies in these basins.

Nassau County Recharge Basin 62 is presently being used for the recharge of storm runoff. This basin would be used primarily for the emergency disposal of reclaimed water. Whenever sufficient water is available studies may be conducted in the recharge basin itself to evaluate the effectiveness of using storm-runoff basins for the supplemental recharge of reclaimed wastewater.

The quantity of water entering each recharge basin would be monitored by a tube meter and the flow measurements recorded on digital-punch tape at the Operations Building.

The quality of water would also be monitored and recorded at the Operations Building.

2. <u>Basin Recharge Program - 1 through 7</u>

When basins are used for recharging wastewater, the emphasis is usually placed on either engineering the system to optimize the amount of water recharged, or engineering the system to take advantage of the ability of the unsaturated zone for improving the quality of water. Because of the high cost as well as the limited availability of land, the emphasis in Nassau County is placed on engineering the system to optimize the amount of water recharged. Therefore, basins 1 through 7 would be designed and operated to establish the most effective procedures for maintaining high recharge rates. Studies concerning the effectiveness of the unsaturated zone for improving water quality will be restricted to an elevation of the changes in water quality that occur when water percolates through the unsaturated zone at high rates.

The general approach for the studies would be as follows:

(a) Different management practices would be used at each of the seven basins.

(b) Infiltration rates would be determined at short time intervals throughout the periods of water application for evaluating the effectiveness of the different management procedures for maintaining high recharge rates.

(c) The clogging phenomenon associated with the application of reclaimed wastewater would be evaluated at two of the basins for determining its effect on infiltration.

(d) Changes in the quality of the percolate as it moves through the unsaturated zone would be determined at two of the basins for evaluating the potential of the unsaturated zone for improving water quality.

3. Selection of Management Practices

A proposal of the management practices to be used at the seven basins is shown in Table IV-1. The practices represent several application-rest cycles as well as several methods for promoting dispersal of clogging materials that may accumulate or form near the land surface during recharge. A discussion of the reasons for selecting these practices follows.

TABLE IV-1

Management Practices For Basins 1 through 7

Basin Number	Application-Rest Cycle	Treatment
1	1:2	None
2	Continuous Operation	None
3	1:1	None
4	2:1	None
5	1:1	Basin cleaned and cultivated during rest period
6	1:1	Layer of porous media on basin floor (perhaps pea-size gravel or coarse sand)
7	1:1	Seeded to grass (perhaps Reed canary grass)

The selection of the most effective management practices for optimizing recharge would depend upon how different practices affect the infiltration rate of a basin. Infiltration rate is governed by the hydraulic conductivity of the materials below the basin.

A reduction of infiltration rate is associated with a reduction in hydraulic conductivity which, in turn, can generally be associated with clogging. Even when relatively clean water is recharged, such as the water to be recharged in this study, it can be anticipated that a reduction in rate will occur as water is applied. The most important causes for the reduction are likely to be clogging by biological agents and the activities of these agents. Such clogging is generally a surface phenomenon. It is most pronounced under anaerobic conditions that are conducive to the development of organic slimes and the formation of ferrous sulfide. When aerobic conditions are reestablished by allowing the materials in the unsaturated zone to drain periodically, the infiltration rate will generally increase, often to near its initial capacity.

The preceding discussion indicates that the selection of an appropriate application-rest cycle would be an important consideration for basin recharge. Another consideration will be the selection of management practices that would cause disruption and dispersal of the clogging materials. Practices applicable for this purpose are (1) establishing a plant cover that will provide root channels through the clogging zone, (2) tilling the basin floor so as to modify the physical characteristics of the clogging zone, and (3) covering the land surface with a highly permeable material such as graded sand and gravel to extend the clogging zone and thereby disperse the clogging agents.

4. Design and Operation of Basins

The proposed arrangement of Basins 1 through 7 is shown on Sheet G-2 of the Contract Drawings. The basins will be 5 feet deep and have a floor area of 50 ft. x 100 ft. The

construction of the basins is shown on Sheet G-5 of the Contract Drawings.

Water will be applied in a manner that will insure a minimum disturbance of the basin floor. The distribution system will be constructed so that it can readily be removed during periods when water is not being applied in order to treat the basin floor as required. The design for accomplishing this is shown on Sheet G-5 of the Contract Drawings.

The infiltration rate per unit area as related to temperatures and basin stage will be determined for short time intervals. To determine the infiltration rate for each basin the flow rate into and the water level in each basin shall be monitored. The following will be monitored at the Operations Building:

(a) Water Temperature

(b) Pan Evaporation

(c) Precipitation

All measurements will be continuously recorded on digital punched tape housed in the Operation Building.

The tentative proposal of the management practices to be used in the seven basins (see Table IV-1) is developed such that four basins will be in operation at any specified time. They will include basin 2, and one each of basins 1 or 4, 3 or 5, and 6 or 7.

The water requirement for operating four basins will depend on the infiltraton rates. An average infiltration rate of 100 gpd per sq. ft. was assumed for planning purposes. With four basins in operation, the total infiltrating area will be 20,000 sq. ft. Multiplying this area by the assumed infiltration rate gives a water requirement of 2 mgd.

The assumed infiltration rate is somewhat higher than the rates observed at the basins being used for the recharge of effluent from the Meadowbrook Treatment Plant, which are one the order of 25 to 50 gpd per sq. ft. Inasmuch as the water to be applied will be more highly treated than that from the Meadowbrook Plant, a higher rate is to be expected.

The average infiltration rate of the four operating basins probably will not often exceed 100 gpd per sq.ft. If the rate frequently exceed 100 gpd per sq. ft., the size of the basins can be reduced by constructing a partition within the basin.

5. Special Studies at Basins 1 and 4

Data for the evaluation of the clogging phenomenon and the potential of the unsaturated zone for improving water quality will be collected at basins 2 and 3.

The location of the clogging zones as well as the effect of clogging on the hydraulic conductivity of the soil materials will be determined from pressure head and moisture measurements.

The causes of clogging would be evaluated from physical, chemical, and microbiological analyses of core samples collected from the clogged zones. The type of analysis to be made as well as the frequency of sampling will be determined as the study progresses.

The monitoring of these special studies will be discussed in Section D, "Monitoring and Control".

6. Recharge Program at Basin 8

The deep pit designated as basin 8 will be used virtually as is (Refer to Sheet G-2 of the Contract Drawings). Recharge studies will be directed towards obtaining data for evaluating the effectiveness of deep pit recharge as compared to shallow basin recharge.

The selection of the application-rest cycle and land management practices will be based on test results at basins 1 through 7.

Because a major consideration for optimizing recharge at a deep pit will be the management of the water level in the basin, initial tests will concentrate on this aspect. Infiltration rate and the flow pattern below the spreading area will be determined. The need for other types of tests and additional measurements will be decided as the study proceeds.

Techniques for applying water, determining the infiltration rate, and characterizing the flow pattern will be as described in the discussion of the proposed recharge program for basins 1 through 7.

C. Shallow-Well Recharge Program

Five wells are proposed for the Meadowbrook Recharge Site to recharge the reclaimed water. Four of the wells will be in operation at any specified time with one on standby. Each well will recharge 0.5 mgd thus the water requirement for the recharge wells will be 2.0 mgd. Preliminary testing of wells and the aquifer system will be done during the first year of the study. Then,

the recharge wells will be operated as continuously as feasible for a 3-year period. The wells will be redeveloped as needed.

1. Recharge Well Design

The basic well design is shown on Sheet G-7 of the Contract Drawings (Well Type 1). The recharge wells will be drilled by the reverse-rotary method, a method successfully used on Long Island for the installation of large diameter wells. Three of the five wells will be constructed according to this basic well design.

Two variations of the basic well design would be included for study of the most effective features. One well will be completed without an artificial gravel pack (Well Type 2). This will permit comparision of the operating effectiveness of natural-pack wells with artificial-pack wells. The generally coarse-grained texture of the upper glacial aquifer lends itself to natural development of wells and such wells cost less to construct than those with artificial gravel packs. Morever, a well which is not gravel packed may cause injected particulate matter to be deposited closer to the screen, and this may facilitate redevelopment of the well. On the other hand, a gravel pack provides for a larger aquifer-face surface area on which to distribute the clogging material and, thus, the period of recharge between redevelopment cycles should be longer for a gravel-packed well.

A second well will incorporate features proposed by the Institute of Drilling Research, Inc. The Institute's proposal includes a built-in system for redevelopment of the well by air lift pumping and surging. The system requires installation of an

eductor pipe and an air line, in addition to other components in the basic design, and, in order to provide proper submergence of the air line, the well must be deepened to about 135 feet by extending the sand trap (sump) beyond that shown in the Basic Design. (See Sheet G-11 of the Contract Drawings, Well Type 3).

Test drilling has been done at each proposed recharge-well site to validate design of the recharge wells. Analyses of lithologic samples collected during the test drilling permits tailoring of the recharge-well specifications to each site. It was not necessary to raise or lower the screened interval of a well from that shown by the preliminary design because of local well-site lithology.

In developing the preliminary well design the following factors were considered:

(a) The water table at the site is about 35 feet below land surface.

(b) Saturated thickness of the upper glacial aquifer in southern Nassau County averages 50 feet; therefore the demonstration well, to be representative, should be screened in an interval above 100 feet depth.

(c) The well screen should be as long as possible so as to have maximum surface area and thereby reduce the effect of clogging materials. But its top should be far enough below static level to prevent dewatering the screen during redevelopment of the well. Placing a 30-foot-long screen in the depth interval 65 to 95 feet allows for 30 feet of drawdown during redevelopment. At this drawdown the well when first installed should produce at least 1000 gpm. Based on a gravel-pack thickness of 0.5 feet, a 30 foot screen length will provide a loading at the aquifer face of 2 gpm per square foot at a recharge rate of 350 gpm.

Well "C" will have observation wells constructed adjacent to it as shown in Sheet G-9 of the Contract Drawings. These observations wells would help define the causes of well clogging and the distance of penetration of the clogging agent into the aquifer.

2. Preliminary Testing

After completion of each recharge well, a step-drawdown pumping test will be made to determine the hydraulic characteristics of the well. A carefully controlled pumping test to determine the hydraulic characteristics of the aquifer system will be made after all recharge wells and observation wells are installed.

3. Recharge Operation

In order to insure continuous operation of the well system at a constant rate, four wells will be operated at a time. The fifth well will serve as a standby and will be put into service when one of the other wells requires redevelopment. Excess water will be routed to the basin recharge system or nearby Nassau County Storm Runoff Basin No. 62 for disposal.

Each well will be operated at a constant rate of 350 gpm or 0.5 mgd. Injection will continue until the well-head pressure reaches 25 feet of water, or approximately 10 psi. A well-head pressure of 10 psi is far below the geostatic pressure opposite the top of the well screen, which is equivalent to a well-head pressure of about 23 psi. Proper grouting of the annulus around the well casing should prevent upward "piping" of the water as it leaves the well screen.

Water to be recharged will be received into a central reservoir from where it will be distributed to the recharge wells (and also to the recharge basins). Pumping facilities for proper line pressure will be provided. Flow rate, line pressure, and water quality parameters will be monitored and recorded. The water quality parameters will include total chlorine residual, turbidity, temperature, specific conductance, dissolved oxygen, and pH. A sample compositor will collect water samples from the distribution line for complete chemical analyses. Weekly composite samples are planned as a regular routine with samples for selected parameters collected intermittently as needed. The turbidity monitor will be connected to a controlling device which would bypass the water to waste or to the basin site whenever the turbidity exceeds 1 JTU. The distribution reservoir will also contain chlorination equipment which can be used to adjust the chlorine residual prior to distribution.

Each recharge well will be equipped with a flow regulator which will maintain a constant rate of 350 gpm against a varying well-head pressure. Each well will also have a pressure sensor. The monitored flow rate and injection head will be telemetered to the Operations Building where the data will be recorded.

On reaching a well-head pressure of 10 psi a recharge well will be taken off the recharge system. It would then be redeveloped. Except for the one well constructed with built-in air pumping equipment, redevelopment will necessitate removal of the injection pipe from the well. Redevelopment will consist of sur-

ging and air-lift pumping 5-foot sections of the screen isolated between inflatable packers. After one section of the screen is redeveloped,the assembly will be moved and the process repeated. By isolating and working short intervals of the screen, maximum agitation will be obtained per unit of screened area, but with little water production. This system of redevelopment was found to work best in the Los Angeles County Project.

Installing air pumping facilities as an integral part of the well, as in Well "E" to be constructed according to the design of the Institute of Drilling Research, will reduce labor costs for set-up time for each redevelopment event. However, it will not be possible to isolate short intervals of the screen and, hence the agitation will be distributed over the full screen length. Redevelopment may be less effective, and a greater frequency of redevelopment may be needed, thus negating the savings in set-up costs. Comparision of the costs and effectiveness of the two methods will be possible after the 3-year period of operation.

The effectiveness of the redevelopment by air agitation will be evaluated by test pumping the well for several hours. If considerable loss in well capacity remains, the well should be treated chemically, as with hydrochloric acid. The redeveloped well will then be returned to service as needed.

D. Monitoring and Control

1. General

The Meadowbrook Recharge Site will be surrounded by an observation well network as shown in Sheet G-13 of the Contract Drawings. This network calls for observation wells to be installed near the center of both the injection well and basin recharge sites.

In addition to the observation well network, a reservoir sampling pump coupled with a composite sampler will be included. The composite sampler will analyze the reservoir water for dissolved oxygen, turbidity, temperature, pH, chlorine residual, and specific conductance. Water from recharge basins one (1) and four (4) will be analyzed for pH, dissolved oxygen, and sulfide as shown on Sheet I-2 of the Contract Drawings. This drawing illustrates all process instrumentation associated with the recharge site.

2. Observation Well Network

The proposed observation-well network for collecting data to evaluate aquifer response is shown on Sheet G-13 of the Contract Drawings. This arrangement calls for observation wells to be installed near the center of both the injection well and basin recharge sites and at selected distances up-gradient, cross-gradient, and down-gradient from these points. The network places an emphasis on measuring points down-gradient (south) from the direction of flow.

The network will consist of either a single well or nest of wells at 22 locations. Wells screened at the water

table (a depth of about 40 feet) will be installed at all locations. Wells screened in the basal part of the upper glacial aquifer (a depth of about 100 feet) will be installed at 17 locations, and wells screened in the upper part of the Magothy (a depth of about 200 feet) will be installed at 9 locations.

The observation wells will consist of 6-inch Fiberglass casing and a 5-foot section of 6" 316 S.S. screen. The annular spacing above the screened interval will be sealed with cement.

Core samples will be collected during the drilling of the observation wells. Geophysical logs will be run after completion of the drilling. Water-level and water-quality measurements will be collected prior to and during the period of recharge.

The description of the lithology and hydrologic properties of the aquifer system will be needed for mathematical modeling. Core samples and geophysical logs will be used for developing a detailed lithologic description of the aquifer system. Evaluation of the hydrologic properties of the system will be based on laboratory analyses of core samples. The analyses will include grain-size distribution, porosity, specific yield, hydraulic conductivity, and mineral composition of samples considered representative of the principle formations in the system.

Continuous water level records will be obtained at all of the observation wells. Float-activated digital water

level recorders will be used for obtaining the data.

Water samples will be collected periodically by temporarily installing a submersible pump suspended from a reel-mounted discharge line. The chemical constituents to be measured as well as the frequency of measurements will depend on how water quality is observed to change with time.

One of the main purposes of the observation well network of the Meadowbrook Recharge Site is for the evaluation of the aquifer response to recharge. The response will be expressed in terms of pressure head and water quality changes in the system. Since the measurements at the observation wells will provide an assessment of changes in only part of the system, mathematical models will be used for projecting changes over a larger area of concern. Data collected from the wells will be used for selecting an appropriate model, evaluating parameters required in the model, and assessing the reliability of model predictions.

3. Recharge Basins and Wells

Manholes will be installed in the unsaturated zone at the two basins to facilitate the installation of pressure-head and water-quality monitoring equipment and the collection of soil and water samples. They will be 8 feet in diameter and will extend to a depth of 15 feet below the basin floor (Refer to Sheet G-5 of the Contract Drawings). The base of the manholes will be 2.5 feet above the elevation of the projected ground-water mound.

Soil samples for laboratory analysis will be collected through ports in the walls of the manholes.

Water samples from the unsaturated zone monitoring samplers for each level, from which automated continuous records of pH, dissolved oxygen content, and sulfide content will be collected from sampling wells installed through the walls of the manhole casings. Additional samples will be collected for more detailed laboratory analysis. The sampling wells will be driven such that the point of inflow will be at least 1 foot above the point of outflow. Such an installation will allow for the collection of unfiltered water samples.

Pressure-head monitoring devices will be installed in the walls of the manhole casings. The devices will include piezometers, tensiometers, and pressure transducers. The piezometer and tensiometer records will be obtained by manual readout. The pressure-transducer records will be on digital-punch tape.

The relationship of clogging to the aeration of the percolating water will also be evaluated. Measurements of pressure head, infiltration rate, and parameters that reflect the aeration of the percolating water, such as dissolved oxygen, sulfide content, and pH, will be used for the evaluation. Measurements will be made continuously on digital punch tape so that analyses can be made by computer techniques.

The evaluation of the potential of the unsaturated zone for improving water quality will be based on how the quality of water changes at it percolates through the unsaturated zone. Laboratory analysis will be made of samples collected from ponded water within the basins, from several depths in the unsaturated zone, and from the upper part of the saturated zone. The parameters to be measured, as well as the frequency of sampling, will be determined as the study progresses.

Water wells will be installed adjacent to the recharge basins for measuring water levels in and collecting water samples from the saturated zone. The wells will be 6 inches in diameter. The maximum elevation of the ground-water mound is expected to be about 20 feet below the land surface therefore the wells will be screened from this elevation to a depth of 40 feet below land surface of 5 feet below the present water table. Water level records will be obtained using float-activated digital recorders. Water samples for laboratory analysis will be collected using a submersible pump.

Moisture measurements will be obtained through aluminum 2" diameter access tubes located within and adjacent to the basins. The measurements will be determined with a neutron-moisture meter which has a continuous logging capability.

Tables IV-2 and IV-3 are listings of the monitoring programs for the recharge basins and the shallow-well recharge program, respectively. These tables list the various parameters which will be measured and the methods of measuring and recording the measurements.

TABLE IV-2

MONITORING PROGRAM FOR RECHARGE BASINS

Item	Location	Remarks
Dissolved Oxygen	Reservoir	Composite Sampler
Turbidity	Reservoir	Composite Sampler
Temperature	Reservoir	Composite Sampler
pH	Reservoir	Composite Sampler
Chlorine Residual	Reservoir	Composite Sampler
Specific Conductance	Reservoir	Composite Sampler
Flow Rate	Each Basin	Continuous - Venturi Meter
Basin Level	Each Basin	Continuous Bubbler
pH	Basin 1 and 4	
Dissolved Oxygen	Basin 1 and 4	
Sulfide	Basin 1 and 4	
Pan Evaporation	Control Building	
Precipitation	Control Building	
Infiltration Rate		Continuously Calculated
Ground Water Level	Water Wells @ Basins 1 and 4	Float activated digital recorders
Moisture Measurements	Neutron Access Tubes @ Basins 1 and 4	Continuous logging capability

Item	Location	Remarks
Ground Water Quality		
pH	Manholes @ Basins 1 and 4	Composite Sampler from Sample Wells in Manholes
DO	Manholes @ Basins 1 and 4	Composite Sampler from Sample Wells in Manholes
Sulfide	Manholes @ Basins 1 and 4	Composite Sampler from Sample Wells in Manholes
Core Sample of Soil	Manholes @ Basins 1 and 4	Through ports in wall of Manhole
Pressure Head	Manholes @ Basins 1 and 4	From Sample Wells in Manhole

TABLE IV-3

MONITORING PROGRAM FOR SHALLOW-WELL RECHARGE PROGRAM

Item	Remarks
Flow Rate	Venturi Meter @ Each Well
Line Pressure	@ Each Well
*Chlorine Residual	Reservoir Distribution Line, Continuous Sampling
*Turbidity	Reservoir Distribution Line, Continuous Sampling
*Temperature	Reservoir Distribution Line, Continuous Sampling
*Specific Conductance	Reservoir Distribution Line, Continuous Sampling
*Dissolved Oxygen	Reservoir Distribution Line, Continuous Sampling
*pH	Reservoir Distribution Line, Continuous Sampling

*Same Composite Sampler As Noted In Table IV-2 For Reservoir.

V. PERSONNEL REQUIREMENTS

The personnel requirements for the Cedar Creek Water Reclamation-Recharge Facility have been developed based on the highly technical nature of the program.

Table V-1 shows the personnel requirements for each shift at the recharge and reclamation facilities. Several members of the staff will work at both sites, however in Table V-1 they are only listed once. The personnel which are not designated by either a (*) or (#) could be provided from within the Department of Public Works.

TABLE V-1

PERSONNEL REQUIREMENTS

Reclamation Facility

1st Shift	2nd Shift	3rd Shift
*Project Manager		
*Asst. Project Manager		
*Chief Operator		
Operator - 2	Operator - 2	Operator - 2
*Lab. Superintendent		
Asst. Lab. Superintendent		
Chemist		
*Lab. Specialist		
Lab. Technician	Lab. Technician	Lab. Technician
Clerk-Typist		
Maintenance Man - 2	Maintenance Man	Maintenance Man
Instrument Man	Instrument Man	Instrument Man
Sub-Totals...........14	 5	 5

Recharge Facility

1st Shift	2nd Shift	3rd Shift
Operator - 2	Operator	Operator
#Junior Hydrologist		
#Junior Hydrogeologist		
#Senior Geochemist		
#Senior Soil Physicist - ½		
#Senior Hydrogeologist - ½		
Maintenance Man	Maintenance Man	Maintenance Man
Sub-Totals........... 7	 2	 2

*Technical specialists or administrative personnel.

#Personnel from possible U.S.G.S. - Nassau County Cooperative Study Program.

Based on the staff requirements listed in Table V-1, the twenty-four personnel at the Reclamation Facility will be based in the proposed Tertiary Treatment Building, except the Chemist and Laboratory Specialist who will be based in an available room in the existing Cedar Creek Laboratory. All of the eleven personnel associated with the Recharge Facility will be based at the recharge site.

APPENDIX A

DESIGN CRITERIA OF RECLAMATION FACILITIES

APPENDIX A

DESIGN CRITERIA OF RECLAMATION FACILITIES

A. Design Basis

1. Design Flow = 5.5 MGD
   - (a) Effluent for recharge = 4.0 MGD
   - (b) Waste sludge = 0.05 - 0.10 MGD
   - (c) Filter backwash = 0.6 MGD
   - (d) Carbon backwash = 0.6 MGD
   - (e) Excess = 0.20 - 0.25

2. Waste Concentration

Parameter	Initial Portion of Study (typical Freeport)	Final Portion of Study (typical Bay Park)
	(Concentrations are reported as mg/l)	
BOD_5	140	220
Suspended Solids	120	240
TKN	30	35
NH_3-N	19	22
Organic N	11	13
Total Phosphorus	10.2	16
Ortho Phosphorus	6.5	10
Alkalinity (as $CaCO_3$)	150	185
Hardness (as $CaCO_3$)	130	65
Calcium	27	16
Magnesium	16	6

B. Rapid Mix Tank

Detention Time = 1 minute
Volume @ Average Flow = 4,190 gallons
@ 1.5 * Average Flow = 5,690 gallons
Lime Dose = 200-350 mg/l
pH = 9.5 - 11 units
Provision For $FeCl_3$ Dosage Will Be Made

C. Flocculation Tank - 2

Horizontal Shaft Slow Speed Mixers - 2
Diameter = 4 feet
Detention Time = 6.25 minutes per chamber
12.5 minutes with stop gate open
Provisions included for dosage of Anionic Polymer to Flocculation Tank when $FeCl_3$ is used.

(c) Sludge production (Equation 23)

$a = 0.73$ from Eckenfelder [105]

$b = 0.05/\text{day}$ at F/M 0.2

a_n = nitrification = 0,17 from Haug and McCarty [34]

$\Delta X_v = 0.73\ S_r - 0.05\ X_v + 0.17 \quad NO_3$

BOD removal = 95%

$\Delta NO_3 = 20$ mg/l

Assume N synthesis = 5 mg/l (BOD:N:P = 100:5:1)

$\Delta X_v = 0.73 \times 100 \times 0.95 \times 5.5 \times 8.34 - 0.075 \times 2700 \times 8.34 \times 1.36 + 0.17 \times 20 \times 5.5 \times 8.34 = 3,180 - 1,530 + 156$
$= 1,806$ lbs/day

$t_s = X_v/\Delta X_v = 30,600/1,806 = 17$ days

(d) Oxygen requirement (Equation 24)

$a' = 0.65$ from Hydroscience [58]

$b' = 1.42\ (0.05) = 0.075/\text{day}$ (Equation 13)

95% frequency load = 1.5 x average load (Freeport, L.I. analysis) = 150 mg/l

BOD removal = 95%

Assume N synthesis = 5 mg/l

$\Delta NO_3 = 25 \times 1.5 - 5 = 32.5$ mg/l

(lb O_2 required/day) $= 0.65\ S_r + 0.075\ X_v + 4.57 \Delta NO_3$
$= 4,250 + 2,300 + 6,800 = 13,350$ lbs/day, say, 14,000 lbs/day

(e) Air flow requirement

$$E = E_o \left(\frac{BC_{sm} - C_L}{C_{sm_{20}}}\right) a\ 1.02^{T-20} = o_2 \text{ transfer efficiency under design conditions}$$

Assume: $a = 0.9$, $B = 0.9$, $C_L = 2.0$ mg/l, $E_o = 8\%$ (transfer efficiency under standard conditions) Ref. 167

$$C_{sm} = C_s \left(\frac{P_B}{29.4} + \frac{O_t}{42}\right) = \text{mid tank depth } O_2 \text{ saturation}$$

at H = 13 feet, $P_B = 14.7 + 0.431(13) = 20.3$ psia

$$\text{at } E = 5\%,\ O_t = \frac{0.290\ (0.95)}{0.791 + 0.209(0.95)} \times 100 = 20.1\% \text{ oxygen in exit gas}$$

at 24°C, $C_s = 8.5$ mg/l

$$C_{sm} = 8.5 \left(\frac{20.3}{29.4} + \frac{20.1}{42}\right) = 9.9 \text{ mg/l}$$

Similarly, $C_{sm20} = 10.7$ mg/l

$$E = 8\left(\frac{0.9(9.9) - 2}{10.7}\right) 0.9 \times 1.02^4 = 5\%$$

$$\frac{E}{100} = \frac{\text{oxygen required (lb/day)}}{G_s(\text{scfm})\,0.075\ \frac{\text{lb air}}{\text{scf}} \times 0.232\ \frac{\text{lb oxygen}}{\text{lb air}} \times 1,440\ \frac{\text{minutes}}{\text{day}}}$$

$G_s = 4 \times$ oxygen required/E

at $E = 5\%$, $G_s = 0.8 \times$ oxygen required

$G_s = 0.8 \times 14,000 = 11,200$ scfm

2. Intermediate Clarifier

(a) Size: Use 1 existing

$OR = 700$ gpd/sq. ft.

$Cu = 1.0\%$

$Co = 2,700/0.85 = 3,180$ mg/l

$$R/Q = \frac{3,180}{10,000 - 3,180} = 0.47$$

$$\text{Mass loading} = \frac{1.47(5.5)8.34(3,180)}{7,850} = 27.5 \text{ lbs/sq. ft. /day}$$

Note: Sludge will be wasted as necessary to denitrification unit to maintain a reasonable volatile solids content in denitrification sludge

(b) Effluent quality

BOD_5 = 5-10 mg/l

Suspended solids = 10-20 mg/l

TKN = 2

NH_3-N = 1

NO_3-N = 20

Organic N = 1

3. Denitrification and Post Aeration - Same Last Stage for Both Two Stage and Three Stage Systems

(a) Denitrification size

Use $t_d = 2.0$ hours, MLVSS = 2,700 mg/l (winter)

$$\text{Volume} = \frac{2.0}{24} \times 5.5 = 0.46 \text{ MG}$$

$$\text{Denitrification rate} = \frac{20(5.5)8.34}{0.46(2,700)8.34} = 0.089 \frac{\text{lb } NO_3}{\text{day-lbs MLVSS}}$$

Minimum temperature = $12^{o}C$

(b) CH_3OH required: Contra Costa = 3.54 NO_3^--N (Equation 34)

$$\frac{\text{lbs}}{\text{day}} CH_3OH = 20 \times 5.5 \times 8.34 \times 3.54 = 3,240 \text{ lbs/day}$$

Manassas = 3.04 NO_3-N + 1.07 DO (Equation 33)

Assume DO = 2 mg/l from nitrification tank and R/Q = 0.47 (see Intermediate Clarifier, D. 2)

$$\frac{\text{lbs}}{\text{day}} CH_3OH = 20 \times 5.5 \times 8.34 \times 3.04 + 2 \times 5.5 \times 8.34 \times 1.07$$
$$= 2,790 + 92 = 2,882 \text{ lbs/day}$$

Check F/M ratio: Assume methanol concentration = BOD_5 concentration:

F/M = 3,240/0.46(2,700)8.34 = 0.314 lbs BOD_5/day-lbs MLVSS

(c) Sludge production (see Equation 10)

$a_n = 0.2$ lbs VSS/lb CH_3OH

$\Delta X_v = 0.2\ (3,240) = 648$ lbs/day

(d) Post Aeration

(1) Size: Use t_d = 1.0 hour, Volume = 0.23 MG

(2) Oxygen requirement:

Assume a' = 1.0 based on methanol, b' = 0.10

Post aeration capacity oxidation of

50% methanol overdose = 1,620 lbs/day

Lbs oxygen/day = 1.0 (1,620) + 0.10 (2,700) (0.23) 8.34
= 1,620 + 520 = 2,140 lbs/day

G_s = 0.8 (2,140) = 1,710 scfm

(3) Chemical addition:

Influent total phosphorus to denitrification = 1.5-3.0 mg/l

Al/P = 2/1 at influent P = 3.0 mg/l Effluent P
= 0.3 mg/l, Al^{+3} = 6 mg/l

Al/P = 4/1 at influent P = 1.5 mg/l Effluent P
= 0.1 mg/l, Al^{+3} = 6 mg/l

Alum ($Al_2(SO_4)_3 \times 14\ H_2O$)/Al = 11/1

Alum dose = 11 (6) = 66 mg/l (based on raw flow)

Lb/day alum = 66 x 5.5 x 8.34 = 3,030 lbs/day

Anionic polymer dose (if required) $\approx$ 0.3-0.5 mg/l
(based on raw flow); $\approx$ 14 - 23 lbs/day

H_2SO_4 (if required)

(4) Sludge production (based on 3 mg/l influent phosphorus)

Chemical sludge (see Equations 44 and 45)

$\frac{\text{lbs}}{\text{day}}$ $AlPO_4$ = 3:94 (3-0.3) x 5.5 x 8.34 = 487 lbs/day

Al^{+++} remaining = 6-0.87(2.7) = 3.65 mg/l

$\frac{\text{lbs}}{\text{day}}$ $Al(OH)_3$ = 2.89(3.65) 5.5 x 8.34 = 483 lbs/day

Biological sludge

Denitrification sludge = a_n = 648 lbs/day

Assume 50% methanol overdose to post aeration:
a = 0.3, b = 0.075

ΔX_v = 0.3 (1,620) - 0.075 (2,700) (0.23) 8.34 = 486 - 388
= 98 lbs/day

Total ΔX_v for denitrification and post aeration = 648 + 98
= 746 lbs/day

4. Final Clarifier

(All computations are based on 50% methanol overdose unless otherwise indicated.)

(a) Size: Use 1 existing

A_s = 7,850 sq.ft.

Overflow rate = $\frac{5.5 \times 10^6}{7,850}$ = 700 gpd/sq.ft.

(b) Waste sludge quantity

(1) Chemical ($AlPO_4 + Al(OH)_3$): weight = 487 + 483 = 970 lbs/day

(2) Biological sludge quantity (includes sludge from preceding biological units)

ΔX_v = 1,806 + 648 + 98 = 2,552 lbs/day

at 85% volatile

$$\Delta X = \frac{2,552}{0,85} = 3,002 \text{ lbs/day}$$

(3) Total sludge

ΔX = 970 + 3,002 = 3,972 lbs/day

$$\% \text{ volatile } \frac{2,552}{3,972} = 64\%$$

(4) Recycle: Cu = 1.25%

% volatile = 64%

Co = 2,700/0.64 = 4,200 mg/l

$$R/Q = \frac{4,200}{12,500 - 4,200} = 0.50$$

$$\text{Mass loading} = \frac{4,200\ (1.50)\ 5.5\ (8.34)}{7,850} = 36 \text{ lbs/day-sq. ft.}$$

Note: If greater phosphorus removal is required, construction of additional post precipitation facilities is necessary.

5. Effluent Quality from Biological System

BOD_5 = 4 to 8 mg/l
Suspended solids = 4 - 10 mg/l
Total phosphorus = 0.1 - 0.3 mg/l
NH_3-N = 0 - 1 mg/l
Organic N = 1 - 2 mg/l
NO_3-N = 0 - 1 mg/l
Total N = 2 - 3 mg/l

F. Biological Treatment - Three Stage System

1. Carbon Oxidation - First Stage of a Three Stage System

(a) Size: F/M = 0.6 lbs BOD_5/day - lbs MLVSS, winter conditions; T = 12°C

MLVSS = 2,000 mg/l

$$X_v = \frac{5.5 \times 100 \times 8.34}{0.6} = 7,600 \text{ lbs}$$

$$\text{Volume} = \frac{7,600}{2,000 \times 8.34} = 0.46 \text{ MG}$$

$$t_d = \frac{0.46}{5.5} \times 24 = 2.0 \text{ hours}$$

(b) Sludge production (Equation 9)

$$\frac{Se}{So} = \frac{1}{1 + k' X_v t}$$ (integrated form of Equation 8 for completely mixed reactor)

Where

$k' = 20$ (g - day $^{-1}$)
$\qquad\qquad 1$

$X_v = 2$ g/l

$t = 2/24 = 0.083$ days

$$\frac{Se}{So} = \frac{1}{1 + 20\ (2)\ 0.083} = 0.23 \quad \text{Use } 0.25$$

BOD removal = 75%

$$\therefore X_v = 0.73\ (0.75)\ 100 \times 5.5 \times 8.34 - 0.075\ (7,600)$$
$$= 2,510 - 570 = 1,940 \text{ lbs/day}$$

at 85% volatile

$$\Delta X = \frac{1,940}{0.85} = 2,280 \text{ lbs/day}$$

(c) Sludge age = $X_v / \Delta X_v$ = 7,600/1,940 = 3.9 days

(d) Oxygen requirements (Equation 11)

Based on 95% of maximum load = 150 mg/l BOD_5

lbs O_2 required/day = 0.65 x 0.75 x 150 x 5.5 x 8.34 + 0.10 (7,600) = 3,370 + 760 = 4,130 lbs/day

G_s = 0.8 (4,130) = 3,300 scfm

2. Intermediate Clarifier

(a) Size: Use 1 existing

OR = 700 gpd/sq. ft.

Cu = 1.0%

$$Co = \frac{2,000}{0.85} = 2,350 \text{ mg/l}$$

$$\frac{R}{Q} = \frac{2,350}{10,000 - 2,350} = 0.31$$

$$\text{Mass loading} = \frac{1.3\ (5.5)\ 8.34\ (2,350)}{7,850} = \frac{18 \text{ lb}}{\text{sq. ft.-day}}$$

$$\text{Waste sludge flow} = \frac{2,280}{8.34\ (10,000)} = 0.03 \text{ MGD}$$

Note: Sludge will be wasted as necessary to denitrification unit to maintain a reasonable volatile solids content in denitrification sludge and also, as necessary, to nitrification unit to maintain the desired solids level.

(b) Effluent quality

BOD_5 = 25 - 35 mg/l

TKN = 25 mg/l

NH_3-N = 18 mg/l

Organic N = 7 mg/l

Suspended solids = 20 - 30 mg/l

3. Nitrification - Second Stage of Three Stage System

(a) Size

Sludge age > 15 days

$t_d = 4.0$ hour

Volume $= \frac{4}{24} \times 5.5 = 0.92$ MG

MLVSS = 2,000 mg/l

BOD_5 influent = 25 - 35 mg/l (carbonaceous)

$\Delta NO_3 = 20$ mg/l

Check nitrification rate $= \frac{20\ (5.5)}{2,000\ (0.92)} = \frac{.06 \text{ mg N}}{\text{mg MLVSS-day}}$

$F/M = \frac{35\ (5.5)}{2,000\ (0.92)} = 0.10$ lbs BOD_5/day - lbs MLVSS

(b) Sludge production

$\Delta X_v = 0.73\ (35)\ 5.5 \times 8.34 - 0.05\ (2,000)\ 0.92\ (8.34)$
$+ 0.17\ (20)\ 5.5\ (8.34) = 1,172 - 770 + 156 = 558$ lbs/day

at 75% volatile

$\therefore X = \frac{558}{0.75} = 875$ lbs/day = approximately 20 mg/l in effluent

(c) Sludge age $t_s = \frac{X_v}{\Delta X_v} = \frac{2,000\ (0.92)\ 8.34}{558} = 27.5$ days

(d) Oxygen requirement

95% of maximum BOD_5 load = 60 mg/l, Effluent BOD = 5 mg/l

TKN = 25 x 1.5 - 5 = 32.5 mg/l

a' = 0.65; b' = 0.075/day

lbs O_2 required/day $= 0.65\ (55)\ 5.5 \times 8.34 + 4.57 (32.5) 5.5 \times 8.34$
$+ 0.075 (2,000) 0.92 (8.34)$
$= 1,640 + 6,800 + 1,150 = 9,590$ lbs/day

$G_s = 0.8\ (9,590) = 7,680$ scfm

4. Intermediate (nitrification) Clarifier

(a) Use 1 existing - Same as for carbon oxidation unit except waste flow negligible

(b) $\frac{R}{Q} = \frac{2667}{10,000 - 2667} = .36$

Mass Loading $= \frac{2667\ (1.36)\ (5.5)\ (8.34)}{7850}$
$= 21$ lbs/day - ft^2

(c) Effluent quality
BOD_5 = 5 - 15 mg/l
TKN = 1 - 2 mg/l
NO_3-N = 20 mg/l
Suspended Solids = 20 - 30 mg/l

5. Denitrification and Post Aeration
(See D. 3 for calculations. These facilities will be the same for both Two Stage and Three Stage Systems.)

6. Final Clarifier
(All computations are based on 50% methanol overdose unless otherwise indicated.)

(a) Size - Use 1 existing
$_s$ = 7,850 sq. ft.
$$\text{Overflow rate} = \frac{5.5 \times 10^6}{7,850} = 700 \text{ gpd/sq. ft.}$$

(b) Waste sludge quantity
(1) Chemical ($AlPO_4$ + $Al(OH)_3$) weight = 487 + 483 = 970 lbs/day
(2) Biological sludge quantity (includes sludge from preceding biological units)
ΔX_v = 1,940 + 648 + 98 = 2,686 lbs/day
at 85% volatile
$$\Delta X = \frac{2,686}{0.85} = 3,160 \text{ lbs/day}$$
(3) Total sludge
ΔX = 970 + 3,160 = 4,130 lbs/day
$$\% \text{ volatile } \frac{2,686}{4,130} = 65\%$$
(4) Recycle: Cu = 1.25
% volatile = 65%
Co = 2,700/0.65 = 4,150 mg/l
$$R/Q = \frac{4,150}{12,500 - 4,150} = 0.50$$
$$\text{Mass loading} = \frac{4,150\ (1.50)\ 5.5\ (8.34)}{7,850} = 36 \text{ lbs/day-sq. ft.}$$
(5) Effluent quality from biological system - same as Two Stage Effluent (see D. 5.)

G. Filtration

1. Number of units - 2 (plus 2 on standby = 4)
   Unit size 13'-4" wide by 38'-8" long
   (515 sq. ft. surface area)
   Gravity bed
   Tile underdrain design
   Provision for surface wash during backwash

2. Filter Media - Multi-media (sand, anthracite and garnet)
   Total bed depth approximately 30"

3. Filtration Rate - Average rate for all filters at 5.5 mgd flow will be 3.75 gpm/sq. ft. Filtration rates between 2 and gpm/sq. ft. may be employed.

4. Backwash - Backwash rates of 15 to 20 gpm/sq.ft. will be provided depending on water temperature in order to provide bed expansion. Provision for a surface wash during backwash cycle will be made. Backwash water returned to Cedar Creek Plant influent wet well.

5. Control Elements - Individual filter units to be equipped with rate of flow control devices and loss of head monitoring. Provision to be made for backwash rate control.

H. Activated Carbon

1. Number of Units - 2 (plus 2 stand-by = 4)
   Layout - Arranged as parallel units
   Unit Size - 13'-4" wide by 38'-8" long
   515 ft^2 of surface area
   Open vessel design
   Tile Underdrain design
   Provision for surface wash during backwash

2. Contact Bed - 8 x 30 mesh granular activated carbon
   Bed Depth - 8'-0"
   4120 ft^3/unit
   110,700 lbs/unit

3. Total Carbon Inventory

2 operating beds	221,400 lb.
2 standby beds	221,400 lb.
virgin storage carbon	40,000 lb.
Total	482,800 lb.

4. Treatment Rate

Unit loading rate = 3.74 gpm/ft^2 (2 units in parallel)
= 2.49 gpm/ft^2 (3 units in parallel)

Contact time = 16 minutes (2 units in parallel)
= 24 minutes (3 units in parallel)

Volumetric load = .31 gpm/cu. ft. based on 3 units in parallel
Individual units may be operated at rates between 2 and 7 gpm/ft^2

Backwash- Backwash rates of 15 to 20 gpm/sq. ft. will be provided depending on water temperature and carbon size in order to provide 50% bed expansion. Provision for surfacewash during backwash cycle will be made.

Control Elements - Individual units to be equipped with rate of flow control devices and loss of head monitoring. Provision will be made for backwash rate control.

I. Carbon Regeneration

Provide one multiple hearth furnace including standard accessories for carbon regeneration. Space is provided for a future unit of equal size.
Furnace minimum size - 11'-9" dia. x 5' hearth. Hearth Area = 450 ft^2

Provide a storage tank for regenerated carbon, capacity for 110,700 lbs. or 4120 ft^3.

J. Disinfection and Storage

Chlorination - 30 minute contact chamber at 5.5 mgd flow which is contained in a storage clear well providing 9 hrs. further retention.

Chlorine feed rate 350 lb/day to apply dosage up to 8 mg/l.

Provide final clearwell with effective control volume of 1.6 million gallons.