CDM

REGIONAL WASTEWATER TREATMENT PLANT DESIGN REPORT

CITY OF PLANTATION, FLORIDA
AND
GULFSTREAM UTILITIES COMPANY

SEPTEMBER 1983

Camp Dresser & McKee

06.215

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Camp Dresser & McKee Inc.
2001 Northwest 62nd Street
Fort Lauderdale, FL 33310

6009-42-RT

CDM

environmental engineers, scientists, planners, & management consultants

CAMP DRESSER & McKEE INC.

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September 30, 1983

Mayor Frank Veltri
City Council Members
City of Plantation
400 Northwest 73rd Avenue
Plantation, FL 33317

City of Plantation and
Gulfstream Utilities Company
Regional Wastewater Treatment Plant

Dear Mayor and City Council Members:

We are pleased to present this design report for the proposed regional wastewater treatment plant to serve the City of Plantation and the Gulfstream Utilities Company. Fifteen copies are submitted for distribution to interested parties.

The design report recommends the construction of a 10 million gallons per day (MGD) facility in 1986, to serve the City and Gulfstream, followed by a 5 MGD plant expansion in 1991. However, the possibility of other implementation options, which are listed below, should be investigated:

- o If the Gulfstream and South Plants were granted a variance to continue operation through a later year, a 5 MGD plant could be constructed initially. A 10 MGD plant expansion would be required when those plants join the regional system.
- o Similarly, if Gulfstream were granted a variance and the existing 1.5 MGD interim agreement with Broward County could be maintained, then a 5 MGD plant could be constructed initially.
- o If a 10 MGD facility were constructed initially, then the options described above could be employed to postpone the 5 MGD plant expansion.

These possible alternatives should be given further consideration before the preliminary design phase is begun.

This report was prepared under the general direction of Charles H. Bolton, and Donald G. Munksgaard served as the project manager. Project engineers were Julie Childers, Dan Anderson, and Dan Hutton. Gary Witt was the project hydrogeologist. The firm of Geraghty and Miller, Inc. provided special technical consultation concerning deep well disposal.

We appreciate the contribution to this work made by Mr. Mel Entus and his staff, as well as Mr. John Ring of the Gulfstream Utilities Company. We look forward to a continuation of this cooperative effort in the future.

Very truly yours,

CAMP DRESSER & McKEE INC.

Charles H Bolton

Charles H. Bolton, P.E.
Senior Vice President

Donald G. Munksgaard, P.E.
Project Manager

CHB,DGM/jv

File: 6009-42-RT

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SECTION 1
EXECUTIVE SUMMARY

1.1 INTRODUCTION

The City of Plantation and the Gulfstream Utilities Company (Gulfstream) are in the process of deciding upon a cost-effective method of treating and disposing their existing and future wastewater flows. One of the options available is to construct a joint regional treatment facility to serve the City and Gulfstream. The purpose of this report is to examine in detail the alternative of a joint facility, so that a meaningful comparison with other available options is possible.

A decision must be reached on this matter soon, due to regulatory agency requirements. The Florida Department of Environmental Regulation (FDER) has imposed a compliance schedule on the City's North and South Plants to cease discharge to surface waters by mid-1986. If the City and Gulfstream decide to construct the joint facility, preliminary design must begin by November 1983 in order to meet the deadline.

1.2 WASTEWATER SERVICE AREA AND FLOW PROJECTIONS

The study area for the proposed facility includes two existing wastewater service areas located within the City of Plantation: the East System and the Gulfstream System. Population projections were developed for the study area based upon an average of the estimates found in the City's Master Plan and Comprehensive Plan, as well as the County's 201 Facilities Plan Update. The population projections, which are presented in detail in Section 3, indicate steady growth from the present population of about 57,000 to the year 2005 population of 124,000.

Flow projections were based upon the population values and an estimate of 100 gallons of wastewater flow per capita per day. The per capita estimate was obtained from the three previously mentioned references as well as actual plant flow records. An allowance was added for infiltration and inflow, and a peaking factor of 1.2 was applied to obtain the design flow projections. As seen in Section 3, a design flow of 7.9 million gallons per day (MGD) was estimated for 1985, and the flows projected for 1995 and the year 2005 were 11.4 MGD and 14.9 MGD, respectively.

A present worth analysis was conducted to determine the wastewater plant staging for the planning period which ends in the year 2005. The recommended program involves two phases. Phase I involves the construction of a 10 MGD facility by 1986 which will be sufficient to treat flows from the East System and Gulfstream. An expansion in 1991 will increase the plant's rated capacity to 15 MGD.

1.3 EXISTING FACILITIES

The wastewater treatment facilities within the jurisdiction of the City of Plantation consist of the North Plant, South Plant, Gulfstream Plant, and West Plant. These facilities, with the exception of the West Plant, were evaluated to determine equipment conditions and the facilities' capacities to maintain effluent quality within permit limitations.

The North Plant, located north of Sunrise Boulevard and west of the Holloway Canal, is designed to treat 3.3 MGD. Wastewater treatment is provided through two separate contact stabilization processes. Although some equipment is not operational, such as the comminutors, the digester mixers, and a sludge pump, the North Plant has been meeting its effluent requirements consistently.

The South Plant, a 1.225 MGD combined high-rate trickling filter and activated sludge plant, is located south of Peters Road near Southwest 16th Street and the Holloway Canal. This plant is fully operational and routinely meets its effluent permit limitations. This facility, as with the North Plant, has limited flexibility in that it is not equipped with standby process equipment.

The Gulfstream Plant, a 2.5 MGD activated sludge plant, has all treatment units in service. Since the plant was upgraded in 1976 and includes standby equipment, the facility generally appears to be in better condition than the North or South Plants. The facility appears to be capable of meeting its effluent requirements during the interim operating period.

1.4 RECOMMENDED FACILITIES

Wastewater transmission, treatment, and disposal facilities are required for the complete wastewater system. Facilities recommended for Phase I will be constructed by 1986. Phase II facilities will be place on-line in 1991 and will be sufficient for the service area through the year 2005.

Transmission Facilities

The existing Gulfstream Plant will be replaced with a 16.8 MGD pumping station. A 30-inch force main will be constructed from the pumping station to the proposed plant along the C-12 Canal corridor. An additional 1.2 MGD of flow from Gulfstream will be directed through the existing 8/12-inch force main along West Broward Boulevard.

The existing South Plant will also be replaced with a pumping station, which will be rated at 3.5 MGD. The existing 16/20-inch force main will be sufficient to transport flows from the South service area through the year 2005.

Liquid Treatment Facilities

A present worth analysis determined that the conventional activated sludge treatment process is more cost-effective than the extended aeration process for the proposed facility. The conventional process consists of bar/filter screening, grit collection, primary clarification, aeration, secondary clarification, disinfection, and effluent pumping.

The bar/filter screens are utilized to remove large pollutants from the wastewater stream that could interfere with the operation of pumps, valves, and other mechanical equipment. Two 6 millimeter (mm) continuous self-cleaning bar/filter screens, each capable of passing a maximum flow of 20.8 MGD of wastewater, will be provided for Phase I. One additional unit will be added for the Phase II plant expansion.

Grit includes such materials as particles of sand, gravel, and nonodorous organics such as coffee grounds and fruit seeds. Grit is removed from the treatment process to protect moving mechanical equipment from abrasion and abnormal wear, to reduce conduit clogging from deposition of grit, and to avoid loading the treatment works with basically inert material. Two 16 feet in diameter "forced vortex" type grit collectors will be provided for Phase I, with one additional unit to be installed for Phase II.

Primary clarifiers are utilized for the removal and disposal of settleable organic solids in the wastewater stream. The recommended primary clarifiers are the under-floor center-column feed and concentric weir overflow type. Due to the high odor potential associated with primary clarifiers, they will be covered with aluminum geodesic covers. Two 80 feet in diameter primary clarifiers will be provided for Phase I, and one additional unit will be installed for the Phase II expansion.

Microorganisms, which use the colloidal and soluble organic matter found in wastewater as a source of food and energy, require oxygen to grow and reproduce. Mechanical aeration has been recommended for this purpose in the proposed facility. Two aeration basins with an installed capacity of 325 horsepower per basin will be provided for Phase I, with one additional basin to be added for Phase II.

Final sedimentation tanks, or secondary clarifiers, are utilized to separate the settleable solids produced by the activated sludge process after aeration. The recommended secondary clarifiers are the under-floor center-column feed and peripheral overflow type. Four 75 feet in diameter secondary clarifiers will be constructed initially, with two additional units to be provided as a part of the Phase II expansion.

Chlorine contact tanks will be provided for effluent disinfection. The tanks will be designed with length: width ratios of 10:1 to inhibit short-circuiting. The chlorination system will be sized for a total capacity of 25 milligrams per liter (mg/l) at the design average flow. The chlorinators will be the high capacity, vacuum-operated, solution-feed type. The chlorinators will automatically control chlorine gas feed in proportion to influent flow.

Solids Treatment Facilities

Two alternatives for sludge treatment were examined on a life-cycle cost basis: (1) conventional treatment with centrifuge thickening, anaerobic digestion, belt-press dewatering, and ultimate disposal in a landfill; and (2) the innovative alternative of centrifuge thickening, belt-press dewatering, and mechanical composting. The present worth analysis indicated that the conventional alternative is more cost-effective, based upon the assumptions made in the analysis. However, such factors as the landfill tipping fee and the market value for compost should be re-evaluated during detailed design. Significant changes in these factors could make the innovative alternative more attractive.

Solid bowl centrifuges will be utilized to thicken waste activated sludge from a concentration of about 0.5 - 1.0 percent to 4-6 percent solids. Thickening from one to five percent solids concentration reduces the volume to one-fifth the original volume, thereby reducing the capital and operating costs of subsequent sludge processing. Waste sludge from the primary clarifiers is expected to have a solids concentration of 3-5 percent, so thickening is not required. Two centrifuges with each rated at a capacity of 210 gallons per minute (gpm) will be installed for Phase I, and one unit will be added for Phase II.

Anaerobic digestion will be utilized as the stabilization process unit which is required for sludge to be landfilled. Advantages of anaerobic digestion over other sludge stabilization processes are that it produces methane gas, reduces the total sludge mass, reduces sludge odor, and inactivates pathogens. A two-stage high rate sludge digestion process is recommended for the proposed facility, because higher loading rates can be achieved through improved mixing and higher temperatures. Two fifty-five feet in diameter primary digesters and one fifty-five feet in diameter secondary digester will be provided for the Phase I design. One additional primary digester will be installed for Phase II.

Dewatering is the removal of water from wastewater solids to achieve a volume reduction greater than that achieved by thickening. Belt filter presses will be utilized to increase the sludge solids concentration from about 3-5 percent to about 20-25 percent. Dewatering reduces the sludge volume by three-fourths and reduces the capital and operating costs of the subsequent sludge disposal process. Two 2.0 meter belt filter presses will be provided in the Phase I facility. One additional 2.0 meter press will be included in the Phase II improvements.

Effluent Disposal Facilities

The effluent disposal system will consist of equalization facilities, transfer pumping, effluent pumping, and deep wells.

Equalization facilities will be provided to accumulate flows above the maximum day flow rate. Equalization of the effluent allows the effluent disposal wells to be sized on a maximum day rather than a peak hour basis. The recommended equalization tank is a prestressed composite concrete structure with a capacity of one million gallons for Phase I. An additional tank will be installed for Phase II.

Transfer pumps will pump treated effluent to the equalization tanks from the chlorine contact tanks. Single-stage above-base discharge vertical turbine pumps with constant speed motors will be utilized for this purpose. Three pumps will be provided under Phase I construction, with one pump to be added for Phase II.

Effluent pumps mounted on the chlorine contact tanks will be utilized for disposal of the effluent into deep wells. The pumps will be three-stage, above-base discharge, vertical turbine pumps with two-speed variable-speed motors. Five pumps will be supplied for Phase I and one additional pump will be required for Phase II.

The effluent disposal facilities will consist of a two-well system. One 24-inch diameter well will be developed for Phase I, and a second 24-inch diameter deep well will be installed as a part of the Phase II plant expansion. The deep well system has been designed such that, if one well were temporarily out of service, the other well could dispose of the maximum daily flow through the year 2005.

1.5 COST ESTIMATE AND FINANCING

A construction cost estimate was developed for transmission, treatment, and disposal facilities for the 10 MGD phase. An allowance was made for contingencies, for the cost of related services, and for inflation. The total cost for Phase I was estimated at $27,226,300.

Various alternative sources of funding are available for financing the Phase I wastewater facilities. The major source of financing is expected to be from the issuance of additional debt, possibly revenue bonds. The use of existing funds on hand and the accumulation of revenues from capacity charges and perhaps wastewater bill surcharges could lessen the amount of additional debt required. Increases in wastewater rates would be required to support additional debt. Economies of scale in operating one treatment and disposal facility as compared with three, however, would tend to offset increases in rates needed to service such additional debt. Large user charges to Gulfstream would recover the cost of serving residents in that utility's service area. These charges could vary depending upon any up-front contributions Gulfstream might provide and the basis used for allocating costs. It is estimated that Gulfstream would also benefit from the economies of scale resulting from operating a single treatment and disposal facility. An estimated increase in the range of $6.24 to $9.92 per month could be expected for East System customers, and a range of $1.51 to $2.05 per 1,000 gallons could be expected for a large user charge to Gulfstream.

1.6 IMPLEMENTATION

Due to the regulatory requirement of no discharge to surface waters by mid-1986, the timing for construction of this facility is critical. Two months are allowed for client and regulatory review of this design report, and preliminary design must begin by November 1983. Preliminary and final design will require six months and will be completed by June 1984. The construction contract could be awarded by January 1985, with construction to be completed by mid-1986. Compliance with this schedule, which appears graphically in Section 14, is necessary to meet regulatory requirements.

SECTION 2
INTRODUCTION

2.1 BACKGROUND

A brief review of background information is important in understanding the present wastewater options available to the City and Gulfstream. Several different scenarios have been analyzed in regional planning documents, and still other alternatives have been proposed in response to recent developments in the regional wastewater systems.

The Facility Plan on Wastewater Management Systems for Broward County, Florida, known as the original 201 Plan, was prepared by James M. Montgomery Consulting Engineers in March 1978. This planning document analyzed four alternatives for facilities planning which ranged from local treatment to maximum regionalization. The study recommended that the County be divided into three districts, called the North, Central, and South regions. Flows from Plantation and Gulfstream would be directed to the South region to be treated at the Hollywood regional facility.

The 201 Facilities Plan Update was completed in September 1982 by Russell and Axon, Inc. and PRC Harris. In this report, three possible treatment sites were considered for flows from Plantation and Gulfstream: the Broward County North District Regional Wastewater Treatment Plant (NDRWWTP), the Sunrise #3 plant, and the Hollywood facility. It was concluded that the difference in costs among the three alternatives was negligible. Therefore, since there was an existing contract for treatment of 1.5 MGD at the NDRWWTP, that alternative became more economical. Based upon economic considerations and ease of implementation, the 201 Plan Update recommended that Plantation and Gulfstream transfer their flows to the NDRWWTP.

A report entitled Evaluation of Wastewater Treatment and Disposal Alternatives was prepared in June 1983 by Camp Dresser & McKee Inc. (CDM). The report evaluated the three following wastewater alternatives: transmission of Plantation's flows to the NDRWWTP, with Gulfstream building its own facility;

Plantation and Gulfstream jointly constructing a treatment plant; and Plantation and Gulfstream each building their own facilities. The report concluded that, depending upon the funding methodology used to allocate capacity costs of the NDRWWTP to Plantation, it may be more cost-effective for Plantation and Gulfstream to jointly construct a treatment facility.

The report further recommended that the City proceed immediately with plans to meet the Florida Department of Environmental Regulation compliance schedule for the South Wastewater Treatment Plant. The temporary operating permit for the South Plant calls for diversion of effluent discharge from substandard waters according to the following schedule: design of interconnecting force main and pumping station(s) complete by March 31, 1983; financing complete by November 30, 1983; start construction of interconnecting force main and pumping station(s) by May 31, 1984; and interconnecting system completed and operational by May 31, 1986, with all flow being diverted to a regional facility.

A temporary operating permit was recently granted to the North Treatment Plant. The compliance schedule calls for submission of a permit application to construct an interconnecting line or alternative disposal structure by November 30, 1983. Construction must begin by February 29, 1984, and discharge to the surface water must be ceased by June 30, 1986.

Plantation will have to take action immediately if the deadlines for diversion of flows are to be met. There are presently two primary uncertainties related to Broward County's North Regional Wastewater System that prevent Plantation from proceeding with steps to divert flow to the NDRWWTP. One uncertainty is the question of whether the Western Interceptor will be constructed, especially in light of the apparent lack of federal funding. The other uncertainty relates to the cost allocation methodology that would be used to charge Plantation for transmission, treatment, and disposal capacity in the North Regional Wastewater System. Due to the necessity of meeting the deadlines for diversion coupled with the uncertainties related to Broward County, it is important that the City investigate the possibility of establishing its own regional system, with or without Gulfstream. The purpose of this design report is to provide a contingency plan for the City and Gulfstream in the event that a suitable large user agreement with the North District System cannot be negotiated.

Plantation and Gulfstream have an existing temporary agreement with Broward County for treatment and disposal of 1.5 MGD of wastewater at the NDRWWTP. This capacity is allocated between Plantation and Gulfstream on the basis of 0.8 MGD and 0.7 MGD, respectively. For purposes of this report, it has been assumed that the interim agreement will be discontinued at time of connection to a regional system. Therefore, the plant has been designed for the total flows from Plantation and Gulfstream without subtracting the interim agreement capacity.

2.2 PURPOSE AND SCOPE

The purpose of this design report was to determine the most cost-effective transmission, treatment, and disposal facilities to serve a joint wastewater system between the City of Plantation and the Gulfstream Utility Company. A detailed financial analysis was included in this report to facilitate comparisons between the joint system alternative and the other options available to the City and Gulfstream, which were discussed in Section 2.1.

The scope of the design report includes a detailed presentation of all facilities required for a complete wastewater system. In this phase of design, unit process design criteria, schematic flow diagrams, a preliminary hydraulic profile, and preliminary site layouts were developed. Decisions related to the unit process type, size, and location were based upon an engineering analysis with consideration of total life-cycle costs. Additionally, as a special element of this document, the financial considerations necessary to fund the project were evaluated. This design report will, upon approval, become the directive for subsequent final design and production of detailed construction drawings and specifications.

2.3 AUTHORIZATION

On June 8, 1983, the City Council authorized CDM to proceed with preparation of the design report as described in their proposal of June 6, 1983. A letter confirming this authorization was received by CDM on June 20, 1983.

SECTION 3
WASTEWATER FLOW AND LOAD PROJECTIONS

3.1 INTRODUCTION

The City of Plantation is essentially a residential community that does not exhibit significant seasonal fluctuations in population. Presently no major "wet" industries are tributary to the wastewater systems. Those industrial, commercial, and medical operations which do contribute wastewaters are a small portion of the total system load. Therefore, the flows generated within the City exhibit the typical characteristics of a domestic wastewater.

In order to determine the requirements for future wastewater treatment facilities in the City, it is necessary to analyze past, present, and future wastewater flows. The following three-step procedure was used to arrive at the projected future wastewater flows in this report: population projection, historical wastewater flow data analysis to determine per capita contribution, and wastewater flow projection.

Section 3 includes the development of wastewater flow and load projections, as well as the selection of plant design criteria. The organization of Section 3 is as follows:

- 3.2 Service Area
- 3.3 Population Projections
- 3.4 Historical Wastewater Flow Data
- 3.5 Infiltration/Inflow
- 3.6 Per Capita Wastewater Flow
- 3.7 Wastewater Flow Projections
- 3.8 Treatment Plant Staging
- 3.9 Historical Wastewater Load Data
- 3.10 Plant Design Criteria

3.2 SERVICE AREA

Three distinct wastewater service areas are located within the City of Plantation as seen in Figure 3-1: the East System, the West System, and the area served by Gulfstream. The area served by the West Plant is not within the scope of this report since that facility is presently owned and operated by the City of Sunrise.

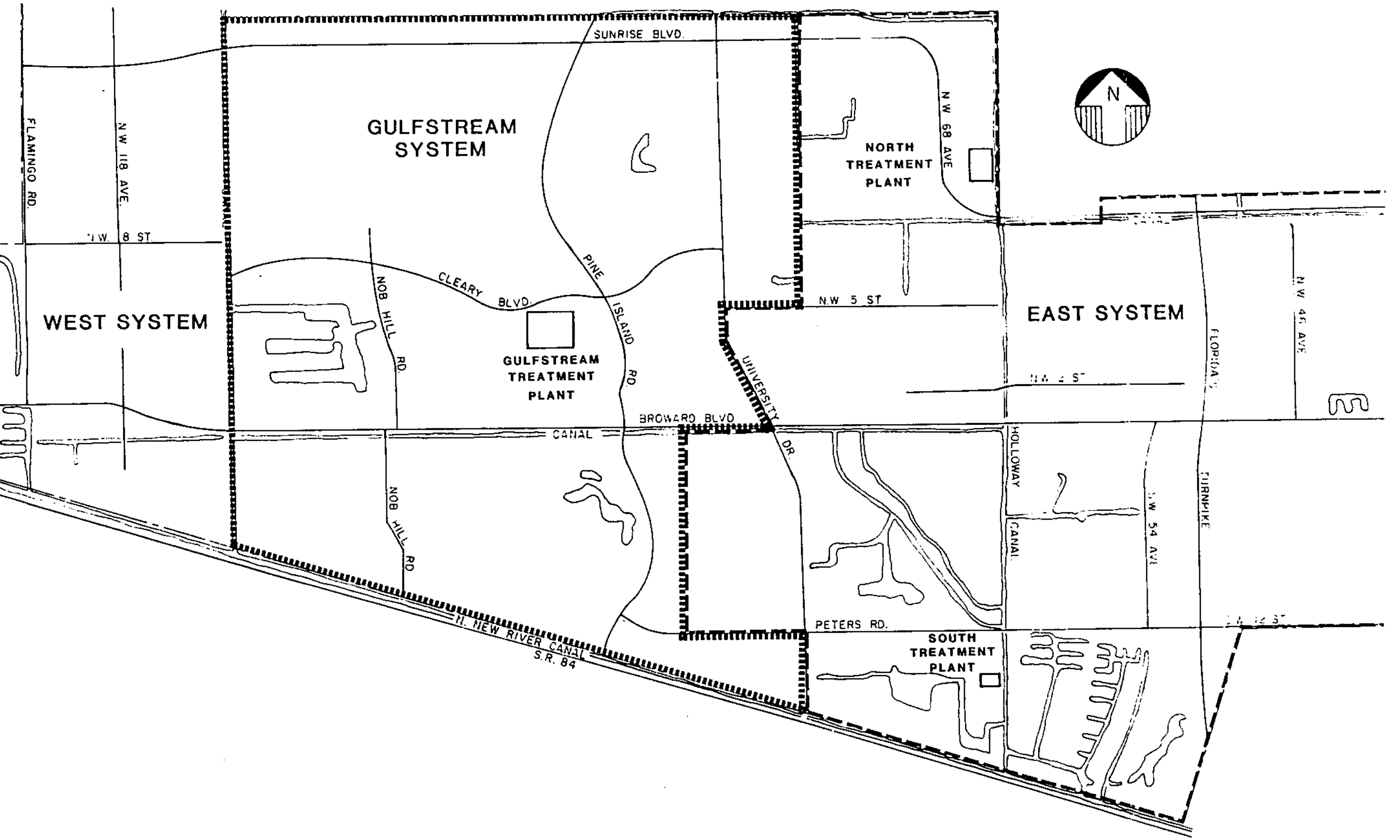
The East System serves the customers within the City limits from approximately State Road 7 to University Drive. There are several unsewered sections in the East System, including areas around the Fort Lauderdale Country Club, Westgate Lake Manor, Old Plantation, and some development along Tropical Way. The East System is served by two treatment plants: the North Plant which has a capacity of 3.3 MGD and the South Plant which is rated at 1.225 MGD.

The area within the City between University Drive and Hiatus Road is served by a 2.5 MGD wastewater plant that is owned by a private company, the Gulfstream Utility Company. As portions of the Gulfstream area are developed, sanitary sewers are constructed according to City standards.

3.3 POPULATION PROJECTIONS

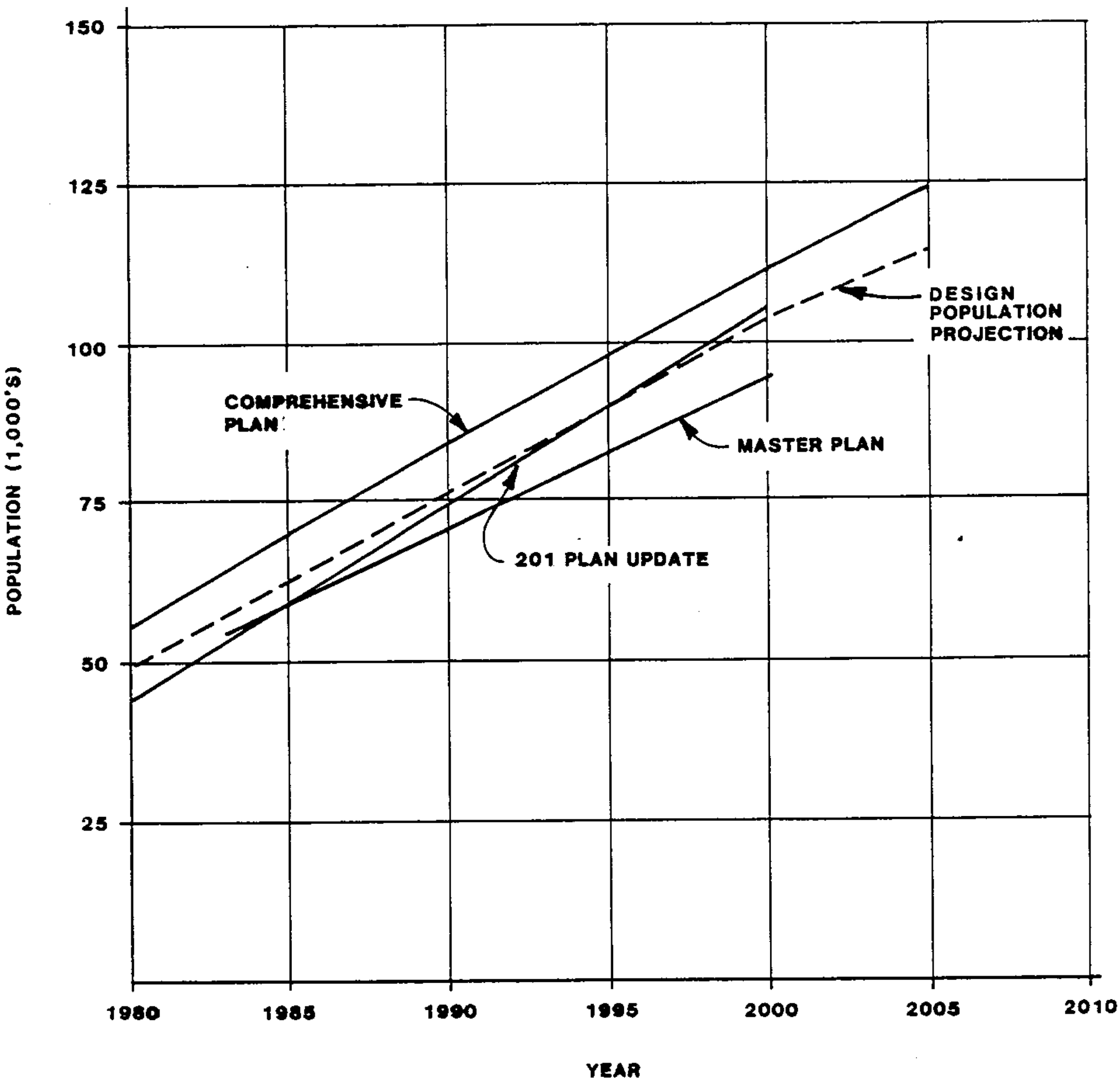
Three references were utilized for the development of population projections: the City's Water and Wastewater Master Plan completed by CDM in January 1978; the Comprehensive Plan prepared by the City's Comprehensive Planning Board in 1981; and Broward County's 201 Facilities Plan Update which was completed in September 1982. An average of the population projections from these three sources formed the basis for the projections used in this report.

Population projections in the Master Plan were based upon the 1977 Broward County Land Use Plan figures. Population estimates were developed in the Master Plan for each year from 1977 through 1985, and then at five-year intervals through the year 2000. The sum of the East System and Gulfstream population figures appears in Figure 3-2 as the Master Plan projection.



SERVICE AREA

FIGURE 3-1



POPULATION PROJECTIONS
FOR THE EAST SYSTEM AND GULFSTREAM

FIGURE 3-2

The Comprehensive Plan estimates the number of dwelling units (DU's) in the year 2000 from the 1981 Land Use Plan. Acreage is multiplied by land use density for each land use category to obtain total DU's for each flexibility zone. Flexibility Zones 73, 74, and 76 comprise the East System, and Flexibility Zone 75 is the Gulfstream service area. The Comprehensive Plan assumed a factor of 2.7 persons per DU to obtain an ultimate population of 135,700 in the East System and Gulfstream. It was estimated that the East System was 80 percent developed in 1980 and would reach maximum development by the year 2000. Gulfstream's service area was assumed to be 20 percent developed in 1980, with build-out occurring in the year 2010. The growth rate between 1980 and ultimate development was assumed to be linear. The result of these calculations appears in Figure 3-2 as the Comprehensive Plan projection.

The 201 Plan Update listed population estimates for 1980 and the year 2000 for the North and South service areas (the East System) and for Gulfstream. The data were derived from the University of Florida Bureau of Economic and Business Research and further disaggregated based upon the Broward Area Transportation Study projections for each traffic analysis zone. The growth rate between 1980 and the year 2000 was assumed to be linear, as seen in the 201 Plan Update projections in Figure 3-2.

An average of these three population estimates was used to derive the projection used in this report, which appears as a dashed line in Figure 3-2. Beyond the year 2000, the slope of the projection is equal to that of the Comprehensive Plan, which is the only reference which projected beyond the year 2000. Figure 3-2 illustrates the relatively close agreement among the three population references, which promotes greater confidence in the population values used in this report.

The percentages of the population sewered in 1980 and 2000 estimated in the 201 Plan Update and the Master Plan were used to calculate the sewered population. It was estimated that the combined East and Gulfstream service areas were approximately 89 percent sewered in 1980, and that they will be 97 percent sewered by the year 2000. A linear increase was assumed in the interim, with 100 percent of the population having sewerage service by the year 2005.

The above population projections were developed based upon the total service area for the proposed treatment plant. However, it is also appropriate to examine the populations associated with each of the tributary service areas individually. Thus, the total and sewered population projections were disaggregated into the East and Gulfstream service areas. Disaggregation of the total population was accomplished by averaging the proportions of East and Gulfstream populations from the three references listed above and applying those factors to the total population as illustrated in Table 3-1.

To determine the disaggregated sewered populations, it was recognized that Gulfstream's sewered population is equal to its total population, since Gulfstream is 100 percent sewered. The East System's sewered population was obtained as the difference between the total sewered population and Gulfstream's population. The results of these calculations appear in Table 3-1.

3.4 HISTORICAL WASTEWATER FLOW DATA

Historical flow data from each of the three wastewater plants were examined to determine such parameters as average annual, peak month, and peak day flows. The ratios of peak month and peak day flows to average annual flow are used in developing design treatment plant capacities later in this report.

Table 3-2 summarizes five years of data from the North and South Plants and three years of data from the Gulfstream Plant. The information was obtained from daily flow records maintained at each treatment plant. As seen in Table 3-2, the ratio of peak month to average annual flow ranged from 1.04 to 1.21. The literature referenced in Table 3-2 recommended a factor of 1.24, and a ratio of 1.2 was selected for design. The ratio of peak day to average annual flow varied from 1.25 to 1.67. The literature referenced in Table 3-2 recommended a value of 1.8, and a design ratio of 1.7 was selected for design.

3.5 INFILTRATION/INFLOW

A sewer system evaluation survey (SSES) was recently completed in the City in order to locate and quantify infiltration and inflow (I/I). The SSES report

TABLE 3-1

DISAGGREGATION OF EAST AND GULFSTREAM POPULATIONS

	1980	1985	1990	1995	2000	2005
TOTAL POPULATION (East and Gulfstream)	49,000	63,000	77,000	90,000	104,000	117,000
AVERAGE PROPORTION (East %/Gulfstream %)	71/29	61/39	54/46	49/51	43/57	39/61
TOTAL EAST POPULATION	34,800	38,400	41,600	44,100	44,700	44,800
TOTAL GULFSTREAM POPULATION	14,200	24,600	35,400	45,900	59,300	70,200
% TOTAL POPULATION SEWERED	89%	91%	93%	95%	97%	100%
SEWERED POPULATION (East and Gulfstream)	43,600	57,300	71,300	85,800	100,600	115,000
MINUS GULFSTREAM SEWERED POPULATION (a)	14,200	24,600	35,400	45,900	59,300	70,200
EAST SEWERED POPULATION	29,400	32,700	35,900	39,900	41,300	44,800

(a) Gulfstream is 100 % sewered as developed, so sewered population = total population.

TABLE 3-2

HISTORICAL WASTEWATER FLOW DATA

	Avg. Annual Flow (MGD)	Peak Month Flow (MGD)	Peak Month/ Avg. Annual	Peak Day Flow (MGD)	Peak Day/ Avg.Ann.
Plantation North					
1978	2.06	2.21	1.07	2.7	1.31
1979	2.16	2.44	1.13	3.6	1.67
1980	2.4	2.5	1.04	3.0	1.25
1981	2.5	2.8	1.12	4.1	1.64
1982	2.49	3.0	1.20	3.6	1.45
Plantation South					
1978	1.11	1.21	1.09	1.40	1.26
1979	1.08	1.15	1.06	1.60	1.48
1980	0.92	1.06	1.15	1.29	1.40
1981	1.09	1.32	1.21	1.72	1.58
1982	1.10	1.20	1.09	1.65	1.50
Gulfstream					
1980	1.74	1.89	1.09	2.18	1.25
1981	1.94	2.15	1.11	2.51	1.30
1982	2.03	2.19	1.08	2.69	1.32
Literature[a]	---	---	1.24	---	1.8
Design Basis	---	---	1.2	---	1.7

(a) "Flow and Load Variations at Wastewater Treatment Plants," Journal Water Pollution Control Federation, August 1980, p. 2131.

indicated the presence of 1.41 MGD of peak infiltration, (or 0.84 MGD of average daily infiltration), in the East System. Gulfstream was not included in the SSES report, but the 201 Plan Update estimated 0.14 MGD of average daily infiltration in that area. The total average daily infiltration for the study area was then estimated at 0.98 MGD.

The SSES report will form the basis for sewer rehabilitation within the City. The report estimated that a cost-effective rehabilitation program could be expected to remove 0.115 MGD of peak infiltration (0.07 MGD on an average day). The average daily infiltration remaining is 0.91 MGD, or 1.1 MGD on a peak month design basis. This quantity of infiltration will remain as a component of wastewater flow after sewer rehabilitation and has been accounted for in the flow projections found later in this report.

3.6 PER CAPITA WASTEWATER FLOW

An accurate estimate of per capita wastewater flow is vital to the development of realistic flow projections. Several sources of per capita flow information were examined and were found to be in close agreement with each other.

The Master Plan determined per capita flows by dividing 1976-1977 annual average flows by the 1977 tributary population for each plant. The results were: 87 gallons per capita per day (gpcd) at the North Plant, 114 gpcd at the South Plant, and 102 gpcd at Gulfstream. A weighted average of these figures yields an annual average figure of 97 gpcd in 1977 for all tributary flows.

The Comprehensive Plan projected flows for 1985 and the year 2000 based upon an overall factor of 100 gpcd on an average annual basis. The figure of 100 gpcd was selected based on past data, allowances for normal I/I, and national trends.

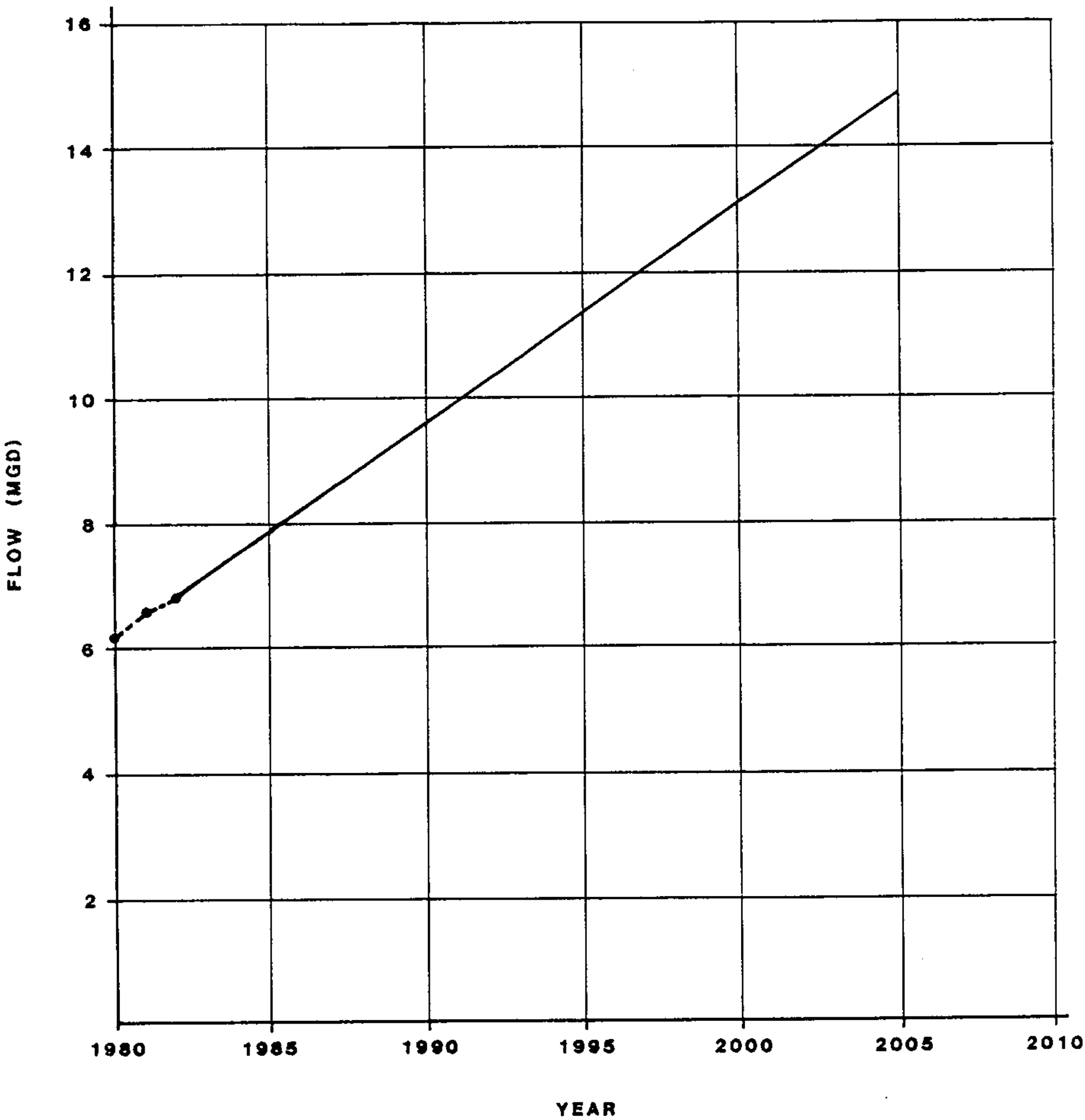
The 201 Plan Update developed overall per capita factors of 100 gpcd for the North Plant, 87 gpcd for the South Plant, and 143 gpcd for Gulfstream. A weighted average of these figures results in a value of 115 gpcd overall on a peak month basis, or 96 gpcd on an average day. However, these figures were based on the "corrected 1980" flows, which exclude excessive I/I. The detailed SSES report for Plantation concluded that a cost-effective rehabilitation program would remove about eight percent of the total I/I. Under this assumption, the per capita figures would be revised to 112 gpcd for the North Plant, 101 gpcd for the South Plant, and 128 gpcd for Gulfstream on an average annual basis. A weighted average of these figures yields 116 gpcd on an average day.

Actual plant flow records were analyzed for 1980 to obtain another estimate of per capita flow. Average annual flows were summed for each plant and divided by the 1980 sewered population to obtain 116 gpcd.

The per capita factors developed from the above sources incorporated commercial, municipal, and industrial flows, as well as domestic flows. An analysis of these sources resulted in the conclusion that 100 gpcd on an average day is an appropriate factor to use for overall base flow. This value was confirmed by subtracting I/I from the most recent per capita value of 116 gpcd, derived from the 201 Plan Update and from actual plant flow records. The I/I component of 0.91 developed in Section 3.5 translates to a 1980 value of 20 gpcd. Thus, the base flow per capita is effectively 96 gpcd on an average annual basis. This factor compares quite favorably with the value of 100 gpcd which has been assumed for base flow in this report.

3.7 WASTEWATER FLOW PROJECTIONS

Future wastewater flow was projected through the year 2005 based upon the sewered population projections developed in Section 3.3 The factor of 100 gpcd for overall base flow developed in Section 3.6 was applied to the population projections to obtain average annual flow. Added to this was the I/I component of 0.91 MGD from Section 3.5. A peak month factor of 1.2 was derived from Table 3-2 and applied to obtain peak month design flows. Figure 3-3 illustrates the results of these calculations. A design flow of 7.9 MGD was estimated for 1985, and the flows projected for 1995 and the year 2005 were 11.4 MGD and 14.9 MGD, respectively.



PEAK MONTH DESIGN FLOW PROJECTIONS

FIGURE 3-3

CDM

3.8 TREATMENT PLANT STAGING

The selection of a staging program for the treatment plant was based upon several parameters. First, modular construction in equal increments was desired due to ease of construction and operation and due to economies of scale. A 20-year planning period was assumed, which resulted in an ultimate design capacity of the plant of 15.0 MGD for the maximum month in the year 2005.

An additional consideration in the selection of the staging program was the timing of the Gulfstream Plant abandonment. It has been estimated that the plant should be abandoned by the early 1990's to avoid prohibitive operation and maintenance costs. However, the plant could be taken out of service earlier without serious economic consequences, since the difference in the remaining plant value is relatively small. The main difference between the two alternatives outlined below is the timing of the Gulfstream Plant abandonment and its effect on the type and size of related facilities.

The first alternative examined the construction in 1986 of an 8.0 MGD treatment plant, comprised of two 4.0 MGD modules, to serve the East System. In 1991, the Gulfstream Plant would be abandoned, and a 7.0 MGD expansion to the new facility would be constructed. Delaying the Gulfstream Plant abandonment necessitates the construction of a force main to transport treated effluent to the new plant and another force main to transport raw wastewater in excess of the 2.5 MGD treatment capacity at Gulfstream. The existing raw wastewater force main in service is insufficient to carry this flow.

The second alternative involves the construction of a 10.0 MGD wastewater plant in 1986, which would consist of two 5.0 MGD modules. This phase would be designed to serve Gulfstream as well as the East System. A 5.0 MGD expansion to the facility would be scheduled for 1991. Abandoning the Gulfstream Plant in 1986 would eliminate the need for dual force mains. One larger force main could be constructed to transport raw wastewater to the new plant site.

A present worth analysis of construction costs was conducted on the two alternatives based on updated cost curves from the <u>201 Plan Update</u>. Operation and maintenance (O&M) costs for the new plant were not included in the analysis since they were assumed to be equivalent for each alternative. However, the incremental O&M cost of the Gulfstream plant remaining in service for another five years was included in Alternative I. As seen in Table 3-3, the difference in costs was less than two percent. Since this difference is not significant at a planning level, a staging program was selected on the basis of ease of implementation. The second alternative was preferred, since the construction of one larger force main for raw wastewater is preferable to construction of two separate force mains for raw and treated wastewater. Furthermore, the second alternative provides for true modular construction with three modules of 5.0 MGD each. Figure 3-4 illustrates the recommended staging plan graphically.

3.9 HISTORICAL WASTEWATER LOAD DATA

Historical values for biochemical oxygen demand (BOD) and suspended solids (SS) were examined to establish loading trends. Data from each of the three treatment plants were compiled for 1980, 1981, and 1982 in Table 3-4. The results are expressed in pounds (lbs.) per day to avoid the unnecessary confusion of weighted averages for BOD and SS concentrations among the three plants.

An estimate of average daily per capita BOD and SS loadings was derived from this data by dividing by the sewered populations for the corresponding years. The data indicates an average loading rate of 0.14 to 0.15 pounds of BOD per capita per day, and 0.14 to 0.17 pounds of SS per capita per day, as seen in Table 3-4. However, the more conservative and widely used figures of 0.17 and 0.20 pounds per capita per day have been assumed for design. These design values are recommended as a minimum by the Ten States Standards, which are referred to by the FDER.

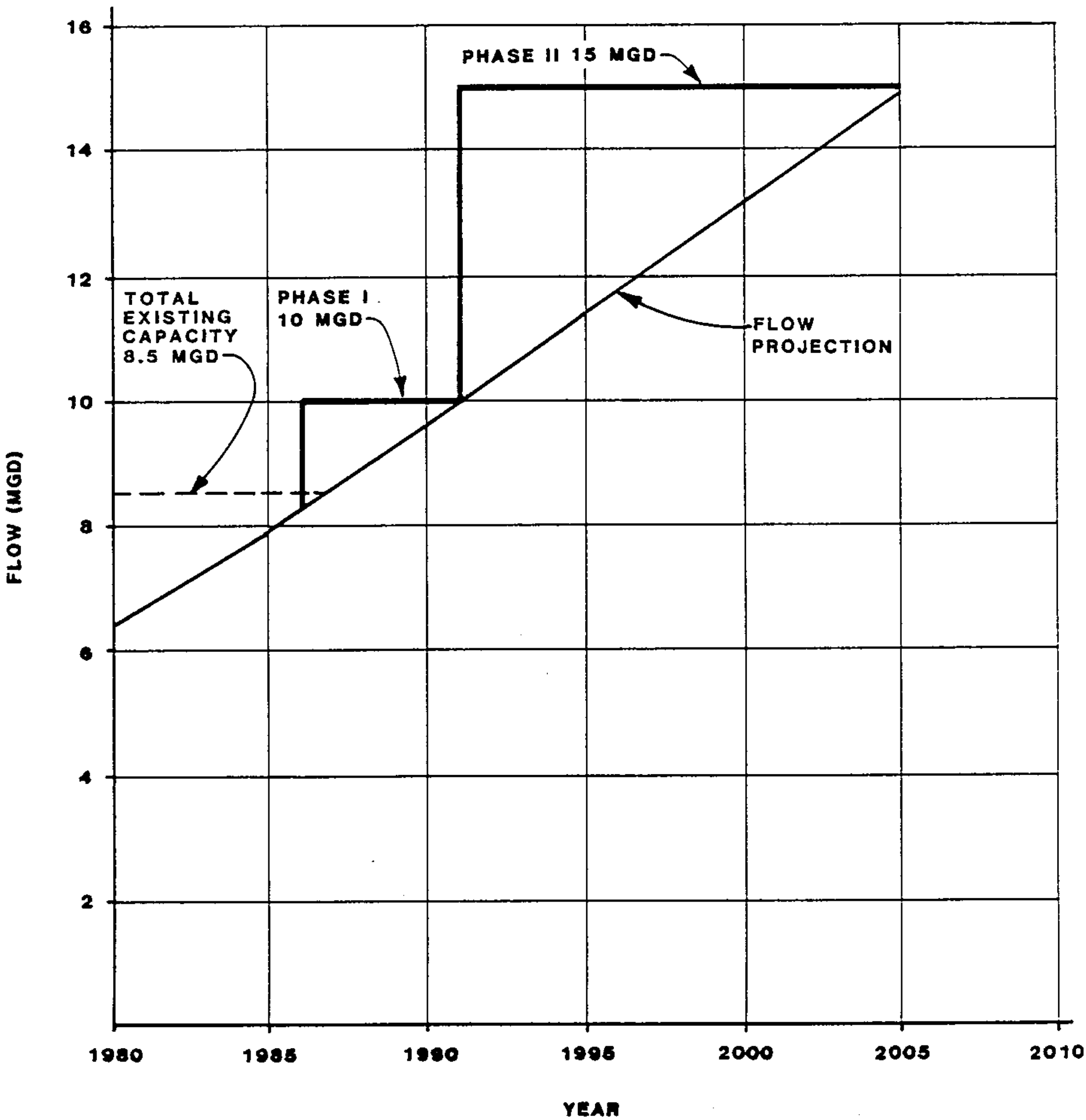
Table 3-5 lists historical values of peaking factors for each of the three plants. The ratios tabulated include the peak monthly to average annual loads for BOD and SS. The historical ratios were compared with those found in the literature before selecting the values to use for design. For purposes of this design, a peak month to average annual ratio of 1.3 was chosen for BOD,

TABLE 3-3
PRESENT WORTH ANALYSIS OF STAGING ALTERNATIVES[a][b]

Treatment or Transmission Facility	Present Worth of Costs in 1986: Alternative I (8/7 MGD, Gulfstream 1991)	Alternative II (10/5 MGD, Gulfstream 1986)
A. 2.67 MGD Pumping Station at South Plant	$ 421,000	$ 421,000
B. Force Main modifications between North and South plants	100,000	100,000
C. Stage 1 Pumping Station at Gulfstream (4.25 MGD for Alt. I, 8.0 MGD for Alt. II)	695,000	1,380,000
D. Stage 2 Pumping Station at Gulfstream (9.6 MGD for Alt. I, 6.2 MGD for Alt. II)	1,043,000	646,000
E. Force mains between Gulfstream and North site (Two 18" lines for Alt. I, One 24" line for Alt. II)	1,090,000	757,000
F. Stage 1 Treatment Plant (Two 4.0 MGD modules for Alt. I, Two 5.0 MGD modules for Alt. II)	10,650,000	12,450,000
G. Stage 2 Treatment Plant (One 7.0 MGD module for Alt. I, One 5.0 MGD module for Alt. II)	6,023,000	4,756,000
H. Incremental O&M Costs of Gulfstream Plant (1986-1991)	138,000	N/A
TOTAL PRESENT WORTH	$20,160,000	$20,510,000

(a) Assumptions:
1. Interest rate @ 10%.
2. Planning period of 20 years.
3. ENR @ 4000.

(b) Information obtained from cost curves in the 201 Plan Update.



RECOMMENDED STAGING PROGRAM

FIGURE 3-4

CDM

TABLE 3-4
HISTORICAL INFLUENT BOD AND SS LOADS[a]

	1980	1981	1982
BOD			
- Average annual loading, (lb/day)	6,188	6,666	7,464
- Estimated Population Served	43,600	46,340	49,080
- Average Per Capita Contribution, (lb/capita-day)	0.14	0.14	0.15
SS			
- Average annual loading, (lb/day)	6,095	7,772	7,614
- Estimated Population Served	43,600	46,340	49,080
- Average Per Capita Contribution, (lb/capita-day)	0.14	0.17	0.16

(a) Combined loading of all three existing wastewater plants.

TABLE 3-5

HISTORICAL PEAK FACTORS

	Peak Month BOD/ Avg. Annual BOD	Peak Month SS/ Avg. Annual SS
Plantation North		
1978	1.13	1.09
1979	1.15	1.18
1980	1.13	1.05
1981	1.13	1.11
1982	1.13	1.23
Plantation South		
1978	1.13	1.10
1979	1.05	1.09
1980	1.12	1.16
1981	1.19	1.17
1982	1.25	1.15
Gulfstream		
1980	1.15	1.33
1981	1.31	1.34
1982	1.24	1.27[a]
Literature[b]	1.31	1.39
Design Basis	1.3	1.4

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[a] SS data for January 1982 was excluded from the analysis, since it was abnormally high.

[b] "Flow and Load Variations at Wastewater Treatment Plants," Journal Water Pollution Control Federation, August 1980, p. 2131.

3.10 PLANT DESIGN CRITERIA

Design criteria for the 10 MGD and 15 MGD phases of the plant are listed in Table 3-6. These parameters were used to size the facilities and include average annual and peak flows, as well as average annual and maximum month influent BOD and SS loadings.

The average annual flow was derived from the peak month design flow by dividing by the peak month factor of 1.2. Peaking factors were then applied to the average annual flow to obtain flow design criteria. The peak month and peak day ratios were developed in Table 3-5. Peak 4-hour and instantaneous peak factors were selected based upon the literature and past experience.

To obtain average annual influent loadings for BOD and SS, the maximum sewered population for each phase was multiplied by the per capita loadings developed in Section 3.9. The peak month loading factors developed in Table 3-5 were applied to obtain maximum monthly influent BOD and SS.

TABLE 3-6

DESIGN CRITERIA

FOR

PLANTATION WASTEWATER TREATMENT PLANT

	10 MGD Phase	15 MGD Phase
FLOW, MGD		
1. Average Annual	8.3	12.5
2. Maximum Month	10.0	15.0
3. Maximum 24-Hour	14.1	21.3
4. Maximum 4-Hour	16.6	25.0
5. Maximum Instantaneous	20.8	31.2
BOD LOAD, (lbs/day)		
1. Population	74,200	115,000
2. Average Annual Influent	12,614	19,550
3. Maximum Month Influent	16,398	25,415
SS LOAD, (lbs/day)		
1. Population	74,200	115,000
2. Average Annual Influent	14,840	23,000
3. Maximum Month Influent	20,776	32,200

SECTION 4
EVALUATION OF EXISTING FACILITIES

4.1 INTRODUCTION

The wastewater treatment facilities within the jurisdiction of the City of Plantation consist of the North Plant, South Plant, Gulfstream Plant, and West Plant. These facilities, with the exception of the West Plant, were evaluated to determine equipment conditions and the facilities' capacities to maintain effluent quality within permit limitations. This evaluation was conducted through a review of operating data from 1981 and 1982, staff interviews, and on-site facility inspections.

Section 4 summarizes the evaluation of the existing facilities as follows:

4.2 Facility Description
4.3 On-Site Investigations
4.4 Equipment Useful Life
4.5 Capacity Analysis
4.6 Summary

4.2 FACILITY DESCRIPTION

North Wastewater Treatment Plant

The City of Plantation North Plant, located north of Sunrise Boulevard and west of the Holloway Canal, is designed to treat 3.3 MGD. Wastewater treatment is provided through two separate contact stabilization processes.

The first process train, rated at 2.2 MGD, was built in 1967 on the site of the trickling filter plant built in 1953. This treatment process consists of:

- Aeration basin utilizing mechanical aeration
- Bar screens (2)
- Clarifiers (2)
- Reaeration utilizing mechanical aeration

- Polishing pond
- Chlorination
- Primary aerobic digestion utilizing mechanical aeration
- Secondary aerobic digestion utilizing diffused air
- Sludge holding tanks (2)
- Sludge drying beds (16)

The new section of the North Plant, rated at 1.1 MGD, is also a contact stabilization process and was built in 1969. The main treatment processes include:

- Aeration basin utilizing floating mechanical aerators
- Bar screens (2)
- Clarifier
- Reaeration basin utilizing floating mechanical aerator
- Polishing pond
- Chlorination
- Aerobic digestion utilizing floating mechanical aerator
- Sludge drying beds (8)

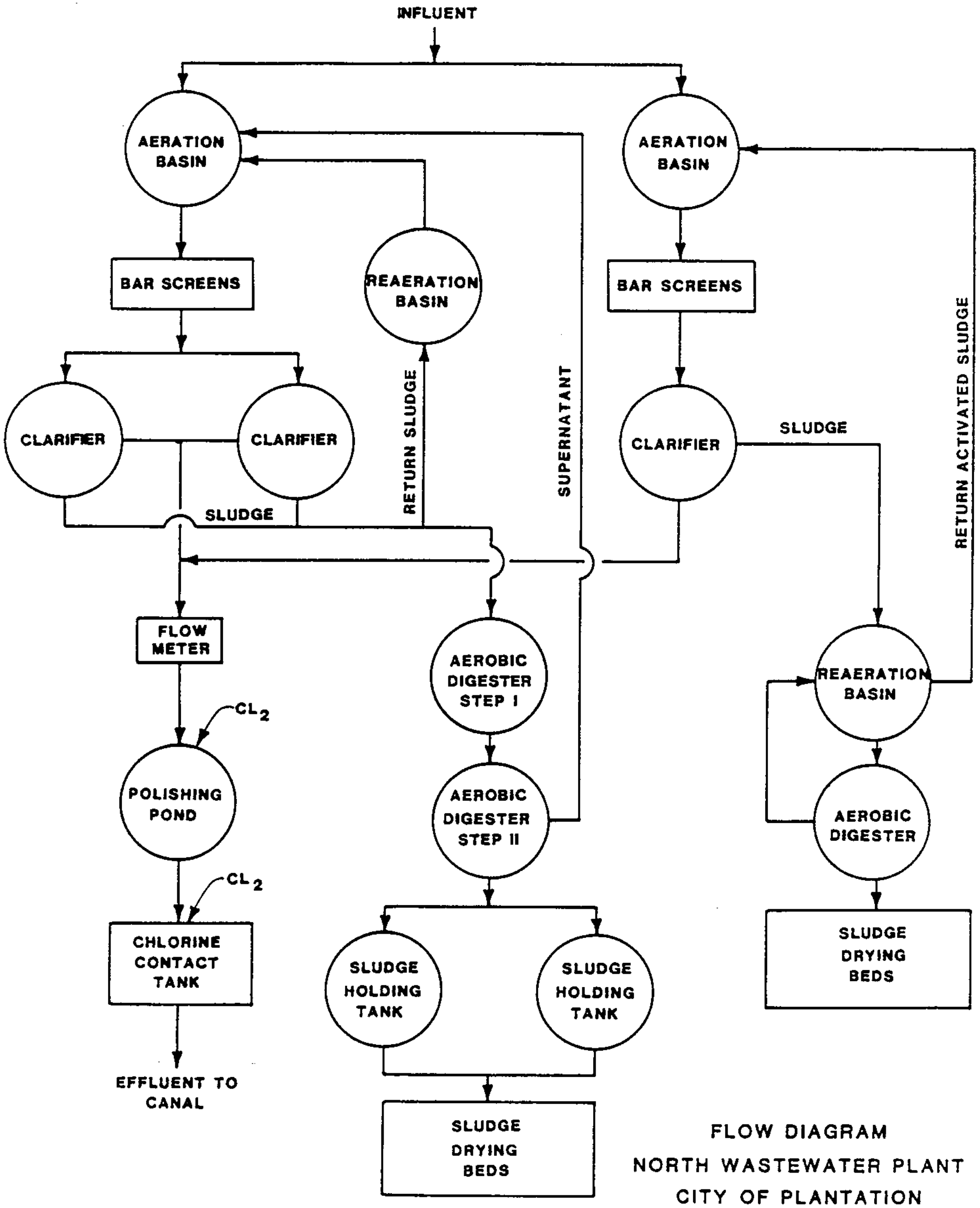
The facility functions as two individual treatment processes with limited flexibility to divert flow from one process train to the other. The North Plant is further illustrated schematically in Figure 4-1.

South Wastewater Treatment Plant

The South Plant, a 1.225 MGD combined high-rate trickling filter and activated sludge plant, is located south of Peters Road near Southwest 16th Street and the Holloway Canal.

The trickling filter portion of the facility was constructed in 1959 and was upgraded to include the activated sludge process in 1968. The treatment process consists of:

- Bar screen
- Aeration basin utilizing mechanical aeration



FLOW DIAGRAM
NORTH WASTEWATER PLANT
CITY OF PLANTATION

FIGURE 4-1

CDM

- Primary sedimentation
- Trickling filter with recirculation
- Final clarifier
- Polishing pond
- Chlorination with reaeration
- Aerobic sludge digestion utilizing mechanical aeration
- Sludge holding
- Liquid sludge disposal

Figure 4-2 provides a schematic flow diagram of the South Plant.

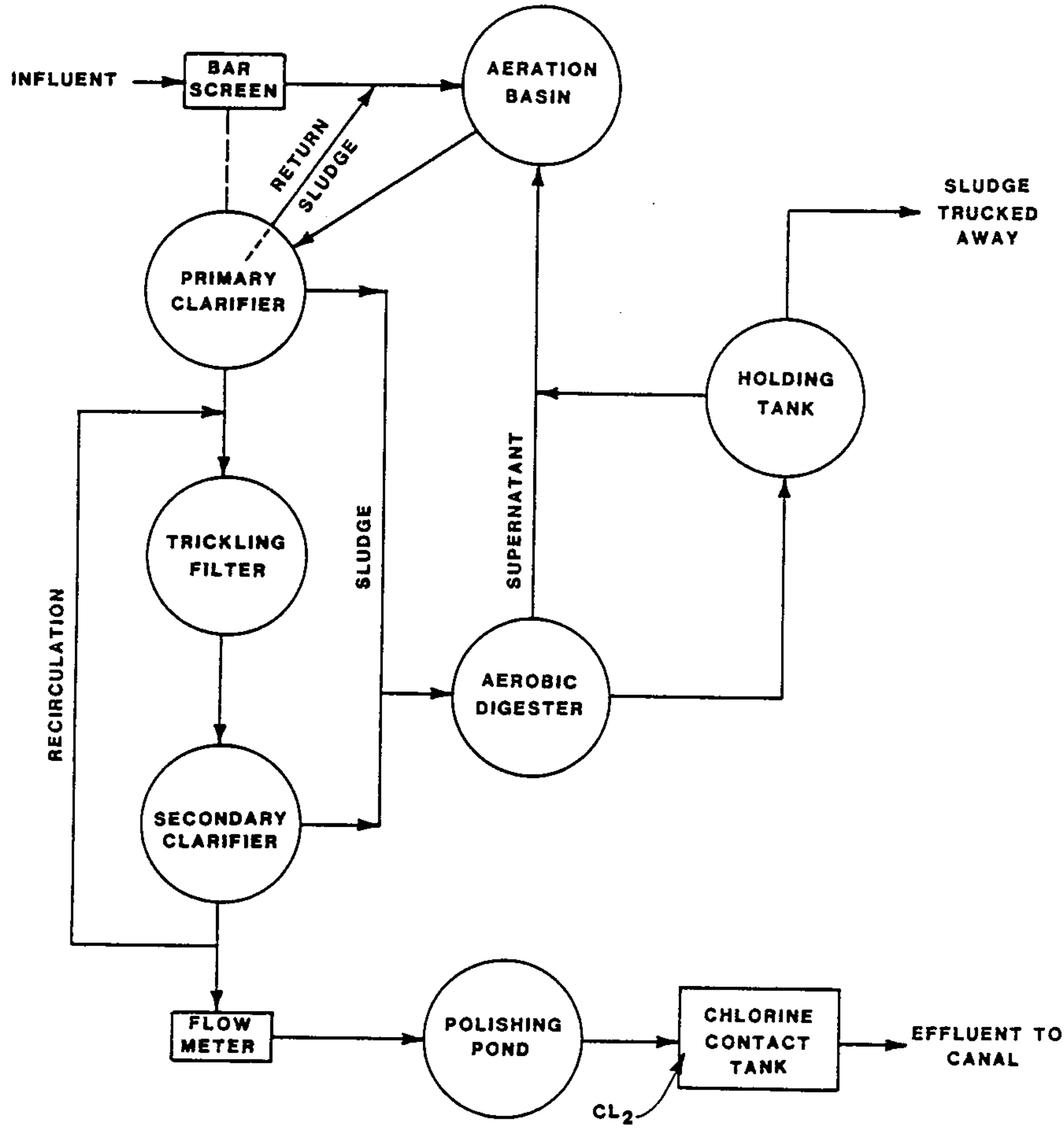
As with the North Plant, there are no redundant treatment units or equipment, which tends to limit the facility's flexibility.

<u>Gulfstream Wastewater Treatment Plant</u>

The Gulfstream Plant, a 2.5 MGD activated sludge plant, was expanded to its present capacity in 1976. The facility consists of:

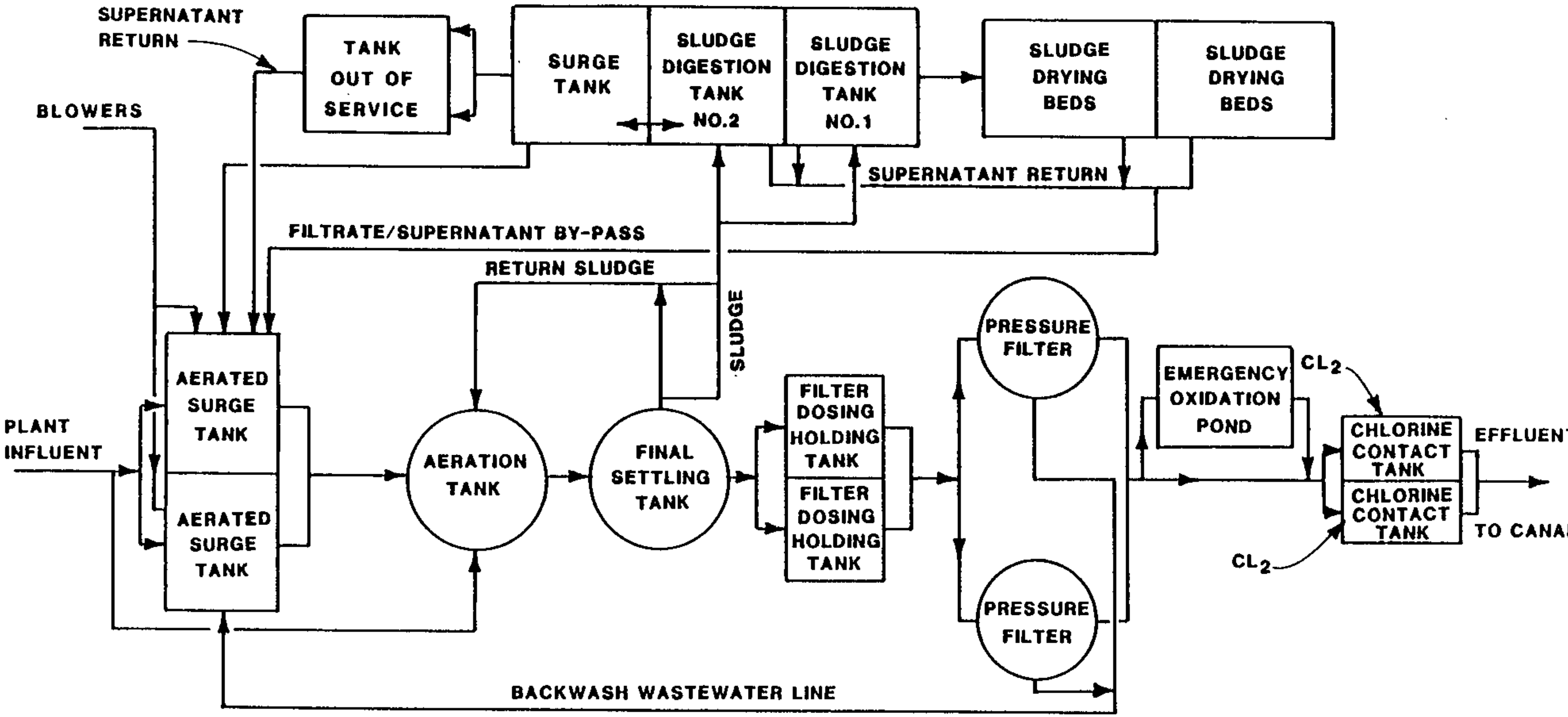
- Aerated surge tanks (3) utilizing diffused air
- Aeration basin utilizing mechanical aeration
- Final settling
- Filter dosing tanks (2)
- Pressure filters (2)
- Oxidation pond (emergency use only)
- Chlorination
- Aerobic sludge digestion (2) utilizing mechanical aeration
- Sludge drying beds or liquid disposal

Figure 4-3 provides a schematic flow diagram of the Gulfstream Plant. Unlike the North and South Plants, Gulfstream has the additional flexibility provided by redundant process units and equipment.



FLOW DIAGRAM
SOUTH WASTEWATER PLANT
CITY OF PLANTATION

FIGURE 4-2



FLOW DIAGRAM
GULFSTREAM WASTEWATER PLANT
(PRIVATELY OWNED)

FIGURE 4-3

CDM

4.3 ON-SITE INVESTIGATIONS

General

An on-site investigation of existing conditions at each of the plants was conducted to evaluate the conditions of the facility's equipment. Based upon these observations and discussions with the facility's staff, a determination of the equipment's remaining useful life has been made.

The following section details the conditions observed during the on-site investigations:

North Wastewater Treatment Plant

The North Plant, divided into two process trains, is fully operational with most major process equipment units functioning. While a few pieces of equipment are not operational, the facility's ability to consistently meet the effluent limits has not been adversely affected.

Specifically, these units include the Worthington comminutors, in both the old and new sections, the secondary aerobic digester mixers, and the Carter positive displacement piston pump. The comminutors have proven to require extensive maintenance to remain operational and provide limited benefit in terms of rag removal. The in-place bar screens provide adequate rag removal. Adequate mixing is provided in the secondary digester through the diffused air system.

The Carter sludge pump also required a significant amount of maintenance to keep in operation. This unit being out of service has not affected operation since an FMC Scruflo pump provides for the sludge transfer from the aerobic digesters to the holding tanks.

No specific equipment operational problems were identified, with the exception of the floating aerators in the "new" plant. These units have required a significant amount of maintenance to keep the units operational. Failures have included cable breakage, motor failures, and floating ring baffle breakage. To address these recurring problems, spare motors are stocked, and a replacement fiberglass ring is being fabricated.

A general observation is that very few units are provided with redundant equipment. The impact of this condition on equipment maintenance is twofold: (1) equipment must be shut down to be serviced, limiting the time available to maintain the equipment due to the adverse affect to the overall treatment, and (2) due to process requirements, the equipment must run continuously, accelerating normal wear.

Table 4-1 lists the major equipment units and provides comments regarding their general condition.

South Wastewater Treatment Plant

The South Plant is fully operational and routinely meets it effluent permit limitations. This facility, as with the North Plant, has limited flexibility in that it is not equipped with standby process equipment. Table 4-2 provides additional detail regarding individual equipment units.

All process equipment was in operation during the site investigation, with the exception of the trickling filter recirculation pump and the aerobic digester mechanical aerator. These units are operational and were shut down for process reasons. Recirculating only at night provides additional hydraulic loading to the trickling filter during low flow conditions. During the decanting of the aerobic digester, the mechanical aerator is shut down due to insufficient submergence of the aerator blades.

Specific problems identified during the inspection include an inoperable inlet valve and check valve on the return sludge pump, final clarifier flooding, and trickling filter distribution arm plugging. Due to the lack of standby equipment and associated piping, the primary clarifier must be drained to make the required repairs to the inoperable valves.

TABLE 4-2

MAJOR EQUIPMENT - SOUTH PLANT

Unit	Manufacturer	Rating	Comment
Primary Clarifier	Dorr Oliver	1/2 HP	Constant use
Return Pumps	Peabody Barnes	3 HP	Constant use; check valve and pump inlet valves do not operate
Aeration Basin Aerator	Yeoman	30 HP	Constant use (no spare)
Trickling Filter Distributor	Dorr Oliver	--	Constant use
Secondary Clarifier	Dorr Oliver	1/2 HP	Constant use
Recirculation Pump		5 HP	Night use during low flow conditions
Final Sludge Pump	Gorman Rupp	5 HP 300 gpm	Intermittent use
Reaeration Blower	Roots	1-1/2 HP	Constant use
Chlorinator	Wallace & Tiernan	1000 lb/day	Pressure type
Flow Meter	BIF		Meter range increase 40 percent to reading range increased due to peaking flow not in range
Generator	Waukesha Engine & Onan Generator	170 kW 213 KVA	275 hours

Due to high flow conditions, the final clarifiers regularly flood during the morning hours. While this is not adversely affecting the plant equipment, it does affect the plant's efficiency.

During the on-site inspection, the trickling filter distribution arms stopped. Typically this is caused by either plugging of the distributor orifices or deteriorated bearings in the distributor turntable.

Plugging of the distributor is indicative of rags passing through the bar screens. This condition is easily remedied at the distributor by cleaning and flushing the distribution arms. This procedure is routinely performed by the plant staff. However, if rags are reaching the trickling filter, it must be assumed that they are present throughout the plant. Rag build-up on aerator blades can cause imbalance and significantly increase wear on the units.

Given the age of the trickling filter, it is reasonable to assume there has been deterioration of the bearings. Due to the impending abandonment of the facility, further investigation to determine the extent of the wear is not warranted.

The facility has an excellent general appearance, although several metal stairways and catwalks pose potential safety hazards. Many locations are severely corroded, and a step to the aeration basin is broken.

This facility appears to be capable of providing adequate treatment through the interim period, but limited capital costs can be anticipated to provide for unpredictable equipment failures and employee safety.

Gulfstream Wastewater Treatment Plant

The Gulfstream Plant, during the on-site investigation, had all treatment units in service. Due to the facility's age and standby equipment, the facility generally appears to be in better condition than either the North or South Plant. Table 4-3 lists major equipment items and provides additional details regarding general equipment conditions.

TABLE 4-3

MAJOR EQUIPMENT - GULFSTREAM PLANT

Unit	Manufacturer	Rating	Comment
Surge Tank Blowers	Spencer	40 HP 846 cfm 20 HP 260 cfm	2 units available; 1 large and unit on constantly 3 units available; 1 small unit dedicated to converted digester constantly
Surge Pumps	Morris	7.5 HP 600 gpm	3 units available
	Morris	40 HP 1500 gpm	2 units available; constant duty
Aeration Surge Tanks	Clow	100 HP	Spare unit available; constant duty
Final Clarifier Drive	Winsmith	3/4 HP	Constant duty
Return Pumps	Morris	40 Hp	2 units; hydraulic varidrives. As drives fail replacing with belts and sleeves, constant duty - one in use
Waste/Scum Pumps	Morris	2 HP	2 units; intermittent use
Filter Feed Pumps/Backwash	Pacific	15 HP 600 gpm	5 units; 3 units one filter, 2 units second unit constant use
Filters	Califilco	4 cell	Control panels show signs of corrosion
Chlorination	Advance	1000 lb/day	Vacuum facility shared with H_2O plant.
Aeration Digester	Yeoman	15 HP	Constant duty, rebuilt 1971
Aeration Digester	Clow	20 HP	Constant duty, rebuilt 1971

Few operational or mechanical problems were evident during the inspection. Those observed include the sludge drying beds, corrosion at the pressure filter control panel, and miscellaneous equipment repairs in progress. Several of the sludge drying beds are not useable due to vegetation growth. This situation has occurred since sludge disposal has been facilitated through liquid hauling and disposal. Plans are in progress to renovate 9 of the 20 beds which are not useable.

The corrosion at the filter control panels has not affected the filter operation but does pose a potential problem. Another factor regarding the filter operation, identified by the plant staff, is the use of final clarifier effluent for backwashing. During occurrences of solids carry-over from the clarifier, more frequent backwashes are required. The backwash water carries the same solids concentration that caused the more frequent backwashing, thereby limiting the effectiveness of the backwash. Although this is not a routine occurrence, it does occassionally limit the filter effectiveness.

The surge pumps and the return sludge pumps are equipped with hydraulic variable-speed drives. As failures have occurred, these units have been replaced with belt and sheave drives. This modification has been undertaken due to reduced cost and reduced maintenance requirements.

In summary, this facility, as with the North and South plants, appears to be capable of meeting its effluent requirements during the interim operating period. Limited capital improvements to address unpredictable equipment failures should be required to meet these treatment needs.

4.4 EQUIPMENT USEFUL LIFE

Generally, equipment designed for wastewater treament plant applications has an expected useful life of 15 to 20 years. Many factors impact whether or not the predicted useful life is realized. These factors include equipment design, care prior to installation, application, maintainability, and mode of operation, i.e., constant or intermittent use. The plant staff has little or no control over most of these factors; they are a function of the facility design and the equipment furnished. The plant staff does have control over the most important aspect, which is equipment maintenance.

As evidenced by the age of the equipment at the South Plant and the "old" section of the North Plant, an adequate level of maintenance has been provided. The equipment in these two areas has provided at least 25 years of service.

While no significant mechanical problems are now being experienced, it can be expected that the equipment in these areas will require an increasing amount of the maintenance budget and the maintenance crews' time. Potential problem areas that will have the most significant impact on the level of treatment provided include the mechanical aerators in the reaeration basin and in the aerobic digester at the North Plant, and the aerators in the aeration tank and in the aerobic digester at the South Plant. These units have not required excessive maintenance in the past, but their age, the lack of a spare unit, and critical impact on the treatment processes warrant close monitoring. Efforts should be undertaken to assure that a substitute unit can be obtained quickly to minimize the adverse affects caused by a failure.

The "new" North Plant mechanical aerators have proven, through past experience, that their expected useful life is significantly less than the 25 years referenced above. A stock of spare parts is currently maintained to keep these units in service. It is anticipated that the equipment reliability will not change, requiring that this stock of spare parts be maintained.

The mechanical aerators at the Gulfstream Plant are as critical to the treatment process as those at the North and South Plants. However, due to their condition and age, a major failure should not be anticipated.

Pumping equipment at each of the facilities appears adequate to maintain operations through the interim operating period. Although the pumping equipment is the same age as the aerators, repairs do not pose the same problems. Repair parts and service are locally available for most of the units, unlike the aerators which must be sent out for an extended time to be repaired.

Also, the availability of standby pumps and portable pumping equipment at the North and Gulfstream Plants provides added flexibility.

In summary, the North Plant and South Plant equipment are nearing the end of their anticipated useful life. An increase in unpredictable equipment failures can be anticipated. However, through routine preventive maintenance and corrective maintenance, the facility's equipment will provide the level of service required to maintain compliance with the effluent requirements through the interim operating period.

The Gulfstream Plant equipment, due to the availability of standby units and lesser number of years in service, can expect a relatively consistent maintenance budget and effort. Should this interim operating period extend beyond 1986, significant repairs can be anticipated for all of the facilities.

4.5 CAPACITY ANALYSIS

The North, South, and Gulfstream Plants have been consistently meeting their respective effluent requirements as indicated in Tables 4-4, 4-5 and 4-6. BOD and SS removal pose no significant problems at any of the three plants. Flow appears to be the limiting factor at each of the facilities.

High flows have affected each of the facilities. Flooding of the final clarifier at the South Plant is routinely experienced. High flows have caused overflows of the trough between the aeration basin and the reaeration basin at the North Plant. In addition, Gulfstream has modified an existing digester to serve as an additional surge tank.

Average daily flows are below design levels at each of the facilities. However, each of the facilities has experienced daily flows in excess of design flows. During the period from January 1981 through December 1982, the North Plant did not exceed design flows. Based upon the review of plant operating

TABLE 4-4

OPERATING SUMMARY - NORTH PLANT

Month	Flow (MGD)	BOD (mg/l) Influent	BOD (mg/l) Effluent	BOD (mg/l) % Removal	Suspended Solids (mg/l) Influent	Suspended Solids (mg/l) Effluent	Suspended Solids (mg/l) % Removal
1981							
January	2.43	156	6	96.2	163	11	93.3
February	2.59	157	5	96.8	160	11	93.1
March	2.44	166	5	97.0	176	9	94.9
April	2.29	168	6	96.4	157	8	94.9
May	2.26	152	10	93.4	160	10	93.8
June	2.59	158	5	96.8	169	10	94.1
July	2.62	142	5	96.5	161	10	93.8
August	2.83	149	6	96.0	167	10	94.0
September	2.73	167	9	94.7	164	9	94.5
October	2.56	148	11	92.7	168	11	93.5
November	2.61	148	4	97.3	165	11	93.3
December	2.58	170	11	93.5	174	11	93.7
AVERAGE	2.54	156.8	6.9	95.6	165.3	10.1	93.9
1982							
January	2.58	159	12	92.5	158	15	90.5
February	2.61	146	10	93.2	162	16	90.1
March	2.60	163	14	91.4	161	15	91.9
April	2.59	164	13	92.1	171	18	89.5
May	2.61	153	9	94.1	171	13	92.4
June	3.00	134	8	94.0	167	13	92.2
July	2.61	155	6	96.1	165	7	95.8
August	2.72	149	7	95.3	171	9	95.7
September	2.60	150	4	97.3	156	8	94.9
October	2.16	146	6	95.9	158	8	94.9
November	1.98	139	5	96.4	166	8	95.2
December	1.83	144	3	97.9	152	8	94.7
AVERAGE	2.49	150.2	8.1	94.6	163.2	11.3	93.1

TABLE 4-5

OPERATING SUMMARY - SOUTH PLANT

Month	Flow (MGD)	BOD (mg/l) Influent	BOD (mg/l) Effluent	BOD (mg/l) % Removal	Suspended Solids (mg/l) Influent	Suspended Solids (mg/l) Effluent	Suspended Solids (mg/l) % Removal
1981							
January	1.01	147	8	94.6	164	11	93.3
February	1.08	135	6	95.6	155	10	93.5
March	1.04	165	11	93.3	144	9	93.8
April	.92	143	8	94.4	167	12	93.8
May	1.02	155	9	94.2	163	11	93.3
June	1.32	138	6	95.7	155	9	94.2
July	1.16	140	8	94.3	159	9	94.3
August	1.21	138	9	93.5	165	12	92.3
September	1.21	140	7	95.0	152	8	94.7
October	1.09	134	7	94.8	161	11	93.2
November	1.07	127	6	95.3	157	10	93.6
December	1.08	147	6	95.9	155	8	94.8
AVERAGE	1.10	142.4	7.6	94.7	158.1	10.2	93.5
1982							
January	1.08	140	11	92.1	143	8	94.4
February	1.16	132	8	93.9	150	12	92.0
March	1.01	139	9	93.5	162	12	92.6
April	1.04	143	9	93.7	158	13	91.8
May	1.09	132	6	95.5	150	10	93.3
June	1.31	144	8	94.4	150	9	94.0
July	1.06	139	6	95.7	150	7	95.3
August	1.09	133	5	96.2	159	8	95.0
September	1.08	130	5	96.2	156	8	94.9
October	1.14	136	8	94.1	161	10	93.9
November	1.20	138	6	95.7	154	10	93.5
December	.99	136	5	96.3	156	10	93.6
AVERAGE	1.10	136.8	7.2	94.7	154.1	9.8	93.6

TABLE 4-6

OPERATING SUMMARY - GULFSTREAM PLANT

Month	Flow (MGD)	BOD (mg/l) Influent	BOD (mg/l) Effluent	BOD (mg/l) % Removal	Suspended Solids (mg/l) Influent	Suspended Solids (mg/l) Effluent	Suspended Solids (mg/l) % Removal
1981							
January	1.76	101	2	98.0	108	8	92.6
February	2.15	151	6	96.0	198	10	94.9
March	2.05	111	2	98.2	215	5	97.7
April	1.73	142	1	99.3	152	4	97.4
May	1.97	119	5	95.8	136	8	94.1
June	1.92	128	3	97.7	165	7	95.8
July	2.00	115	1	99.1	225	5	97.8
August	2.09	118	2	98.3	163	7	95.7
September	1.82	161	4	98.1	208	9	95.7
October	1.78	111	3	97.3	186	10	94.1
November	2.05	150	5	96.7	144	14	90.2
December	1.93	125	4	96.8	178	16	91.0
AVERAGE	1.94	127.7	3.1	97.6	173.2	8.6	95.0
1982							
January	2.03	171	5	97.1	275	16	94.2
February	2.16	141	2	98.6	180	7	96.1
March	2.19	201	8	96.0	-	9	-
April	2.04	214	14	93.5	193	7	96.4
May	1.92	138	7	97.1	128	16	87.5
June	1.95	162	7	95.7	104	9	91.3
July	1.97	194	5	97.4	190	10	94.7
August	1.96	145	4	97.2	150	12	92.0
September	2.02	157	4	97.5	130	6	95.4
October	2.02	212	6	97.2	103	7	93.2
November	2.16	168	5	97.0	165	8	95.2
December	1.99	178	6	96.6	200	14	93.0
AVERAGE	2.03	181.8	6.1	96.6	165.3	10.1	93.9

operating records and staff interviews, the facilities will be able to maintain this level of treatment during the interim operating period, providing flows do not significantly increase.

The present plant permit ratings for the North, South, and Gulfstream Plants are 3.3, 1.225, and 2.5 MGD, respectively. With the addition of the 1.5 MGD interim treatment agreement with the Broward County North District, the total permitted available treatment capacity is 8.5 MGD. Since a wastewater flow projection of 8.25 MGD was developed in Section 3.7 for the year 1986, adequate capacity is available through the interim period until such time as the new regional facility is operational.

4.6 SUMMARY

The North, South, and Gulfstream Plants continue to provide the treatment necessary to remain within their respective permit limits. By maintaining the current levels of operation and maintenance, these facilities should remain in compliance.

However, critical equipment failures can impact a facility's ability to remain in compliance. Specifically these include:

- Failure of the reaeration basin aerator at the North Plant.
- Failure of the aeration basin aerator or the aerobic digester aerator at the South Plant.

As previously stated, these units have not required an extraordinary amount of maintenance in the past. However, due to their critical impact on the treatment process, lack of spare or standby units, and the time required for repair, failure of any of these units has a detrimental impact on the effluent quality.

The Gulfstream Plant has the flexibility of standby units and/or spare units to address equipment failures. This flexibility limits the time any unit will be out of service due to failures, therefore minimizing the effects on effluent quality.

As these facilities are abandoned, usefulness of the existing equipment will be limited. The effect of continuous use, abrasiveness of wastewater, and equipment age preclude the use in any new facility that may be constructed. Salvage value is all that can be realistically expected from the abandoned equipment, unless the units are interchangeable with other equipment within the jurisdiction or other area utilities.

Should the facilities be required to remain in operation beyond 1986, significant expenditures can be anticipated to maintain the current level of treatment.

SECTION 5
WASTEWATER TRANSMISSION FACILITIES

5.1 INTRODUCTION

The existing force main network was analyzed to determine the available transmission capacity. Peak flows were compared with the capacity of the existing system to determine the required size of the proposed force main from the Gulfstream Plant to the new treatment facility.

The Gulfstream Plant and the South Plant will be phased out when the proposed treatment facility is completed. Pumping stations will be built at the Gulfstream and South Plant sites to transport flows from those service areas to the new treatment facility.

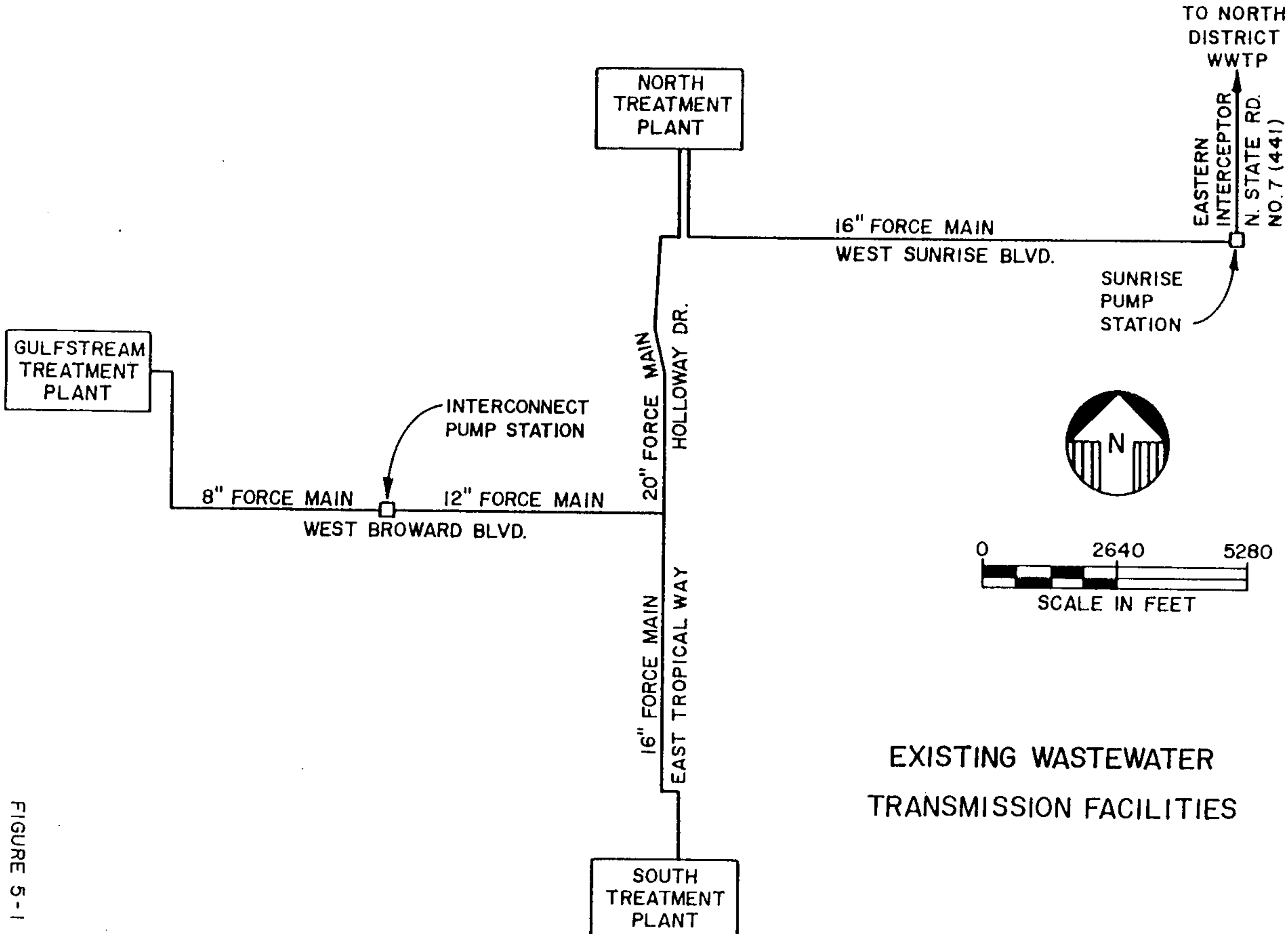
The following aspects of the transmission facilities are covered in Section 5:

- 5.2 Description of Existing Transmission Network
- 5.3 Capacity of Existing Transmission Network
- 5.4 Proposed Transmission Facilities

5.2 DESCRIPTION OF EXISTING TRANSMISSION NETWORK

There are several existing force mains that will be utilized to transport flows to the proposed plant. These facilities are shown in Figure 5-1 and are discussed below.

From the Gulfstream Plant, an 8/12-inch force main runs eastward along West Broward Boulevard to East Tropical Way. At that point it connects with a 20-inch force main which runs northward to the North Treatment Plant. The purpose of this interconnect is to divert excess flow from the Gulfstream Plant to the North Plant for treatment and disposal.



EXISTING WASTEWATER TRANSMISSION FACILITIES

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environmental engineers, scientists, planners, & management consultants

FIGURE 5-1

From the South Plant, a 16-inch force main runs northward and connects with the aforementioned 20-inch force main. This 16-inch force main is presently not in service.

From the North Plant, a 16-inch force main runs eastward along West Sunrise Boulevard and eventually connects to Broward County's Sunrise Pumping Station. Wastewater is then pumped into the Eastern Interceptor on State Road 7, which discharges to the NDRWWTP. Currently, only the portion of the 16-inch force main east of the Big Seven Pumping Station is used for wastewater transmission to Broward County's system.

5.3 CAPACITY OF EXISTING TRANSMISSION NETWORK

An analysis of the existing force main network was conducted to determine the transmission capacity available in the system. The following force mains were analyzed: the 8/12-inch line from Gulfstream, the 16-inch line from the South Plant, and the 20-inch line which begins at the intersection of the 8/12-inch and the 16-inch force mains.

The capacity of the existing force mains was evaluated by balancing two objectives: to minimize pumping and energy costs by reducing the flow rating of the existing mains; and to eliminate the capital outlay for supplementary pipelines by maximizing the flow through existing mains. In addition to these economic objectives, certain hydraulic conditions, such as velocity, must be within an acceptable range. Low velocities will allow solids to settle in the force main. High velocities can result in damage to the pipe and excessive head loss per foot of pipe. The range of acceptable velocities assumed for this design report was 3 to 6 feet per second (ft/sec), with an optimum velocity of about 5.5 ft/sec.

Gulfstream Force Main

The Gulfstream force main consists of 3700 linear feet of 8-inch line which delivers wastewater to the Gulfstream/Plantation interconnect at Pumping

Station No. 112. This station pumps into a 12-inch force main which continues along West Broward Boulevard and joins with the 20-inch force main to the North Plant.

At University Drive, which is just beyond the interconnect, three Plantation pumping stations discharge their flows into the 12-inch force main. Pumping Stations Nos. 105, 106, and 108 serve the Broward Mall and other commercial and proposed multi-family units along University Drive. Flows from these pumping stations were incorporated in the hydraulic analysis of the Gulfstream force main beyond the interconnect.

The capacity of the Gulfstream force main was determined in the following manner. It was assumed that peak hour flows from the University Drive pumping stations would continue to be served by the 12-inch portion of the force main during the planning period. The amount of flow to be carried from Gulfstream was determined by optimizing the velocities in the 8-inch and 12-inch portions of the force main.

The hydraulic analysis determined that the optimal amount of peak hour flow to be diverted from Gulfstream was 1.2 MGD, or 0.58 MGD on a maximum month basis. The peak hour flow from the three pumping stations on University Drive was 1.8 MGD. These flows resulted in velocities of 5.1 ft/sec in the 8-inch portion of the force main and 5.8 ft/sec in the 12-inch portion after University Drive.

South Plant Force Main

The South Plant force main consists of 7000 linear feet of 16-inch pipe along East Tropical Way from the South Plant to West Broward Boulevard. At that point, it connects to the 20-inch force main which is discussed later in this section.

The design capacity for the South Plant was determined by disaggregating the East System into North and South service areas. For present conditions, actual plant flow records were averaged for the last three years. The records indicated that 70 percent of the East System flows were treated at the North Plant and 30 percent at the South Plant. To obtain the future split, the ultimate number of dwelling units in each service area was determined from the Comprehensive Plan flexibility zones. Zones 73 and 74 were assumed to be tributary to the North Plant. Zone 76 was assumed to be tributary to the South Plant, except for the three pumping stations on University Drive, which will continue to pump to the North Plant. Based upon these assumptions, 74 percent of the flow was estimated to be tributary to the North Plant and 26 percent to the South Plant in the year 2005. A linear rate of change was assumed between the present 70/30 split and the 74/26 split in the year 2005.

The average annual daily flow (AADF) at the South Plant in the year 2005 was calculated to be 1.4 MGD based upon the preceding assumptions. Therefore, the South pumping station will be designed for a peak hour rating of 3.5 MGD, and the capacity of the existing 16-inch force main was evaluated on that basis.

Hydraulic analysis of the 16-inch force main indicated that the velocity in the force main under design conditions would be 3.8 ft/sec, which is within the acceptable range. Therefore, the existing 16-inch force main will be sufficient to transport flows from the South Plant.

Combined Gulfstream/South Plant Force Main

The 8/12-inch force main from Gulfstream and the 16-inch force main from the South Plant intersect at West Broward Boulevard and East Tropical Way to form the combined Gulfstream/South Plant force main. This 20-inch main runs parallel to the Holloway Canal to the North Plant for a distance of 7500 linear feet.

The design flow in the 20-inch line is 6.5 MGD on a peak hour basis, which is the sum of the flows from the 8/12-inch and the 16-inch force mains. The resultant velocity in the 20-inch line is 4.6 ft/sec, which falls within the acceptable range. Therefore, the 20-inch main is sufficient to carry a portion of the flows from Gulfstream, the three pumping stations along University Drive, and the South Plant during the planning period.

5.4 PROPOSED TRANSMISSION FACILITIES

In the previous section, the existing transmission network was found to be adequate to transport flows from the South Plant, the three pumping stations along University Drive, and a portion of the flow from Gulfstream. An additional force main is required to transport most of Gulfstream's flow to the proposed treatment plant.

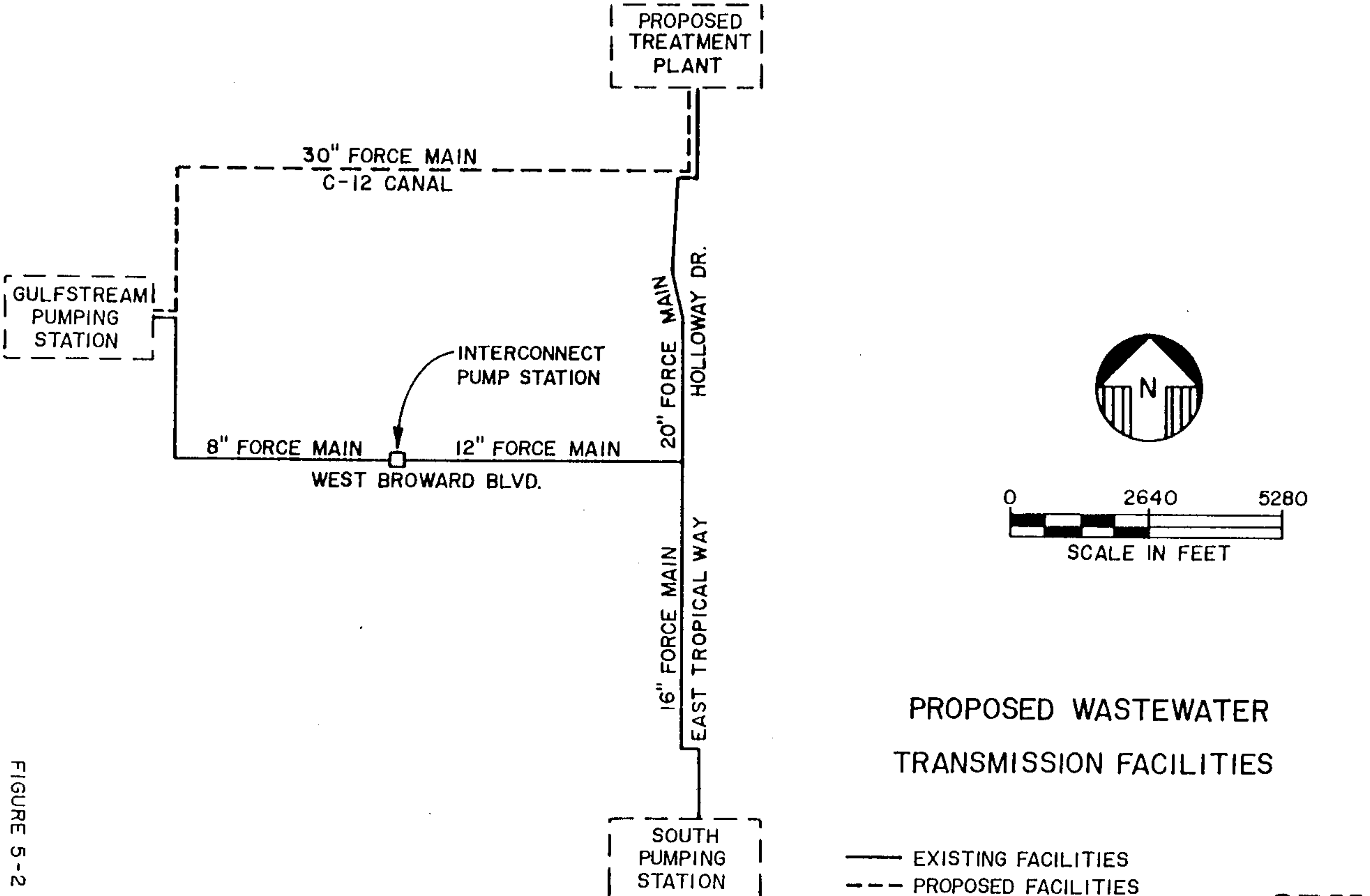
The existing treatment plant at Gulfstream will be abandoned when the proposed facility is built, and a pumping station will be constructed at that site. For purposes of this report, it was assumed that the existing South Plant will also be replaced with a pumping station. However, it may be possible to avoid construction of the South Pumping Station by modifying the existing small pumping stations in the South service area to pump directly into the existing force main. The feasibility of this alternative will be thoroughly investigated in the final design phase. The proposed pumping stations and force main are illustrated on Figure 5-2 and described in the following paragraphs.

Proposed Gulfstream Force Main

The proposed location for the new Gulfstream force main is north from the treatment plant to the C-12 Canal, east along the C-12 Canal to the Holloway Canal, and then north to the plant site, for a total distance of 14,500 linear feet. This route is more direct than the route of the existing force main along West Broward Boulevard.

The total peak hour flow generated in the Gulfstream service area in the year 2005 will be 18.0 MGD. As discussed in the previous section, the existing 8/12-inch transmission main is capable of transporting 1.2 MGD of peak hour flow. The remaining peak hour flow of 16.8 MGD must be served by the proposed force main.

A computer program was used to determine the most cost-effective diameter for the proposed transmission main. The following assumptions were made in this analysis:



PROPOSED WASTEWATER TRANSMISSION FACILITIES

FIGURE 5-2

- Construction cost of 30-inch diameter pipe = $75 per linear foot.
- Roughness coefficient: C = 120.
- Planning period = 20 years.
- Total Overall Efficiency (pump and motor) = 50%.
- Energy Cost = $0.07 per kilowatt hour.
- Peaking Factor = 2.5.
- Interest Rate = 10%.

The results of the analysis indicated that a 30-inch pipe is the most cost-effective selection for the Gulfstream force main. This sizing results in a velocity of 5.35 ft/sec in the pipe, which is within the acceptable range.

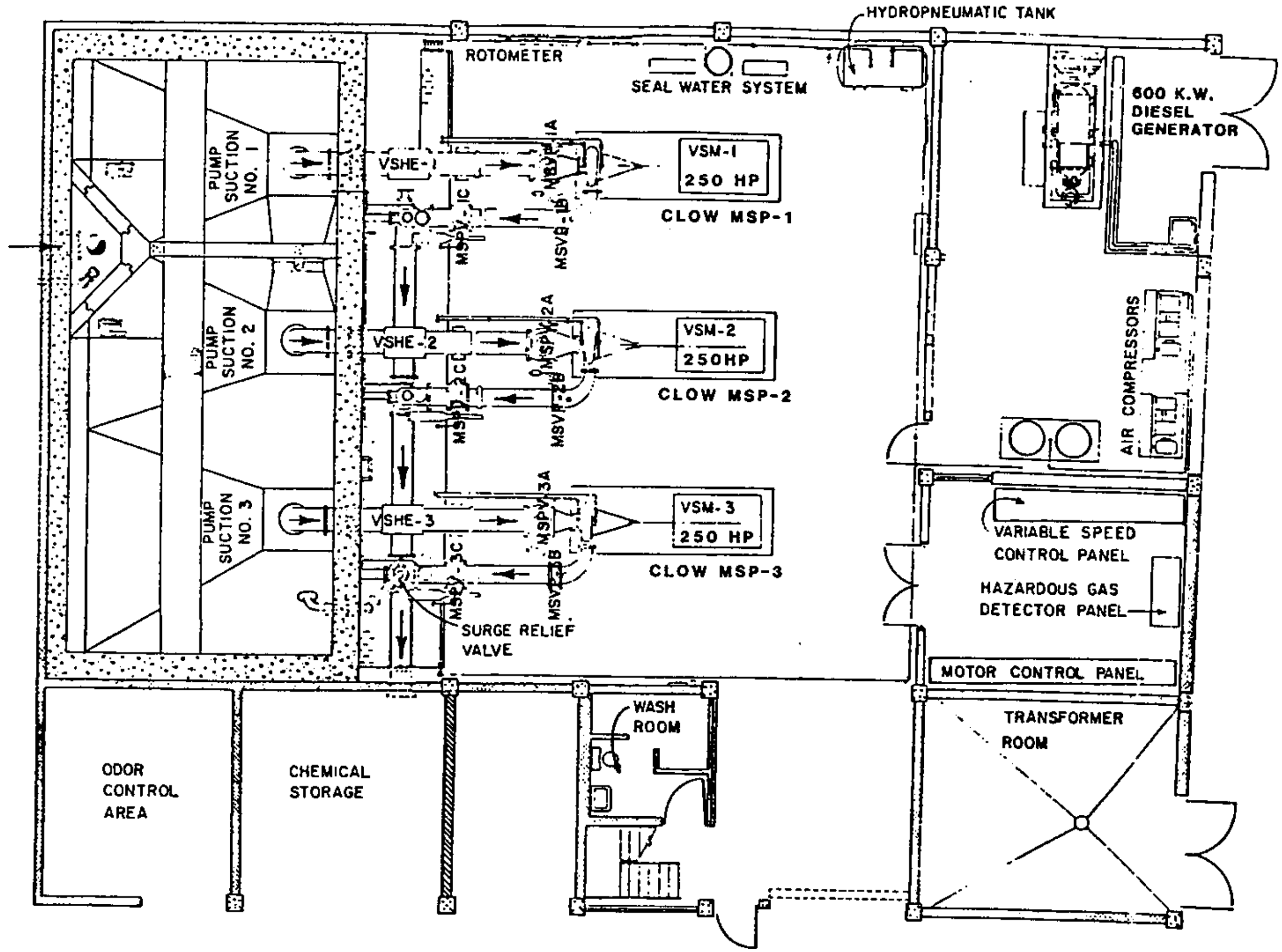
Gulfstream Pumping Station

The existing treatment plant at Gulfstream will be converted to a pumping station when the proposed treatment facility is constructed. As discussed in the previous paragraphs, the design flow for the Gulfstream force main and pumping station is 16.8 MGD.

Figure 5-3 illustrates the proposed layout of the Gulfstream Pumping Station. The station facilities include a wet well and a pump room with three pumps, one of which operates as a standby unit. Individual rooms are provided for odor control equipment, chemical storage, the transformer, the diesel generator, and the pumping station controls.

Pump Selection. In order to select appropriate pumps for the station on a preliminary basis, primary and secondary system curves were developed for the proposed force main. The primary condition was represented by the head loss in 14,500 linear feet of 30-inch pipe at a C-factor of 100, with a static head of 20 feet. The secondary condition was developed similarly using a C-factor of 140.

The system curves for the force main, as well as the pump curves for the selected pump, are illustrated in Figure 5-4. Two Model 18530 pumps manufactured by the Clow Corporation were selected for this application. During Phase I, the pumps will be supplied with 22-inch impellers and 150-HP motors. As seen on



GULFSTREAM
PUMP STATION
LAYOUT

FIGURE 5-3

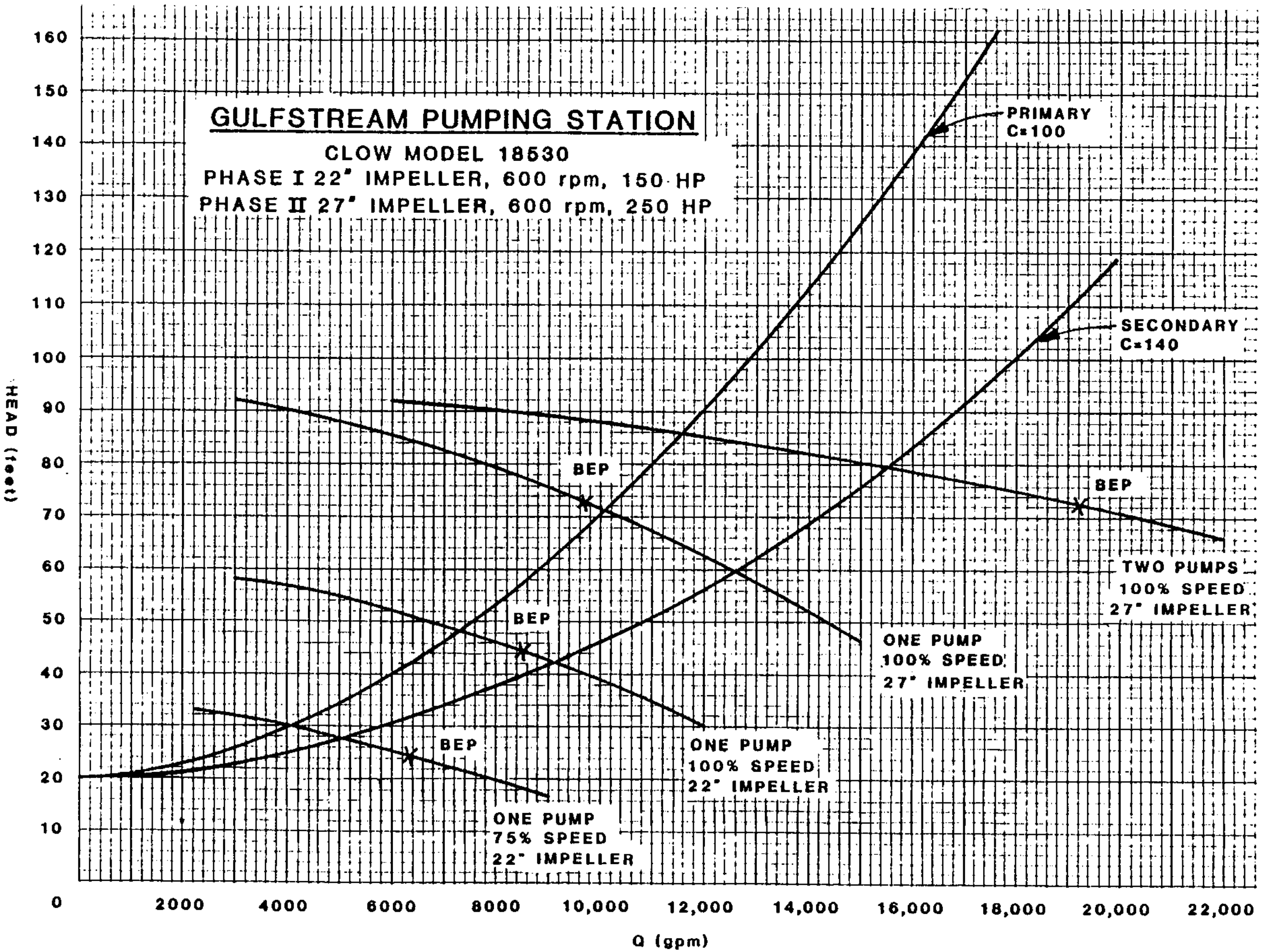


FIGURE 5-4

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Figure 5-4, the primary operation point of the 22-inch installation with one pump in operation is 7200 gpm at 48 feet of head. This size will be sufficient until time for the Phase II improvements, at which point the impellers will be replaced with a 27-inch size and 250-HP motors will be required. As seen in Figure 5-4, the intersection of the primary system curve with the pump curve for two pumps in parallel indicates a primary operation point of 11,700 gpm (16.8 MGD) at 86 feet of head.

Pump Control System. Pump drives may be classified into three general groups: constant-speed, multi-speed, and variable-speed. Selection of a pump control system is governed by the factors of cost, performance, simplicity, and dependability.

Variable-speed drives are recommended for this application due to the diurnal flow variations that will occur at the pumping station and due to the large volume of storage required for constant-speed operation. Although a variable-speed system is more expensive than a constant-speed operation, the performance characteristics are a definite advantage. The system can be designed such that good pump efficienices may be maintained even during low influent flow rates. In addition, variable-speed pump control equipment has demonstrated good performance and reliability.

Three general classifications of variable-speed drives are wound-rotor induction, fluid coupling, and slip coupling. The slip coupling, or eddy-current coupling, was selected for this application due to its simplicity in design and operation and the reduced complexity involved in maintenance.

The eddy-current coupling is an electromechanical torque-transmitting device installed between a constant-speed motor and a pump to obtain adjustable-speed operation. The device resembles a motor in appearance. A constant-speed input shaft is connected to the motor, while the output shaft is connected to the load. The input and output members are mechanically independent, with the output magnet member revolving freely within the input ring or drum member. An air gap separates the two members and a pair of anti-friction bearings maintains their proper relative position. The magnet member has a field winding which is excited by direct current, resulting in eddy currents in the ring. The interaction between these currents and the magnetic flux develops a tangential force tending to turn the magnet in the same direction as the rotating ring. The net result is a torque available at the output shaft for driving a load. An increase or decrease in field current will change the value of torque developed, thereby allowing adjustment of the load speed.

Standby power. Provision of an emergency power supply is necessary in order to maintain operation of the pumping station during an outage of the primary power source. Standby power can be provided by an independent public utility when available, by direct connection of a gasoline or diesel fuel engine to the pumps, or by an electric generator.

For this application, the option of a diesel engine driven generator set was selected. A 600 kilowatt (kw) standby generator set is sufficient for the pumping station's power requirements.

Odor Control. The odor of fresh wastewater is faint and not necessarily objectionable. As the dissolved oxygen in wastewater is depleted, however, anaerobic conditions are created. When the wastewater is putrified by anaerobic bacteria, foul-smelling gases start to form. If the anaerobic condition exists for an extended period, the wastewater will become septic. Bubbles of gas may rise to the surface, and black scum may be present. Two methods of controlling these odors are in-line treatment to prevent the formation of odors, and air-scrubbing at the pumping station to remove odorous substances.

Although other methods of in-line treatment such as ozone, Odophos, and hydrogen peroxide will be investigated as possible methods of in-line treatment during the detailed design phase, past studies and experience indicate that chlorine is the most cost-effective method. Chlorine is effective in odor control because it is a strong oxidizing agent and an effective bactericide. The point of application should be at a sufficient distance upstream of the pumping station to permit effective mixing with the wastewater prior to release of the sulfide gas. To avoid the risk of possible release of chlorine gas to the atmosphere, in-line treatment is normally performed in force mains rather than in gravity flow sewers. Chlorine is injected directly into the force main through a diffuser, and it is necessary that the main be full of liquid at all times.

Air scrubbing, or absorption, is one of the more effective and economical methods of odor control once the odorous gases have been released into an enclosed area, such as a wet well. Odor scrubbers are designed to remove odorous constituents by dissolving them in a liquid, by chemical combination, or by chemical absorption. During the process, some odorous particulates will also be removed. The scrubbing medium is water with a chemical additive. Potassium permanganate solution and sodium hypochlorite are commonly used. Some of the odors that may be removed are hydrogen sulfide, mercaptans, sulfur dioxide, ammonia, and some organic compounds.

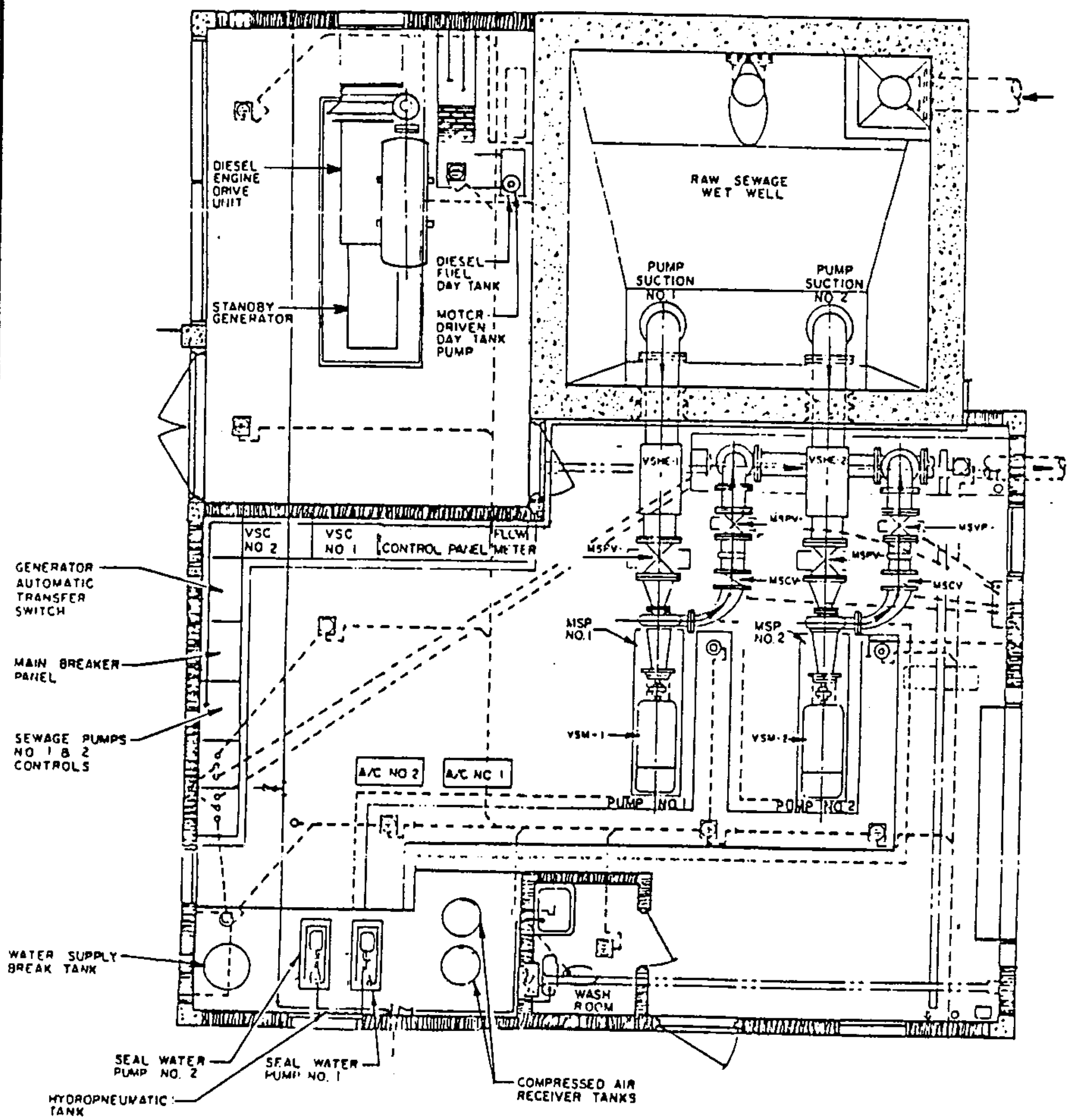
For proper odor control, adequate contact time between the gas and scrubbing medium must be provided to allow interphase diffusion of the gases being absorbed. Equipment that is used to provide the required contact time includes spray chambers, jet scrubbers, venturi scrubbers, packed towers, or tray towers. Because of its relatively low pressure losses, the packed tower currently is the most commonly used type of scrubbing equipment.

In summary, both in-line treatment and air scrubbing are viable methods of odor control. A combination of both methods is recommended for the proposed Gulfstream pumping station.

South Pumping Station

The South Plant will be abandoned when the proposed treatment facility is constructed. The South Pumping Station will be built on that site with a peak hour design flow of 3.5 MGD in the year 2005.

Figure 5-5 illustrates the proposed layout of the South Pumping Station. The configuration of the station is similar to that proposed for the Gulfstream facility, but on a smaller scale. Two wastewater pumps are provided, with one of those to operate as a standby. The pump room also houses the pump station



SOUTH PUMP STATION LAYOUT

FIGURE 5-5

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controls, and a separate room is provided for the diesel generator set. The electric transformer and the diesel fuel storage tank are detached from the main station building.

Pump Selection. Primary and secondary system curves for the South force main appear in Figure 5-6. The primary system curve was based upon 7000 linear feet of 16-inch pipe and 7500 linear feet of 20-inch pipe with a C-factor of 100. A static head of 20 feet was assumed. The primary condition was developed on the assumption that a peak flow of 3.0 MGD enters the transmission system from the 8/12-inch force main. The secondary condition is based upon the assumption that the South Pumping Station is operating by itself, and there is no flow contribution from the Gulfstream 8/12-inch force main. Two secondary curves were developed, for C-factors of 100 and 140, respectively.

For this application, one Model 8518 pump manufactured by the Clow Corporation was selected. The intersection of the pump curve with the primary system curve indicates a primary operation point of 2400 gpm (3.5 MGD) at 99 feet of head.

Pump Control System. A variable-speed drive with an eddy-current coupling is recommended for the South Pumping Station. The reasons for this selection are the same as for the Gulfstream facility, and they were enumerated in the previous section.

Standby Power. A diesel engine driven generator set is recommended for the South Plant as an emergency power source. A 200 KW unit is sufficient to meet the station's power requirements.

Odor Control. Similar to the Gulfstream facility, odor control by in-line treatment and by air scrubbing is recommended for the South Pumping Station. Although odor control facilities are not integrated into the station building shown on Figure 5-5, the equipment will be housed adjacent to the station.

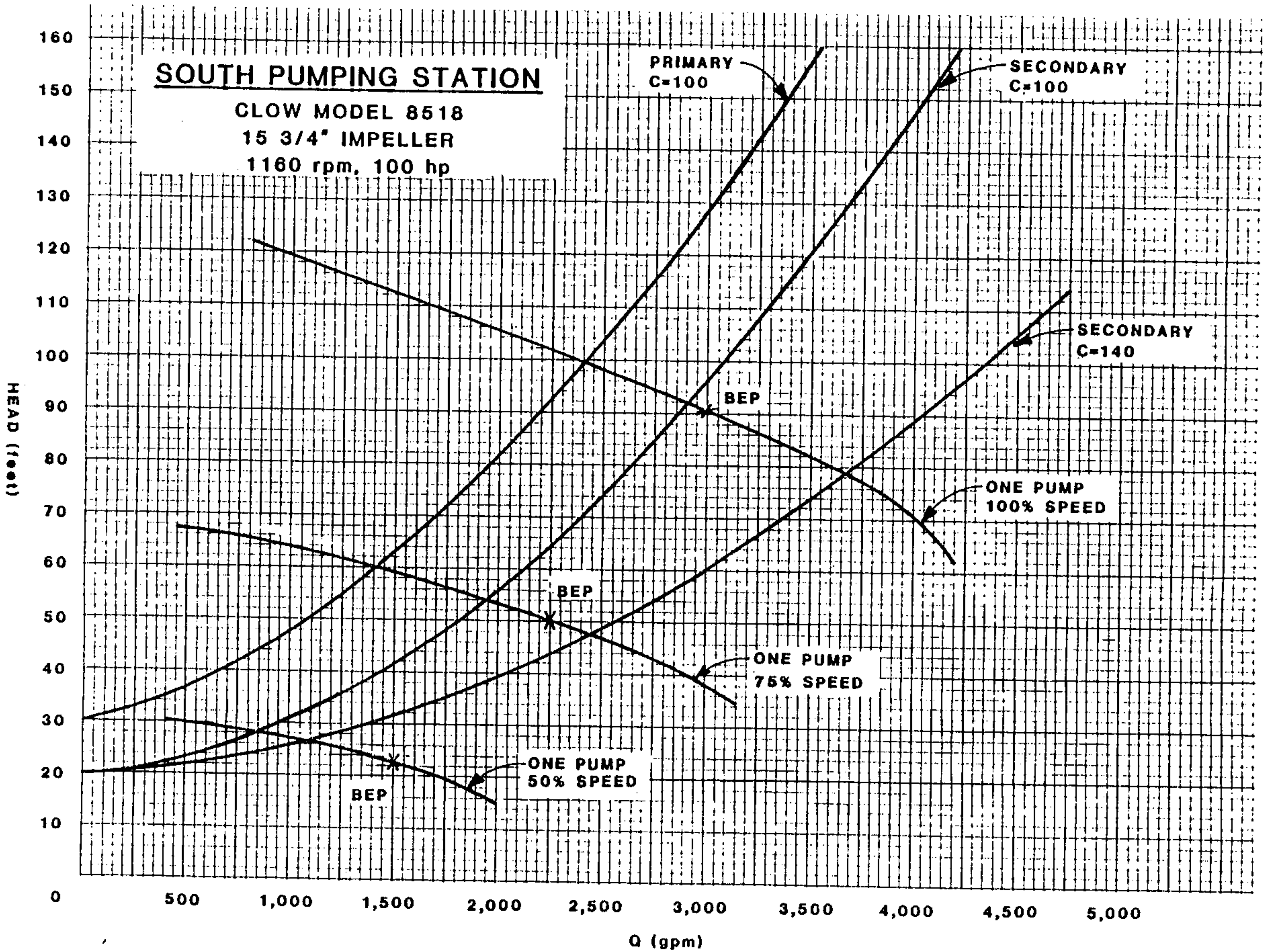

FIGURE 5-6

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SECTION 6
LIQUID TREATMENT FACILITIES

6.1 INTRODUCTION

This section describes, evaluates, and sets forth the design of the individual process units required for the liquid treatment facilities. Based upon the flow projections presented in Section 3, design criteria, equipment selection, and typical drawings are developed for each process unit. Typical cut sheets for equipment are included in Appendix A. The section is subdivided as follows:

- 6.2 Process Selection
- 6.3 Bar/Filter Screens
- 6.4 Grit Collectors
- 6.5 Primary Clarifiers
- 6.6 Aeration Tank and Aeration Equipment
- 6.7 Secondary Clarifiers
- 6.8 Disinfection
- 6.9 Effluent Pumping Equipment

Chapter 17-6 of the Florida Statutes requires a secondary level of treatment for deep well disposal of effluent. Facilities must be designed to achieve an effluent after disinfection on an annual average basis containing not more than 20 mg/l BOD and 20 mg/l TSS, or 90 percent removal of each of these pollutants from the influent wastewater, whichever is more stringent. Additionally, discharge standards of 30/30 on a monthly basis, 45/45 on a weekly basis, and 60/60 at any time are mandated. A properly operated secondary treatment plant will achieve these effluent standards consistently.

Effluent standards for nitrogen and phosphorus removal are not required for a deep well disposal system. However, nitrification is expected to occur, because it is difficult to suppress at the high wastewater temperatures experienced in southern Florida. Limited phosphorus removal is expected because of biological uptake of soluble phosphorus. Total nitrogen and total phosphorus in the effluent is expected to average about 25 mg/l and 9 mg/l, respectively.

The proposed treatment plant will be classified as a "Type I Facility" since Chapter 17-6 defines this category as a wastewater facility having a design average daily flow of 500,000 gallons per day or greater. Additionally, the design will be based upon Class I reliability as described in the publication by the Environmental Protection Agency entitled Design Criteria for Mechanical, Electric, and Fluid System and Component Reliability.

6.2 PROCESS SELECTION

Based upon past experience, two alternative treatment schemes were considered for the proposed treatment plant, conventional treatment and extended aeration. A present worth analysis of the capital and operation costs was conducted to determine the more cost-effective treatment scheme. Cost information was obtained from the Environmental Protection Agency's (EPA's) publication entitled Innovative and Alternative Technology Assessment Manual. All costs were updated by using an Engineering News Record (ENR) Construction Cost Index of 4000. It was assumed that Phase I would be constructed in 1986. Phase II costs and annual operation and maintenance (O&M) costs were converted to 1986 dollars. The two alternatives that were evaluated are outlined below.

The "conventional treatment" alternative consisted of the following unit processes:

- Preliminary treatment - bar screening and grit removal.
- Primary treatment.
- Conventional activated sludge system with mechanical aeration. Power cost of full nitrification was added as a separate item.
- Final clarification.
- Chlorine disinfection.
- Centrifuge thickening.
- Two-stage anaerobic digestion.
- Belt filter press dewatering.

The "extended aeration" alternative consisted of the following unit processes:

- o Preliminary treatment - bar screening and grit removal.
- o Extended aeration - activated sludge process with cost of nitrification included.
- o Final clarification.
- o Chlorine disinfection.
- o Centrifuge thickening.
- o Belt dewatering.

Table 6-1 lists the unit processes for the two alternatives and their associated costs. The present worth of operation and maintenance (O&M) expenses of the plant during the planning period is included as a separate item. Table 6-1 indicates that the cost of the extended aeration alternative is approximately 50 percent greater than that of conventional treatment. The conventional treatment alternative is clearly more cost-effective for the proposed plant. Therefore, a more detailed analysis is not warranted in this case.

6.3 BAR/FILTER SCREENS

Bar/filter screens are utilized to remove large pollutants from the wastewater stream. This process unit primarily results in the removal of solids and trash that may interfere with the downstream operation of treatment plant equipment, such as pumps, valves, and mechanical equipment.

Reliability Class I criteria require a back-up bar screen. Additionally, a plant with only two bar screens must have at least one bar screen designed to permit manual cleaning. Therefore, two (2) units will be provided for the 10 MGD phase with each rated at the peak hour flow rate. Moreover, an emergency by-pass channel will be incorporated into the structure, which will include a manually cleaned bar rack.

The screening process operation will consist of two (2) continuous self-cleaning bar/filter screens. Each screen will be capable of passing a maximum flow of 20.8 MGD of wastewater. The system shall be capable of removing miscellaneous suspended objects (screenings) greater in any dimension than 6 mm

TABLE 6-1

PROCESS COST-EFFECTIVENESS ANALYSIS[a][b]

ULTIMATE 15 MGD PLANT SIZE

Unit Process	Conventional Treatment	Extended Aeration
Preliminary Treatment	$ 340,000	$ 340,000
Primary Treatment	840,000	---
Conventional Secondary Treatment	2,020,000	---
Extended Aeration Treatment	---	5,450,000
Final Clarification	1,490,000	2,400,000
Chlorination/Disinfection	370,000	370,000
Centrifuge Thickening	710,000	860,000
Two-Stage Anaerobic Digestion	1,150,000	---
Belt Filter Press Dewatering	330,000	270,000
Operation and Maintenance	4,740,000	9,740,000
Power Cost of Full Nitrification	890,000	---
TOTAL	$12,880,000	$19,430,000

[a] Present Worth of Costs in 1986.

[b] Costs derived from EPA manual, Innovative and Alternative Technology Assessment, updated to ENR = 4000.

(1/4-inch) from the wastewater stream. The fully-automatic, mechanical bar screen shall be installed in a flow channel 4' - 0" wide X 5' - 0" high.

The screens will provide continuous dual filtration. The coarse filtration will remove all material larger than 14 mm diameter while the fine filtration shall remove all material larger than 6 mm diameter. The screened materials and all other materials up to 7 inches in diameter will be automatically transported by the bar screen for deposit into screening containers.

Recommended design criteria for bar screens from five separate design manuals are summarized in Table 6-2. As noted in the table, the design approach velocity is 3 fps, the angle of inclination is 60 degrees from the horizontal, and the bar width is 3/8-inch.

The preliminary design of the screens using the aforementioned criteria is summarized in Table 6-3 and illustrated on Figure 6-1. Two bar/filter screens will be provided for the 10 MGD phase, and one additional unit will be added during the Phase II plant expansion.

6.4 GRIT COLLECTORS

Wastewater grit materials are characterized as inert and having a settling velocity greater than organic solids. Materials falling into these categories include particles of sand, gravel, and the nonodorous organics such as coffee grounds and fruit seeds. Grit is removed from a wastewater system to protect moving mechanical equipment from abrasion and abnormal wear; to reduce conduit clogging caused by deposition of grit particles; and to prevent loading of treatment works with basically inert matter that might interfere with the operation of treatment units, such as silt in anaerobic digesters or aeration tanks.

The wastewater treatment system must be designed to include grit removal components to meet Class I reliability requirements. Therefore, two (2) units will be provided at the peak hour flow rate.

TABLE 6-2

BAR SCREEN DESIGN CRITERIA

Design Parameter	Design Reference # 1	# 2	# 3	# 4	# 5	Proposed Design
Bar Width Size, (inches)	----	1/4-5/8	1/4-5/8	----	----	3/8
Clear Opening, (inches)	5/8-1 3/4	5/8-3	5/8-3	----	3/4-2	1/4
Approach Velocity, (fps)	>1.25	2-3	2-3	----	1.3-3	3
Slope, (degrees horiz.)	30-45	60-90	60-90	----	45-90	60

References:

#1 - Recommended Standards for Sewage Works ("Ten States Standards"), Great Lakes - Upper Mississippi River Board of State Sanitary Engineers, 1978 Edition.

#2 - Wastewater Engineering: Collection, Treatment, Disposal; Metcalf & Eddy, Inc., 1972.

#3 - Design Criteria for Subregional Wastewater Treatment Facilities, Orange County, Florida; Camp Dresser & McKee Inc., 1982.

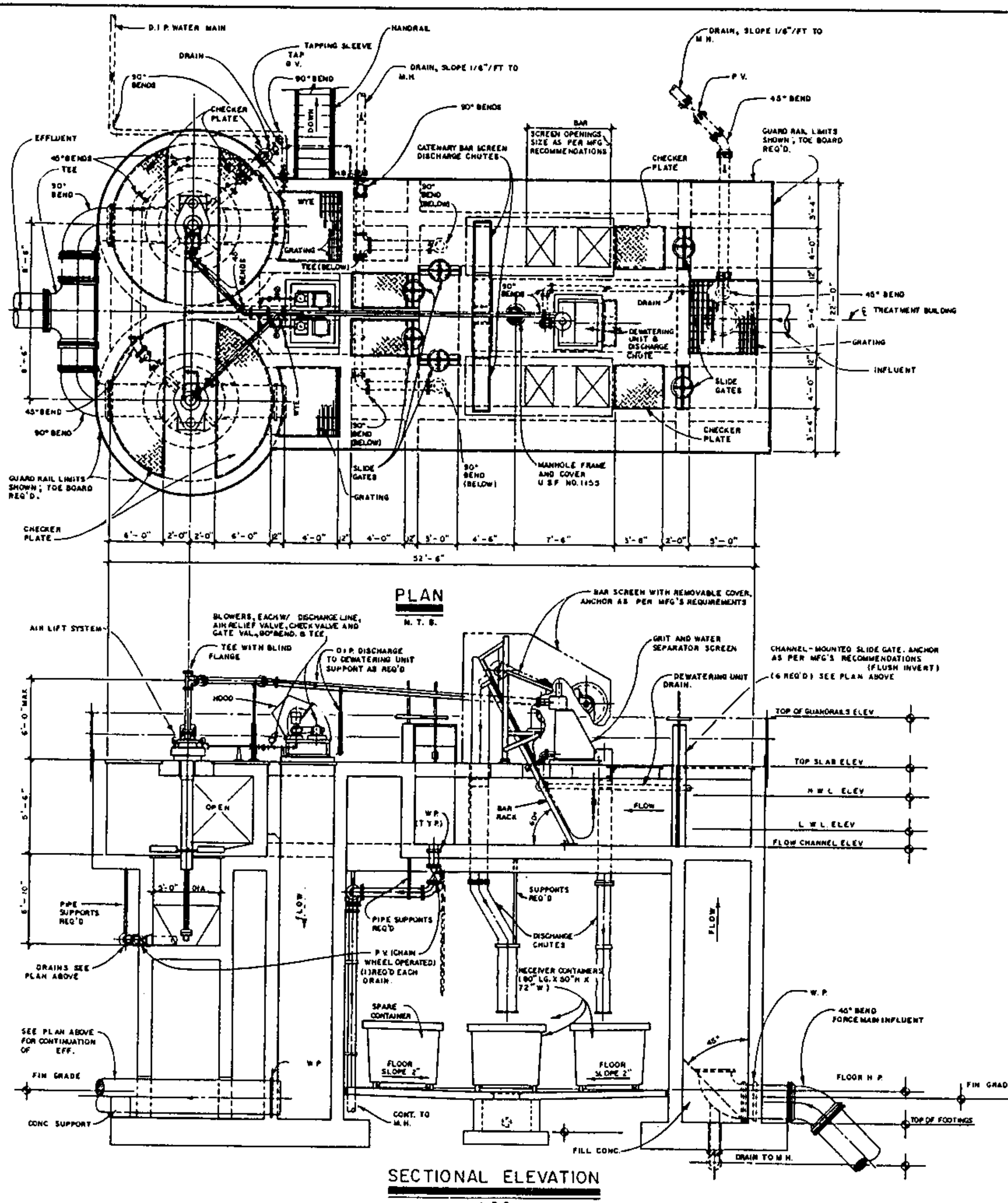
#4 - Process Design Manual for Upgrading Existing Wastewater Treatment Plants, Environmental Protection Agency, October 1974.

#5 - Wastewater Treatment Plant Design, American Society of Civil Engineers and the Water Pollution Control Federation, 1977.

TABLE 6-3

PRELIMINARY DESIGN OF SCREENS

Number of Mechanical Screens (Phase I)	2
Manual Screen By-Pass Channel	1
Type of Screens	Aqua-Guard
Peak Design Flow	20.8 MGD
Channel Width	4 feet
Channel Depth	5 feet
Bar Spacing	1/4 inch



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REGIONAL WASTEWATER TREATMENT PLANT
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6009-42

FIGURE NO
6-1

The grit removal system will consist of two (2) "Forced Vortex" type grit collectors which will be 16 feet in diameter. Each grit collector will be capable of passing a maximum flow of 20.8 MGD of wastewater. The mechanism will be capable of removing 95 percent of the grit greater than 50 mesh in size and 65 percent of grit greater than 100 mesh in size.

The mechanisms will remove grit from the raw sewage by means of a variable pitch propeller powered by a drive tube. Grit-type pumps will remove the material from the storage hopper, and it will be dewatered in grit separators.

Figure 6-1 illustrates the grit removal devices, and Table 6-4 summarizes the pertinent design criteria. Two grit collectors will be provided initially, and one additional unit will be added for the Phase II plant expansion.

6.5 PRIMARY CLARIFIERS

The objective of primary treatment is the removal and disposal of the settleable organic solids contained in the wastewater stream. Primary clarifiers have the following advantages:

- o A decrease in quantity of secondary sludge produced.
- o A decrease in BOD loading to secondary treatment process units.
- o Resultant reduction in aeration tank volume, oxygen requirements, and solids thickening requirements.
- o Resultant energy conservation.

Class I reliability criteria require that there be a sufficient number of units of a size such that with the largest flow capacity unit out of service, the remaining units have a design flow capacity of at least 50 percent of the total design flow to that unit operation. Therefore, two (2) primary clarifiers will be provided with each rated at 50 percent of the design flow.

Table 6-5 summarizes the recommended design criteria for primary clarifiers from five separate design manuals. The design surface loading rates for this

TABLE 6-4

PRELIMINARY DESIGN OF GRIT COLLECTORS

Number of Units (Phase I)	2
Peak Design Flow	20.8 MGD
Type	Pista
Diameter	16 feet

TABLE 6-5

PRIMARY SEDIMENTATION TANK DESIGN CRITERIA

Design Parameter	Design Reference # 1	# 2	# 3	# 4	# 5	Proposed Design
Surface Loading, (gpd/ft^2)						
Average Flow	1000	----	1000	800-1200	800-1000	1000
Peak Flow	1500	1200	1500	2000-3000	----	1500
Depth, (ft.)	----	----	----	10-12	8-10	10
BOD Removal, (%)	30-35	----	30	30-35	30-35	30
SS Removal, (%)	----	----	50	50-60	50-60	50

report are 1,000 gpd/ft^2 and 1,500 gpd/ft^2 for average flow and peak flow conditions, respectively. BOD and SS removal will be assumed to be 30 and 50 percent, respectively.

Two (2) 80 feet in diameter clarifiers will be provided. Each clarifier shall be of the under-floor, center-column feed and concentric weir overflow type. A central drive mechanism shall support and rotate two attached rake arm assemblies. Blades attached to the bottom of the rake arms shall be designed to move sludge from the tank floor to a central sludge pocket where it shall be removed by pumping.

The primary waste sludge pumps will be heavy duty, self-priming, positive displacement, peristaltic type. The sludge feed pumps will be capable of delivering 60 gpm while operating at 45 rpm and will be Model No. SP/65 manufactured by the Waukesha Pump Division.

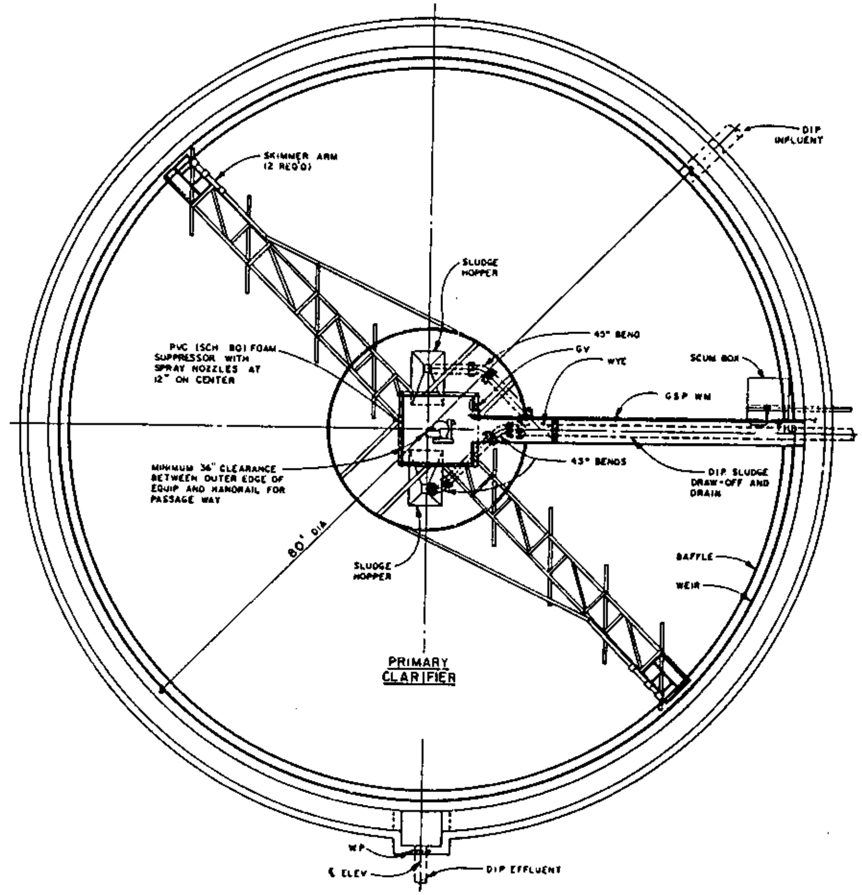
Due to the high odor potential of primary clarifiers and the close proximity of surrounding neighbors, the primary clarifiers will be covered using aluminum, geodesic covers. An odor control system will be used to treat odorous gases collected in the units.

Figure 6-2 illustrates a typical scraper-type primary clarifier. The proposed primary sludge clarifier design is summarized in Table 6-6. Two primary clarifiers will be constructed for the initial phase of the plant, and one additional clarifier will be provided for the Phase II expansion.

6.6 AERATION TANK AND AERATION EQUIPMENT

An activated sludge process cultivates large populations of bacteria and other microorganisms commonly found in the aquatic environment. The microorganisms utilize the colloidal and soluble organic material found in wastewater as a source of food and energy. The soluble material is thus converted to a form which can be readily separated from the liquid phase by sedimentation.

The conventional activated sludge process will be utilized as the basis of design. This process consists of aeration tanks, secondary clarifiers, and a



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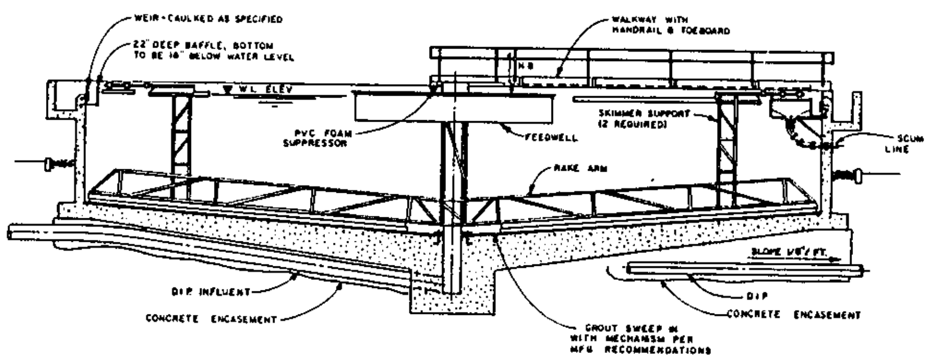

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PRIMARY CLARIFIER - PLAN & SECTION

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6009-42

FIGURE NO.

6-2

TABLE 6-6

PRELIMINARY DESIGN OF PRIMARY CLARIFIERS

Number of Units (Phase I)	2
Diameter	80 feet
Type	Circular, Upflow, Scraper Bottom
Sidewater Depth	10 ft
Surface Area, each	5026 ft^2
Surface Loading	
Average Flow	825 gpd/ft^2
Peak Day Flow	1403 gpd/ft^2
Settled Sludge Concentration	3-5 percent solids
Waste Sludge Pumps	2
Rating, each	60 gpm

sludge recycle line. The flow model is plug flow with influent settled wastewater and recycled sludge entering the head end of the aeration tanks. The influent wastewater and recycled sludge are mixed by the action of diffused or mechanical aeration. Adsorption, flocculation, and oxidation of the organic matter takes place while the mixed liquor flows down the length of the tank. The mixed liquor is subsequently settled in the final clarifiers, and settled sludge is returned typically at a rate of approximately 25 to 50 percent of the influent flow rate.

Class I reliability criteria do not require a backup basin; however, at least two equal basins must be provided. There shall be a sufficient number of mechanical aerators to enable the design oxygen transfer to be maintained with the largest capacity unit out of service. It is permissible for the backup unit to be an uninstalled unit, provided that the installed unit can be easily removed and replaced. However, at least two units must be installed.

Recommended design criteria for the conventional activated sludge system from five separate design manuals is summarized in Table 6-7. The aeration tank design criteria assumed for this design is a F/M ratio of 0.4 lbs BOD/lbs MLVSS-day. This F/M ratio correlates with an SRT of 5 days. The design MLSS is 2000 mg/l, and the design volumetric loading is 40 lbs BOD/1000 ft^3. An assumed tank depth of 12 feet and a hydraulic detention time of 6 hours will be utilized for sizing of the aeration tanks.

The method for transferring oxygen to the mixed liquor for the purpose of this report is by mechanical aerators. Pure oxygen aeration systems were not considered due to their complexity and labor intensive requirements. However, during the initial design phase, the diffused aeration alternative should be given further detailed evaluation.

Table 6-8 summarizes the design criteria for the mechanical aeration system. The oxygen input requirement for the removal of carbonaceous BOD was determined based upon the maximum day BOD loads. Due to the high wastewater temperature experienced in South Florida, an additional oxygen requirement of

TABLE 6-7

AERATION TANK DESIGN CRITERIA

Design Parameter	Design Reference # 1	# 2	# 3	# 4	# 5	Proposed Design
Volumetric Loading, (# BOD/1000 ft^3)	40	20-40	20-40	20-40	20-60	40
Detention, (hrs.)	----	4-8	4-8	6-8	4-8	6
F/M, (# BOD/# MLVSS-day)	0.2-0.5	0.2-0.4	0.2-0.4	0.2-0.4	0.15-0.4	0.4
SRT, (days)	----	5-15	5-15	5-15	4-8	5
MLSS, (mg/liter)	1000-3000	1500-3000	1500-3000	----	1500-4000	2000
Recycle, (%)	15-75	25-50	15-100	15-75	30-100	15-100
Depth, (ft.)	10-30	10-15	----	----	----	12

TABLE 6-8

MECHANICAL AERATION DESIGN CRITERIA

Standard Oxygen Transfer Efficiency	3.0 lb/hp-hr
Actual Oxygen Transfer Efficiency	1.94 lb/hp-hr
Alpha	0.90
Beta	0.95
T	$30^{o}C$
C_{sw}	7.5
C_L	2.0 ppm
Maximum Day BOD Load (Phase I)	17,660 lbs/day
Maximum Day NH_3-N Load (Phase I)	3,288 lbs/day

4.6 lbs O_2/lb ammonia-nitrogen oxidized was utilized for nitrification demands. A dissolved oxygen (D.O.) concentration of 2.0 mg/l was assumed for design in the aeration tanks.

The resultant peak oxygen requirement was 29,664 lbs/day. Based upon the design criteria in Table 6-8, a total horsepower of 650 is required. Therefore, the two (2) aeration basins would each include one (1) 125 and two (2) 100 horsepower aerators. Since the oxygen demand decreases as the waste moves along the tanks in a plug flow fashion, the 125 horsepower unit would be installed at the head end of the tanks.

Figure 6-3 illustrates a typical mechanical aeration tank, and Table 6-9 summarizes the preliminary design of the aeration tank and aeration equipment. Two (2) basins will be constructed and will include the installation of three (3) mechanical aerators in each basin for Phase I. One identical aeration basin will be added for the Phase II plant expansion.

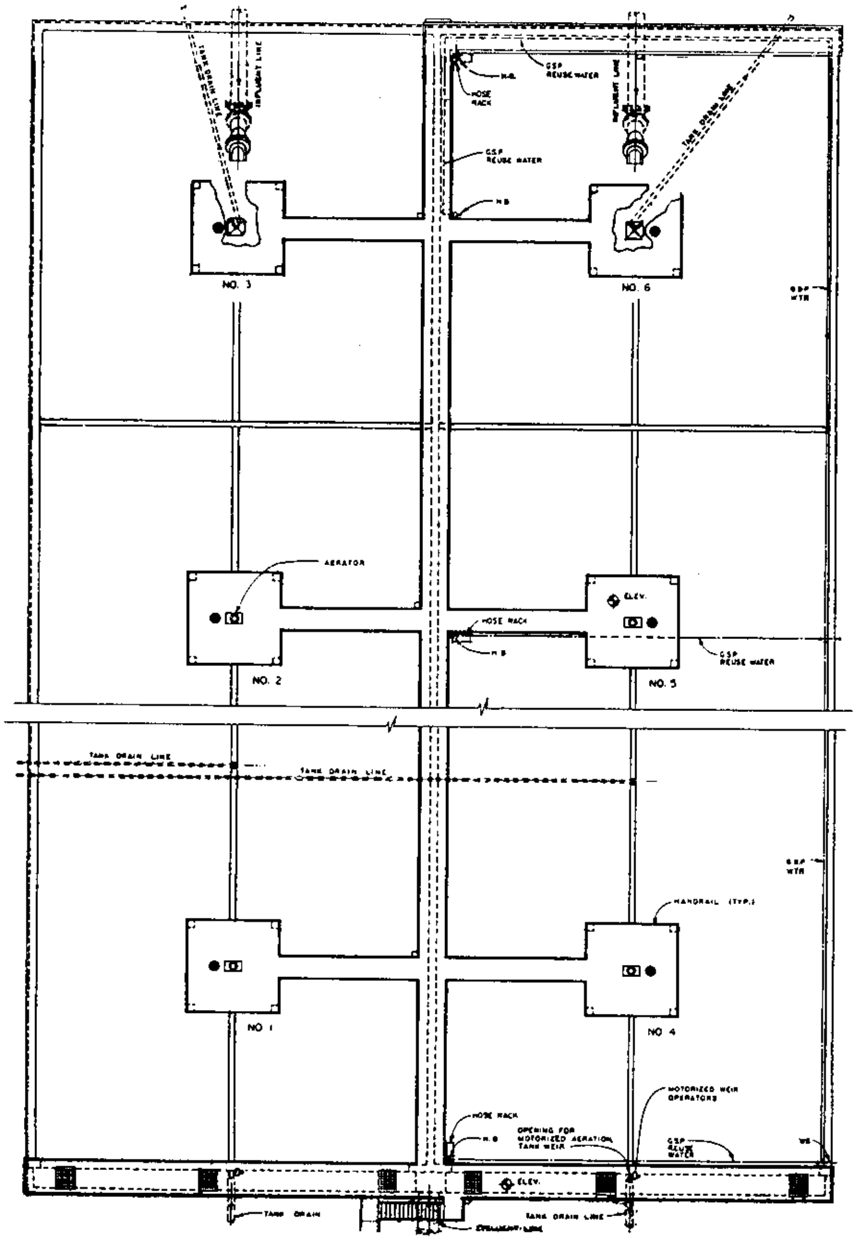
6.7 SECONDARY CLARIFIERS

Final sedimentation tanks are utilized to separate the settleable solids produced by the activated sludge process from the mixed liquor. Circular sedimentation tanks frequently are called clarifiers.

For secondary clarifiers to meet the Class I reliability criteria, a design flow capacity of at least 75 percent of the total design flow shall be met with the largest unit out of operation. Therefore, four (4) units shall be utilized in this design.

Table 6-10 summarizes the recommended design criteria for final sedimentation tanks from five design manuals. Secondary clarifiers are designed typically on the basis of hydraulic and solids loading rates. This design report will utilize hydraulic loadings of 600 and 1200 gpd/ft^2 and solids loadings of 15 and 40 lbs/day/ft^2 for average and peak conditions, respectively.

Four (4) seventy-five (75) feet in diameter sedimentation tanks will be provided for Phase I. Each clarifier shall be of the under-floor, center



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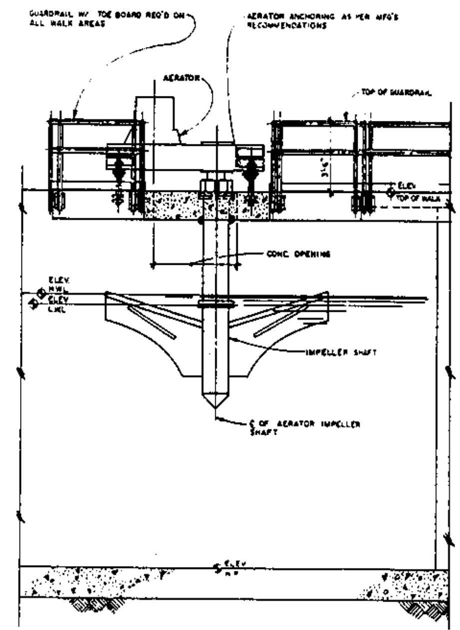

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REGIONAL WASTEWATER TREATMENT PLANT
FOR
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MECHANICAL AERATION TANK

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FIGURE NO.
6-3

TABLE 6-9

PRELIMINARY DESIGN OF AERATION TANKS AND AERATION EQUIPMENT

Number of Tanks (Phase I)	2
Size of Tanks, WxLxD	68 X 188 X 12 (ft)
Total Aeration Tank Volume	308,448 ft^3
F/M Ratio	0.4
SRT	4.9 days
MLSS	2000 mg/l
Volumetric Loading	37 lbs BOD/1000 ft^3
Detention	5.5 hrs.
Number of Aerators	6
Total Horsepower	650 hp
Available Oxygen	30,264 lbs/day

TABLE 6-10

FINAL SEDIMENTATION DESIGN CRITERIA

Design Parameter	Design Reference # 1	# 2	# 3	# 4	# 5	Proposed Design
Overflow Rates, (gpd/ft^2)						
Average	----	----	400-600	400-800	800	600
Peak	1200	1200	1000-1200	1000-1200	1600	1200
Solids Loading Rates, (#/day/ft^2)						
Average	----	14	8-30	20-30	----	15
Peak	50	30	40-50	<50	----	40
Depth, (ft) (71-100' diameters)	>12	>12	13	12-15	13	14
Weir Loading, (gpd/ft.)	15,000	10,000-30,000	----	----	10,000-30,000	22,000

column feed and peripheral overflow type. A central drive mechanism shall support and rotate four attached rake arm assemblies, which are complete with rapid sludge removal uptake pipes, rake blades, and squeegees. Two of the arms shall contain the sludge removal uptake pipes that shall remove the activated sludge to a collection box for return to the aeration tanks. The other two arms shall have raking blades and squeegees and shall rake sludge to a pocket in the center of the tank.

Return sludge pumps are needed to return settled activated sludge to the head of the aeration tanks. The rate of sludge return expressed as a percentage of the average plant design flow will be variable between 15 to 100 percent.

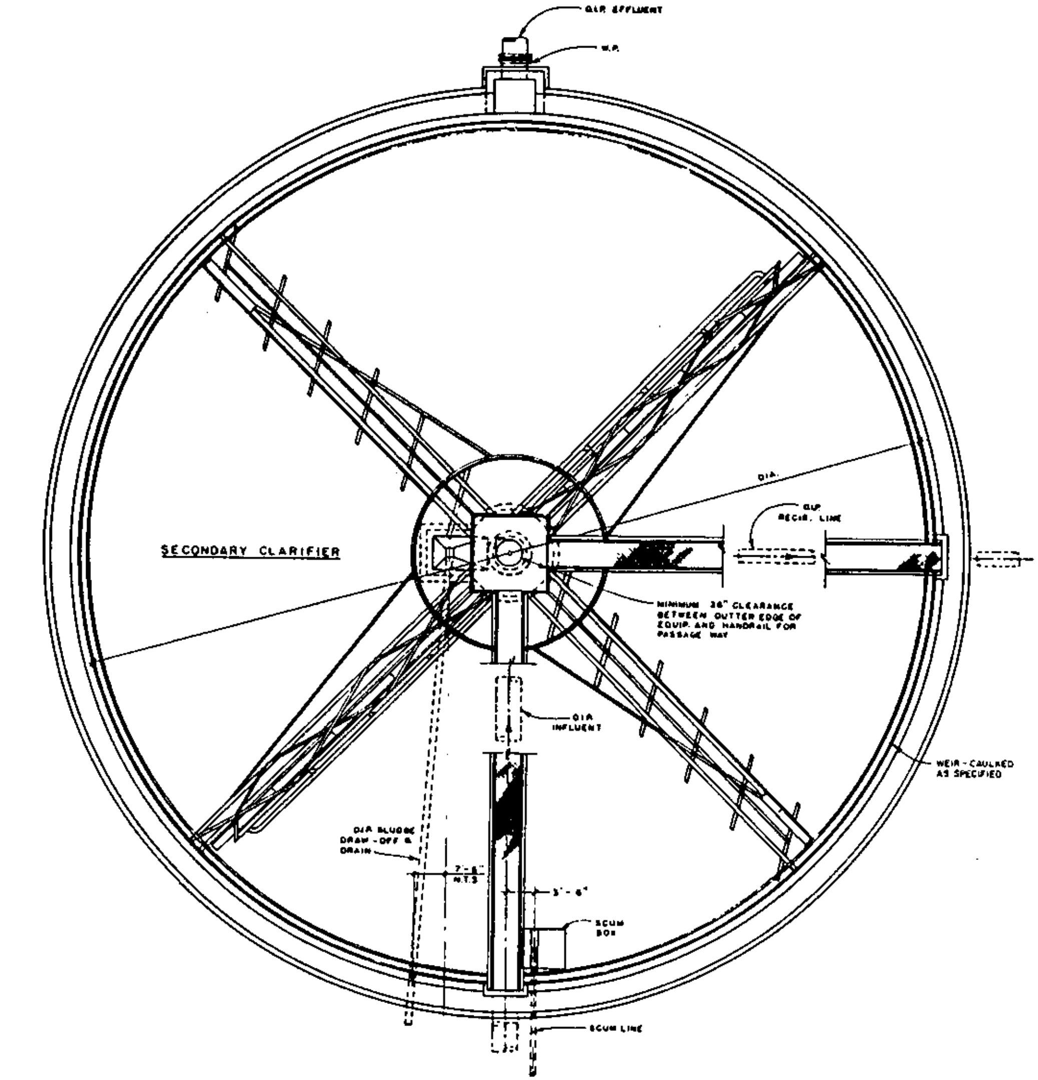
Three (3) horizontal, end-suction, volute-type, centrifugal, nonclog wastewater pumps will be provided for outside installation. Each pump will be rated at approximately 2900 gpm or a firm capacity of 8.3 MGD. The pumps will be driven by an eddy-current adjustable speed drive. The pump design is based upon Morris Pump Model No. 12 EC.

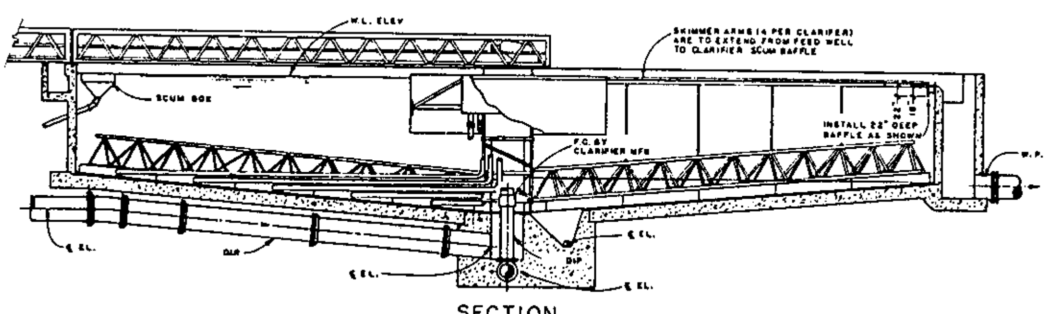
Figure 6-4 illustrates a typical rapid-sludge withdrawal secondary clarifier. The proposed secondary clarifier design is summarized in Table 6-11. Four secondary clarifiers will be constructed initially, with two additional units to be provided as a part of the Phase II plant expansion.

6.8 DISINFECTION

All wastewater treatment plants must be designed such that disinfection to protect the public is provided. Chapter 17-6 of the FDER requires a basic level of disinfection which shall result in not more than 200 fecal coliform values per 100 ml of effluent sample. Where chlorine is utilized for disinfection, maintenance of 0.5 mg/l minimum total chlorine residual after 15 minutes contact time at maximum daily flow or after 30 minutes contact time at average daily flow, whichever is greater, is accepted as evidence that the microbiological criteria will be met.

Class I reliability criteria require that with the largest flow capacity unit out of service, the remaining units shall have a design flow capacity of at





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REGIONAL WASTEWATER TREATMENT PLANT

FOR

CITY OF PLANTATION

SECONDARY CLARIFIER PLAN & SECTION

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DATE SEPTEMBER, 1983

6009-42

FIGURE NO.

6-4

TABLE 6-11

PRELIMINARY DESIGN OF SECONDARY CLARIFIERS

Number of Units (Phase I)	4
Type	Rapid Sludge Withdrawal
Diameter	75 ft.
Sidewater Depth	14
Effective Surface Area	15,831 ft^2
Average Hydraulic Overflow Rate	524 gpd/ft^2
Peak Hydraulic Overflow Rate	1,314 gpd/ft^2
Average Solids Loading Rate	12 $lbs/day\text{-}ft^2$
Peak Solids Loading Rate	33 $lbs/day\text{-}ft^2$
Number of Return Sludge Pumps	3
Pump Type	Centrifugal (nonclog)
Rating, each	2,900 gpm @ 20 ft.

least 50 percent of the total design flow. Therefore, two (2) equally sized basins will be provided for Phase I.

Chlorine contact tanks are designed utilizing the "around-the-bend" type flow pattern. Long, narrow sections with length: width ratios of 10:1 will inhibit short-circuiting within the tank. The chlorine contact tank for this design will have channels 5 feet wide by 50 feet long. To insure a detention time of 15 minutes at peak hour flow, six channels which are 10 feet deep will be required.

Figure 6-5 illustrates a typical chlorine contact tank and Table 6-12 summarizes the preliminary design of the chlorine contact tanks. Two tanks will be constructed initially, with one additional unit to be provided as part of the Phase II expansion.

The chlorination system will be sized for a total capacity of 25 mg/l at the design average flow. Therefore, the Phase I chlorination system will be rated at 1730 lbs/day. The system will utilize ton containers since the average daily demand is over 150 lbs.

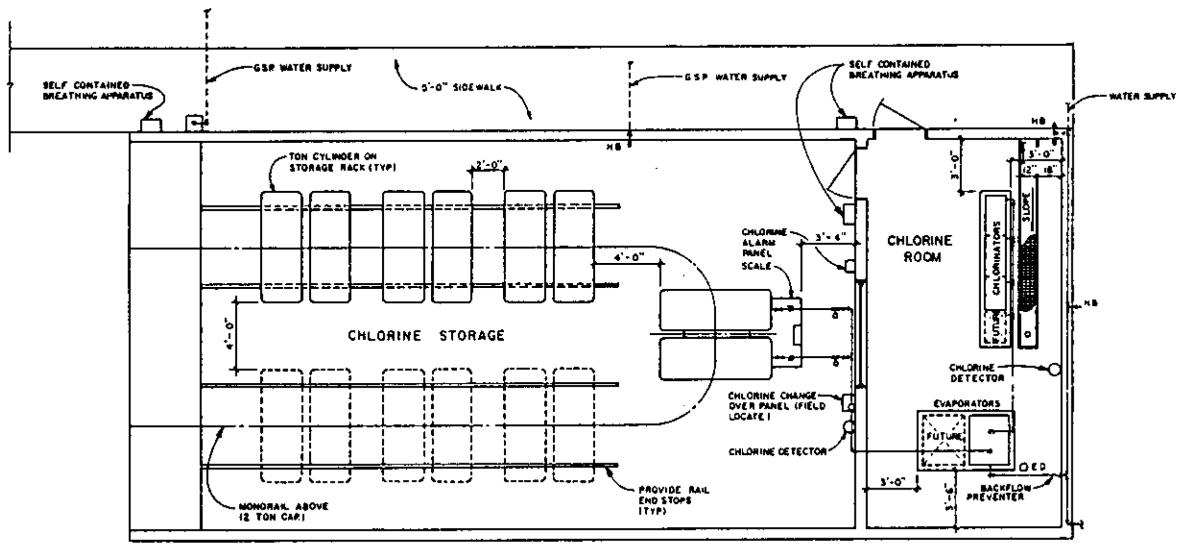
Figure 6-6 illustrates a typical chlorine building. The following major pieces of equipment will be included in the design:

- o Vacuum chlorinators (Pre-, Post-, and Standby)
- o Evaporators (On-line, Standby)
- o Automatic changeover system
- o Remote injectors
- o Annunciator alarm panel

The chlorinators will be of the high capacity, vacuum operated, solution feed type and shall automatically control chlorine gas feed in response to a 4-20 ma-DC flow signal proportional to influent flow. A manual adjustment shall be provided to regulate dosage of chlorine during the flow pacing operation.

CHLORINATION EQUIPMENT SCHEDULE

TON CONTAINER (BY OTHERS)
AUX TON CONTAINER VALVE
FLEXIBLE CONN.
HEADER VALVE
PRESSURE GAUGE (TYP.)
PRESSURE SWITCH
LIQUID LINE PRESSURE RELIEF SYSTEM
CHLORINE LIQUID HDR., SCH 80 SEAMLESS STEEL PIPE
LINE VALVE
EVAPORATOR GAS PRESSURE RELIEF SYSTEM
PVC VENT
CHLORINE PRESSURE REDUCING VALVE, A-784, ELEC
PVC VENT (TO ATMOSPHERE)
CHLORINE GAS HDR., SCH 80 SEAMLESS STEEL PIPE
CHLORINE GAS STRAINER
CHLORINE SCALE FOR TWO 1-TON CYL.
LOW WEIGHT ALARM SWITCH
SUPPORT
PVC BALL VALVE
PVC BALL VALVE
P.V.C VAPOR VENT PIPE
EVAPORATOR DRAIN AND OVERFLOW PIPING
GSP WATER LINE WITH UNION AND G.V.
GSP WATER LINE
LIQUID CHLORINE EVAPORATOR
CHLORINATOR (PRE-CHLORINE)
CHLORINATOR (POST-CHLORINE)
CHLORINATOR (WELL)
MOTOR ACTUATED BALL VALVE
PVC CHLORINE GAS VACUUM LINE
PVC CHLORINE GAS VACUUM LINE
PVC SENSING LINE
CHLORINE LEAK DETECTOR
CHLORINE CHANGEOVER PANEL



CHEMICAL BUILDING PLAN
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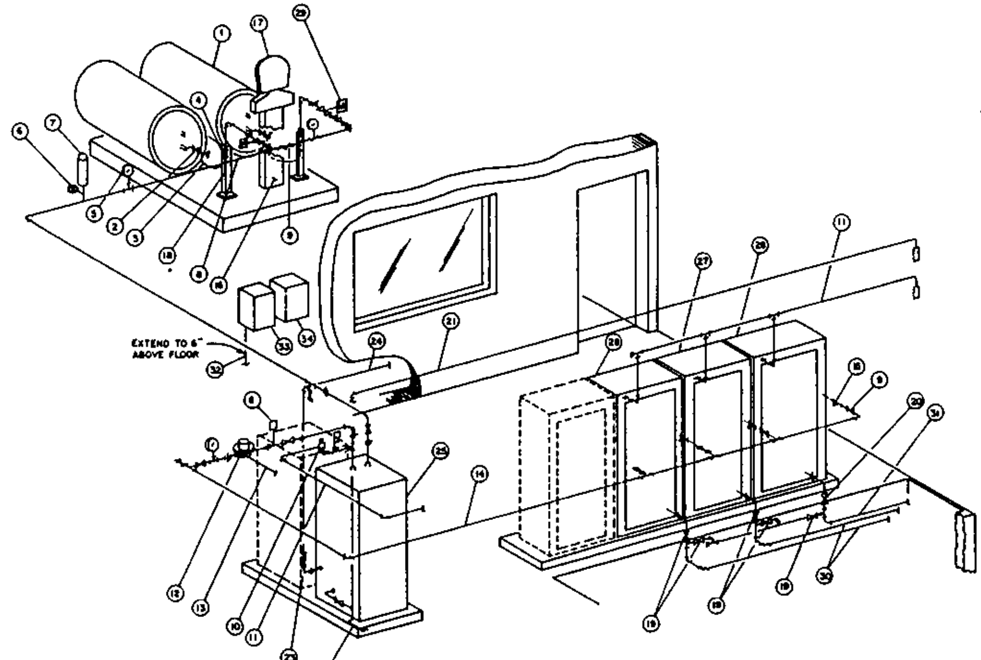

CHLORINATION EQUIPMENT
NO SCALE

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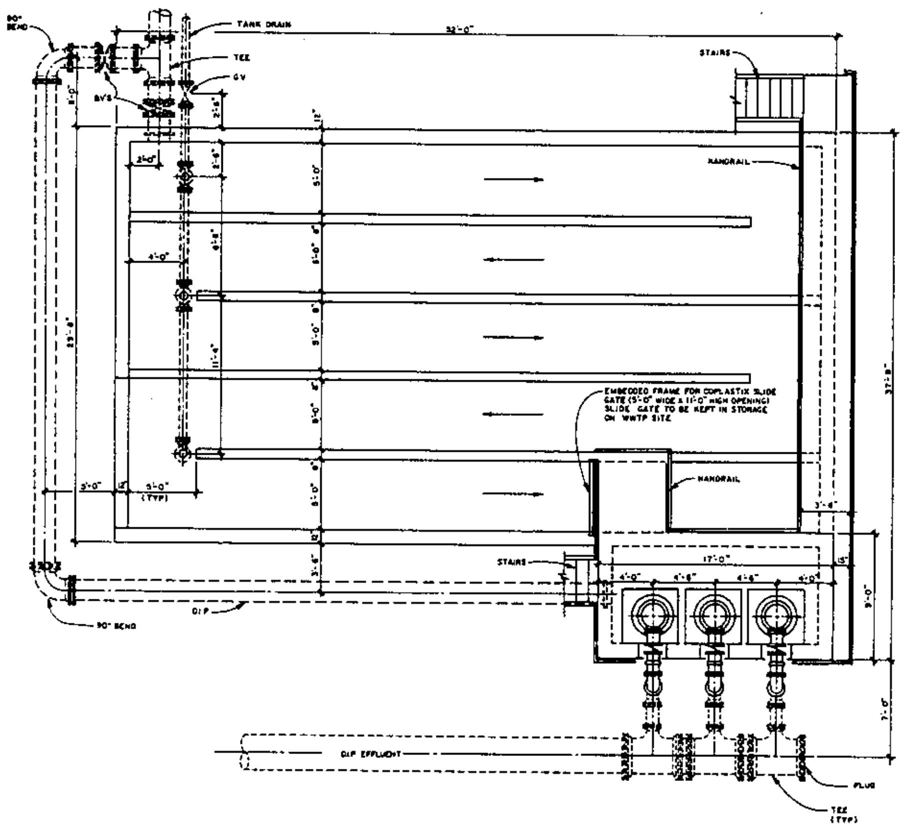
TABLE 6-12

PRELIMINARY DESIGN OF CHLORINE CONTACT TANK

Number of Tanks (Phase I)	2
Size, W x L x D	30 X 50 X 10 (feet)
Channel Width	5 feet
Length: Width Ratio	10:1
Detention Time	
@ Peak Flow	15 minutes



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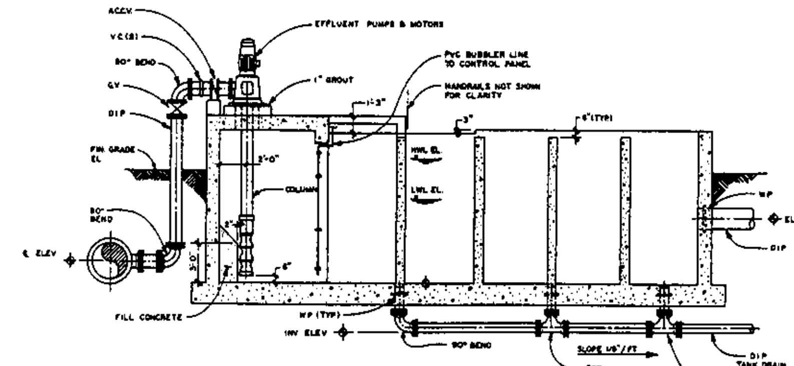

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FOR
CITY OF PLANTATION
CHLORINE CONTACT TANK & EFFLUENT P.S.

FT. LAUDERDALE OFFICE PROJECT NO.
6009-42

FIGURE NO.
6-5

The evaporators will be electrically loaded and thermostatically controlled. Liquid chlorine will be automatically vaporized at a rate controlled by system demand.

An automatic changeover system for liquid chlorine will be supplied to transfer from an exhausted source of chlorine to a standby source without interruption of chlorination. The control panel will include status lights, local alarm, acknowledge button, and contacts for remote alarm.

Remote injectors will be provided and sized for the chlorine feed rate and water pressure to provide operating vacuum for a solution feed, vacuum type gas dispenser. The injectors will normally shut off automatically. A solenoid valve will be provided to start/stop the injectors remotely.

The alarm panel shall include alarm lights, horn, silencer button, and contacts for remote indication. The following alarms will be provided:

- Low chlorine container weight
- High pressure in liquid chlorine manifold
- High/low evaporator temperature
- Low evaporator water bath level
- High evaporator gas pressure
- Low chlorinator vacuum
- Chlorine leak detectors

6.9 EFFLUENT PUMPING EQUIPMENT

The effluent disposal system consisting of two deep wells is described in detail in Section 8. The first phase (10 MGD) will include one 24-inch diameter injection well and the second phase (15 MGD) will include an additional 24-inch injection well which will be in operation by 1991.

The design of the effluent disposal system will be based upon the maximum 24-hour flow. Consequently, as developed in Section 3, the maximum 24-hour event expected for the first phase is 14.1 MGD and 21.3 MGD for the second phase.

Equalization Tanks

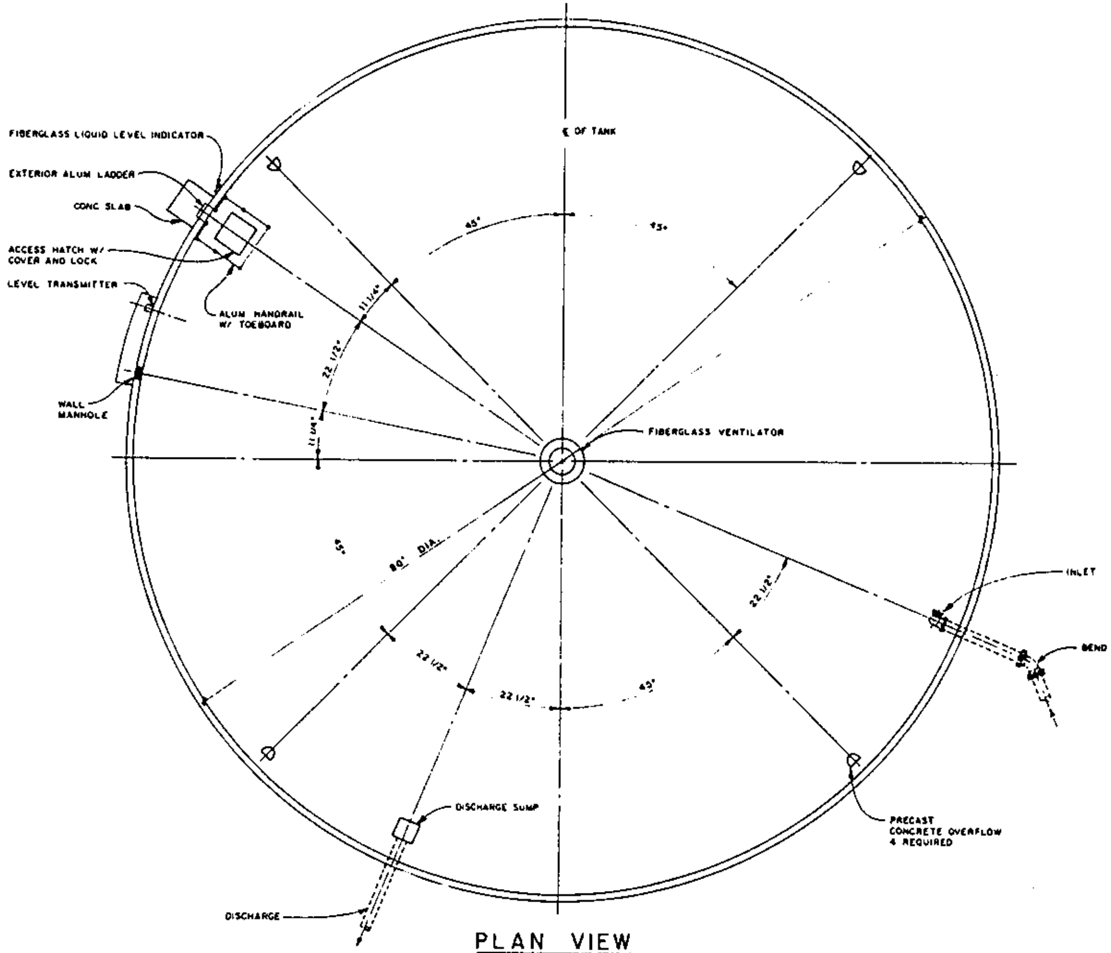
Storage tank volume must be provided to accumulate flows above the maximum day flow rate. This volume is normally equated to 10 to 20 percent of the average daily dry weather flow. Therefore, an assumed total storage of fifteen percent of the average annual flow for the 15 MGD phase must be provided. Two (2) one million gallon storage tanks will be included with one tank being constructed in each phase.

The storage tanks will be 90 feet in diameter and will have a side water depth of 21' - 0". This combination of dimensions is the most economical in terms of construction cost for this size structure. The tank will be a prestressed composite concrete structure as manufactured by the Crom Corporation. The tank will consist of a concrete foundation and floor slab, shotcrete core walls with a steel diaphragm, pneumatic mortar, and a prestressed concrete dome. Figure 6-7 illustrates a typical equalization tank.

The equalization basins will be designed as side-line units. In this type of design, only that amount of flow above the maximum daily average is diverted to the equalization basin. This scheme minimizes pumping requirements.

Transfer Pumping System

Transfer pumps will be mounted on the chlorine contact tank as shown on Figure 6-5. A four-pump design, three operating plus one standby with all pumps being identical, will pump the treated effluent to the storage tanks. The transfer pump system will consist of single-stage, above-base discharge, vertical turbine pumps. The pumping units have been selected based upon curves presented by Byron Jackson Pump Division. The pumping unit used as a standard of quality for design is Model 12" - HQR-H operating at 1770 rpm. Each pump is rated at 2300 gpm at 25 feet. Constant speed motors to be utilized for the transfer pumps will be rated at 25 horsepower and 1770 rpm. Figure 6-8 illustrates graphically the system design. Three of the four transfer pumps will be supplied under the Phase I construction program to meet system demands.



PLAN VIEW

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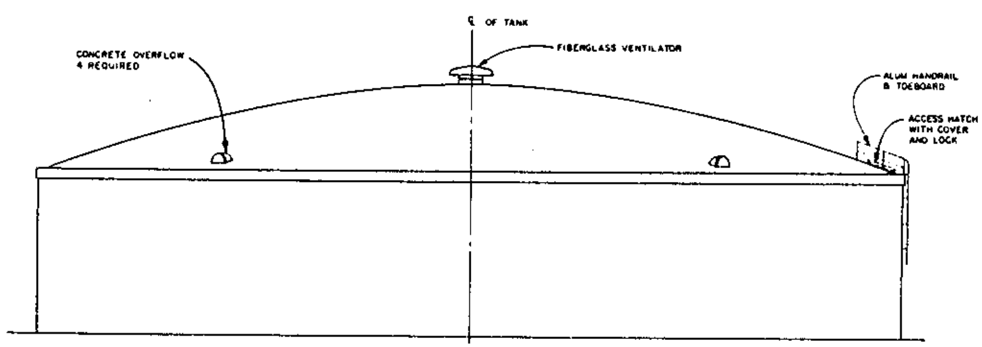

ELEVATION

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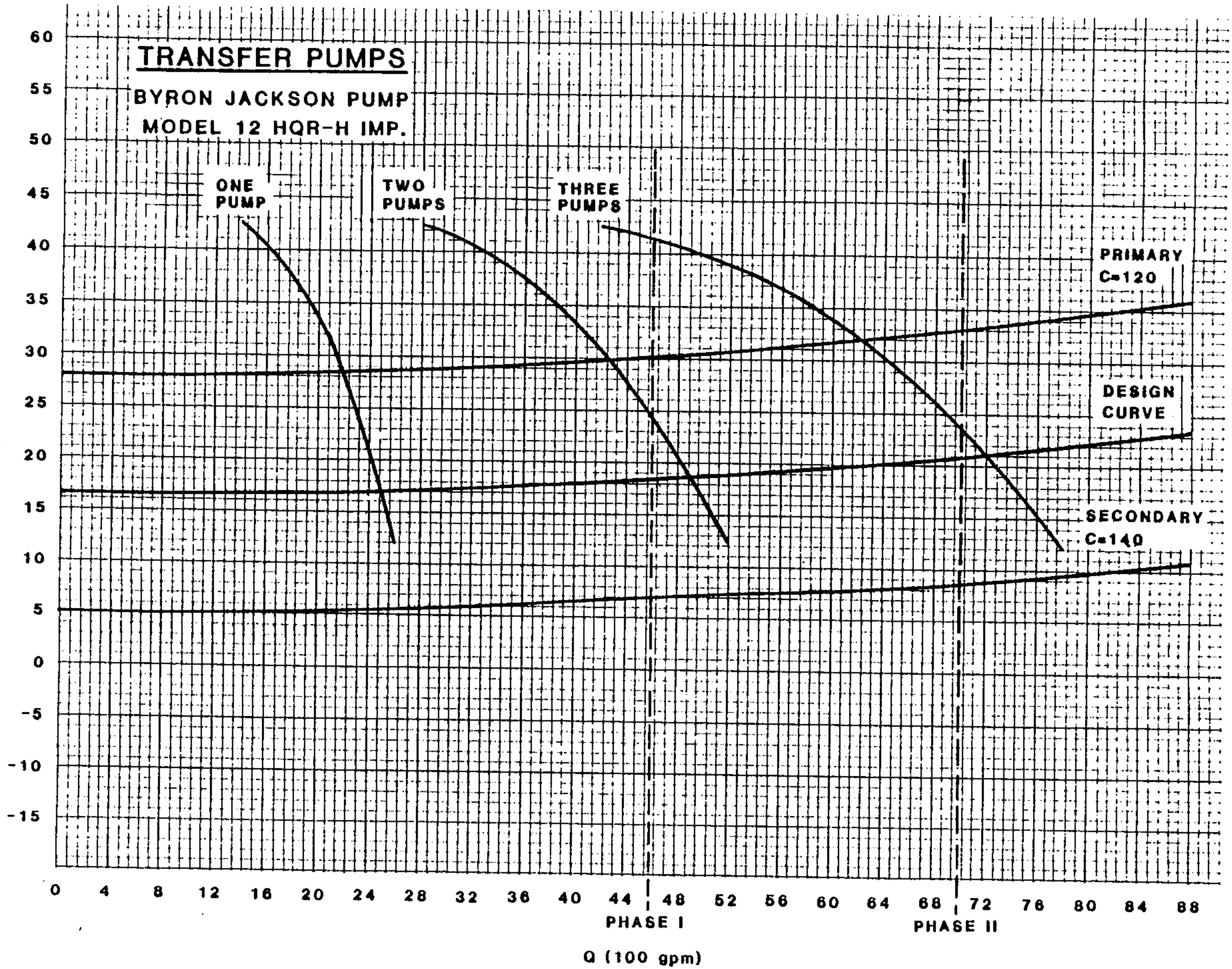

FIGURE 6-8

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Effluent Pumping System

The effluent pumping system is based upon the following three main design operating conditions:

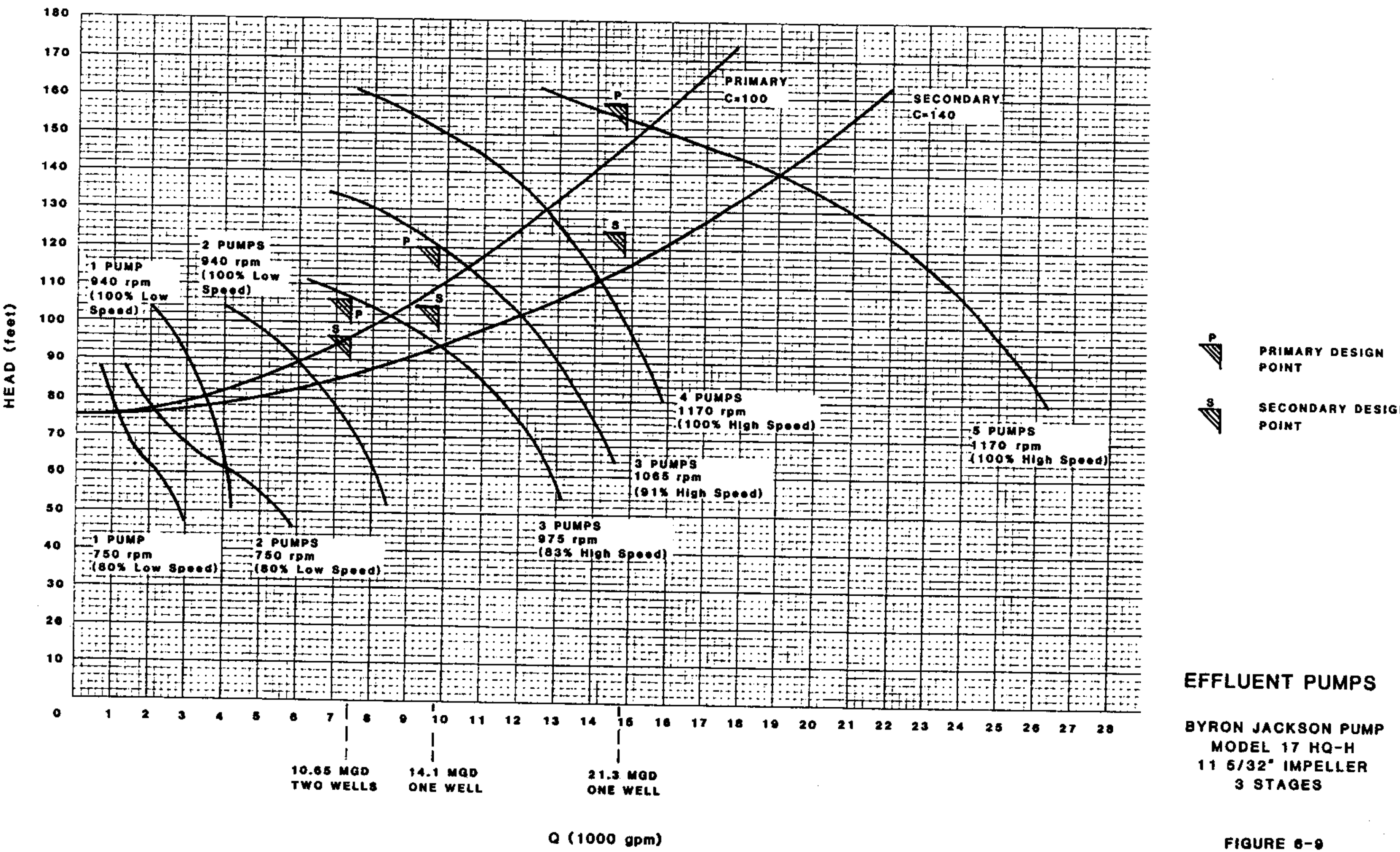
- 10 MGD Phase with one deep well in service.
- 15 MGD Phase with two deep wells in service.
- 15 MGD Phase with one deep well in service and one deep well out of service.

In order to achieve these widely ranging operating conditions, a two-speed, variable-speed pumping system is proposed. This operating scheme will ensure reduced capital and operating costs.

The effluent pumps will be mounted on the chlorine contact tank as shown on Figure 6-5. A six-pump design with five operating and one standby pump will meet the maximum day demand. The effluent pumps will be three-stage, above-base discharge, vertical turbine pumps. The pumping units have been selected based upon curves presented by the Byron Jackson Pump Division. The pump unit used as a standard of quality for design is Model 17 HQ-H. Each pump is rated at 3000 gpm at 154 feet at a full speed of 1170 rpm.

Figure 6-9 illustrates graphically the effluent pump system design. The first design condition of 21.3 MGD with two wells in service is met with three pumps on-line at 975 rpm, which is 83 percent of the high speed of 1170 rpm. The second design condition of 14.1 MGD with one well in service is met with three pumps on-line at 1065 rpm, which is 91 percent of the high speed of 1170 rpm. The third design condition of 21.3 MGD with one well in service is met with five pumps on-line at 100 percent of the high speed of 1170 rpm. Flows below approximately 6000 gpm may be served by one or two pumps operating at a percentage of the full low speed of 940 rpm. Four pumps plus one standby will be supplied under the Phase I construction program to meet system demands. One additional pump will be supplied during the Phase II plant expansion.

The motors will be two-speed squirrel cage induction motors rated at 200 horse-power each. The type of variable-speed drive used for the effluent pumps will



EFFLUENT PUMPS

BYRON JACKSON PUMP
MODEL 17 HQ-H
11 5/32" IMPELLER
3 STAGES

FIGURE 6-9

CDM.

be the "magnetic clutch" or "eddy current" coupling. The selection of this type of drive was based upon its comparatively low first cost, efficiency, ease of maintenance, and low maintenance costs. The drives will be sized for maximum efficiency by a minimum slip loss of 2-3 percent of full speed. The lower speed, 940 rpm, is approximately 80 percent of the high speed of 1170 rpm. The general design concept is not to allow the slip speed (the difference between the drive input speed and the drive output or pump speed) to exceed 25 percent of the drive input speed. This concept is based upon the fact that the maximum energy loss due to slip occurs at the two-thirds point or 67 percent of full speed. Significant energy savings can be realized if the minimum pump speed is maintained above the 67 percent speed point. As can be seen on Figure 6-9, the pumps in this design in combination with magnetic drives utilizing two-speed motors, would operate at speeds greater than 75 percent of each motor speed.

Fine Screening of Effluent

Fine screening of the effluent is contemplated for deep well disposal. Screening of occasional carry-over is recommended as a preventive measure against clogging and blinding of the geologic formation.

Automatic, self-cleaning pipeline strainers will be utilized as the basis of design. Model No. 593 as manufactured by Zurn will be specified in the 24-inch size. The self-cleaning strainer is basically a multi-basket strainer with an integral backwash system. Normal flow is continuous, and a portion of the liquid being strained is used to carry the concentrated debris away from the strainer.

Surge Protection

As developed in detail in Section 8.6, adequate surge or water hammer protection must be incorporated into the design of the effluent pumping system. Surge relief valves will be provided of the sewage surge relief elbow type. The valve will be based on the G.A. Industries Model No. 200C. The valve shall be normally closed and shall open when the system pressure exceeds the pump shut-off pressure by 10 percent. As pressure returns to normal, the valve shall close slowly. The discharge from the valve shall empty into the chlorine contact tank.

SECTION 7
SOLIDS TREATMENT FACILITIES

7.1 INTRODUCTION

Regulatory control of wastewater treatment residuals exists on the Federal, State and local levels. Existing regulations have evolved through the years to the current position that encourages resource recovery of wastewater solids.

The FDER has issued draft regulations that will govern the treatment and disposal of sludge. The draft regulations will be incorporated into Chapter 17-7 of the Rules of the FDER. The regulations propose a grading concept. The present predraft regulations classify sludge as Grade I, II, or III depending on its chemical and physical constituents. The proposed grading criteria are shown in Table 7-1. Based upon the proposed criteria and the expected characteristics of the service area sludge, wastewater solids are expected to fall under either a Class I or Class II FDER classification.

This section describes, evaluates, and designs the sludge treatment process units. The section is divided into the following subdivisions:

- 7.2 Sludge Design Criteria
- 7.3 Alternative Analysis
- 7.4 Centrifuge Thickening
- 7.5 Anaerobic Digestion
- 7.6 Belt Filter Press Dewatering

Based upon the design criteria, two alternatives for sludge treatment were evaluated on a present worth economic basis. A preliminary design, equipment selection, and layout of process units were developed for the preferred sludge treatment alternative.

TABLE 7-1

SLUDGE GRADING CRITERIA

Parameter	SLUDGE GRADE I	SLUDGE GRADE II	SLUDGE GRADE III
1. Chemical Criteria			
- Cadmium, (mg/kg)	≤ 30	≤ 100	≥ 100
- Copper, (mg/kg)	≤ 900	≤ 3,000	≥ 3,000
- Lead, (mg/kg)	≤ 1,000	≤ 1,500	≥ 1,500
- Nickel, (mg/kg)	≤ 100	≤ 500	≥ 500
- Zinc, (mg/kg)	≤ 1,800	≤ 10,000	≥ 10,000
2. Physical and Pathogenic Criteria			
- Stabilization	Required	Required	Not Required
- Dewatering	Not Required	Not Required	Required for Sludge
- Disinfection	Required for processed and composted domestic sludge	Not Required	Not Required

Source: Predraft No. 9 (January 11, 1983) Part IV FAC 17-7.53 (3).

7.2 SLUDGE DESIGN CRITERIA

An estimate of the quantities of wastewater solids has been developed in order to evaluate alternatives and to size process units for the preliminary design.

Solids production estimates presented are based on criteria discussed herein and criteria listed in previous sections. Future solids quantities were estimated incorporating existing data and trends. Sludge production quantities are reported on an average day basis and on a maximum 30-day basis. The latter is used as design criteria for sizing sludge treatment and disposal processes.

Estimated sludge production was based upon the following criteria:

- Projected flow, BOD, and SS loads presented in Section 3.
- Effluent quality based on providing a minimum of secondary treatment for deep well disposal.
- Primary sludge production is based on 50 percent removal of suspended solids.
- Secondary sludge production is based on 30 percent removal of BOD in the primary clarifiers.
- Secondary sludge production is based on 0.68 pounds of total waste solids per pound of BOD removed for the activated sludge process.
- Volatile solids content of the sludge is based on 65 percent for primary sludge and 75 percent for waste activated sludge.

Table 7-2 summarizes the sludge production estimates. The average day sludge productions are utilized in developing projected operation and maintenance costs. As noted earlier in this section, the maximum month figures are utilized in sizing process equipment. Therefore, the design sludge quantities are 18,200 lbs/day and 28,200 lbs/day for the Phase I and Phase II facilities.

TABLE 7-2

SLUDGE PRODUCTION

	Phase I (10 MGD)	Phase II (15 MGD)
Average Day, lbs/day (tons/day)		
Primary	7,400 (3.7)	11,500 (5.75)
Waste-Activated	6,000 (3.0)	9,300 (4.65)
Total	13,400 (6.7)	20,800 (10.4)
Maximum Month, lbs/day (tons/day)		
Primary	10,400 (5.2)	16,100 (8.05)
Waste-Activated	7,800 (3.9)	12,100 (6.05)
Total	18,200 (9.1)	28,200 (14.1)

7.3 ALTERNATIVE ANALYSIS

Several options for the ultimate disposal of sludge were considered. These options included:

- o Landfilling
- o Composting
- o Land Application

Landfilling

The potential to landfill dewatered sludge exists at the Broward County North District Landfill. The Broward County Water and Wastewater Division will own and operate a landfill that will accept stabilized dewatered sludge from the North District Regional Wastewater Treatment Plant (NDRWWTP). The landfill is presently projected to be in operation during the last quarter of 1984. It is designed for twenty years of operation and is intended to serve all the Large Users of the NDRWWTP.

Advantages associated with using the NDRWWTP landfill are as follows:

- o Introduction of the Plantation Region in the user base would decrease the unit cost of disposal to users.
- o The landfill site has been purchased; therefore, no land purchase and/or site acquisition would be needed.
- o An environmental assessment of the landfill operation has been conducted. Adverse impacts from this operation are considered minimal.

Disadvantages of including the Plantation Region as a user of the North Region landfill are:

- o Landfill life would be shortened.
- o The landfill location is in excess of thirteen miles from the Plantation North Wastewater Treatment Plant. Hauling costs will be high due to the significant distance.

- Implementation of this alternative is possible but will be difficult in light of the fact that landfill life is decreased to other users.

Costs used for estimating landfilling costs were obtained from the May 1980 "Solid Residuals Disposal Phase I Design" report prepared by CDM. In that report a unit cost of $15.30 per wet ton of sludge at 40 percent solids was calculated for the combined capital and O&M cost. Escalating this unit cost by the ENR Construction Cost Index of 4,000 and converting to a dry weight basis results in an updated unit cost of $48.74 per dry ton.

Composting

Composting is an age-old degradation process in which aerobic thermophilic microorganisms reduce organic matter (i.e., volatile wastewater solids) to humus-like consistency. This process may be employed on primary solids, waste biological solids, or mixtures of primary and waste biological solids, either digested or undigested. The composting process rapidly generates excess heat, raising the temperature of the material to 55 to 80^{o}C (130 to 175^{o}F). The heat hastens the rate of decomposition, evaporates moisture, and effectively destroys or inactivates most pathogenic microorganisms and parasites. This stabilization of the wastewater solids not only reduces pathogens but also produces an organic material which can be utilized as a soil conditioner for land application in both urban and rural locations.

In general, composting by any of the available methods consists of the following sequential steps:

- Mixing - Dewatered solids (typically 80 percent water) are mixed with a drier bulking material such as wood chips, sawdust, previously composted material, or shredded and sorted municipal refuse to obtain an optimum moisture content (approximately 60 percent) and to facilitate ventilation of the mixture.
- Composting or decomposition - The mixture is placed in piles or in mechanical reactors and allowed to decompose. The rate of decomposition of the mixture and the effectiveness of the pathogen kill may be maximized by optimizing the oxygen, nitrogen, organic material,

temperature, moisture content, and bacteria present. Aerobic conditions, between 5 and 15 percent oxygen, can be maintained throughout the mixture by forced aeration or mechanical agitation. Maintenance of high temperatures (greater than 55°C) can be assured by proper pile geometry or mixture enclosures. Initial moisture content can be optimized in the preceding mixing step.

- Curing - The composted material is allowed to stand in piles to ensure complete stabilization (decomposition, pathogen kill, etc.), and to allow for drying of the material.
- Screening - Recyclable bulking materials and/or unwanted refuse are removed from the cured solids prior to storage and/or distribution of the final composted product.
- Storing - The product may be stored awaiting distribution.

The length of time required for the complete composting process depends upon the composting method utilized, the climatic conditions, and the characteristics of the wastewater solids being composted.

Composted wastewater solids are an excellent source of organic matter and thus an excellent soil conditioner. The addition of composted materials to soils will improve soil physical properties by increasing water content and retention, enhancing aggregation, increasing soil aeration, improving permeability, increasing infiltration, and reducing surface crusting. Addition of compost to sandy soils will increase the soil's ability to retain water. In heavy-textured clay soils, the added organic matter will increase permeability to water and air, and increase water infiltration into the soil profile, thereby minimizing surface runoff. In turn, these soils will have a greater water storage capacity to be utilized for plant growth. Addition of compost to clay soils has also been shown to reduce compaction (i.e., lower the bulk density) and increase the rooting depth.[1]

[1] Epstein, E., "Composting Sewage Sludge at Beltsville, Maryland," not dated.

Concerns often arise when sludge composting is discussed with relation to the occupational and health aspects of microorganisms associated with the process. Both primary and secondary pathogens are present in wastewater sludge and in the sludge composting process. Primary pathogens are microorganisms that can initiate an infection in apparently healthy individuals. Secondary pathogens typically cause disease in humans with a compromised health condition. In addition, some microorganisms are capable of causing allergic reactions which may lead to chronic disease.

One of the most extensively studied secondary pathogens associated with composting is Apergillus fumigatus, a fungus which grows at high temperatures (over 45°C) and uses decaying organic matter as a food supply. This microorganism is present in the general environment from such sources as household dust or potting soils and mulches. Since Aspergillus fumigatus has been found in relatively high concentrations in the composting process, it is an organism of some health concern. The microorganism is capable of causing an allergic reaction, characterized by difficulty in breathing, nausea, and fever beginning 4 to 6 hours after exposure, or it can act as a secondary pathogen which causes apergillosis, an invasive lung disease. However, an infection caused by Aspergillus fumigatus is generally concomitant with the presence of a debilitating disease such as emphysema, diabetes, asthma, leukemia, or tuberculosis. In addition, certain common therapeutic measures, such as immunosuppressive drugs, corticosteroids, and radiation therapy may increase the risk of aspergillosis.

In outdoor operations, spores disperse readily into the ambient air. Indications are that exposure has to occur in enclosed spaces in order to concentrate the spores to a level which will provoke reaction in the small proportion of the population which is susceptible.[2]

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[2] E&A Environmental Consultants, Inc. "Newsletter," April - June 1983, Volume 3, Number 2.

The three alternative methods of composting of wastewater sludge are the following:

- o Windrow
- o Static Pile
- o Mechanical

Only the mechanical reactor method of composting has been evaluated economically, because of space limitations at the proposed plant site and because the windrow method has proven to be a poor neighbor due to odors. The mechanical reactor is a suitable option if there is a need to house the entire composting operation, or if space limitations are evident.

The Taulman-Weiss system is a proprietary system which uses enclosed vessels for composting of sludge. Generally, three reactors are used. The Bio-reactor is used for composting the sludge. The compost is held in the Bio-reactor from twelve to fourteen days. From there it is cured in a separate reactor. A carbonaceous material storage silo is also provided to provide controlled storage of the bulking material.

The necessary oxygen for the composting process is provided by using air supply blowers and exhaust fans. The oxygen concentration of the process is controlled to ensure optimum conditions. Also monitored are temperature and moisture content. Exhaust air is released underwater in the aeration tank to remove microorganisms such as Aspergillus fumigatus from the air.

The Bio-reactor contains a screw conveyor at the bottom of the tank that rotates around the tank's axis. This acts as an outfeed device for the compost. The system is designed for top feeding of dewatered sludge. As the sludge moves in a plug flow manner down towards the outfeed device, it achieves the proper environment to render a stable humus-like by-product.

The Taulman-Weiss system is totally enclosed and quite compact compared to the open air composting system. The operation and maintenance costs of the system are reasonable due to the decrease in labor requirements for operation of an essentially automated system. Capital costs, however, are higher than the conventional static pile method. Because of the limited area available at some sites, this system offers an attractive alternative to open air composting.

Land Application

Land application of liquid wastewater sludge was initially considered for ultimate disposal of sludge but ruled out for further consideration as a long-term disposal option. Several reasons contributed to this decision, which include the following:,

- o Land application is a land intensive operation. Due to land development pressure, land purchase would be costly.
- o Open tracts of land are located in the western sectors of the study area. Only sparse areas have suitable characteristics for land application. Costly site preparation and management will be needed in most of the available area.

There are many private contractors which haul sludge to permitted sites. Liquid sludge (<10% solids) is more readily accepted because it can be easily applied to the land, although dewatered sludge is accepted by some. Use of this method of disposal relieves a municipality of having to find a disposal site, construct facilities for ultimate disposal or utilization, and operate and manage the facilities.

A major disadvantage is the lack of assurance associated with the operation. Interruption of service due to any possible reason will renew the problem of what to do with the large quantities of sludge generated each day. Negotiation of long-term contracts, including a performance bond, will eliminate some of the risk associated with this operation, but some contingency planning should be stressed if this option is used.

Cost-Effective Analysis

Based upon the above discussion, two alternative sludge treatment and disposal trains were evaluated on a life-cycle cost basis. A present worth analysis of the capital and operation costs was conducted to determine the more cost-effective process train. Cost information was obtained from the Environmental Protection Agency's (EPA's) publications entitled Innovative and Alternative Technology Assessment Manual and Process Design Manual: Sludge Treatment and Disposal. Costs associated with landfill disposal are presented herein. The capital and O&M costs for the composting system were obtained from the manufacturer. All costs were updated by using an Engineering News Record (ENR) Construction Cost Index of 4000. It was assumed that Phase I would be constructed in 1986. Phase II costs and annual operation and maintenance (O&M) costs were converted to 1986 dollars. The two alternatives that were evaluated are outlined below.

The "conventional" treatment alternative consisted of the following unit processes:

- Centrifuge Thickening of Waste Activated Sludge
- Anaerobic Digestion
- Belt-press Dewatering (Digested)
- Landfill

The "innovative" alternative consisted of the following processes:

- Centrifuge Thickening of Waste Activated Sludge
- Belt-press Dewatering (Raw)
- Mechanical Composting

Table 7-3 lists the unit processes for the two alternatives and their associated costs. The present worth of O&M expenses of the processes during the planning period is included as a separate item. Table 7-3 indicates that the cost of the innovative alternative is approximately 13 percent greater than that of conventional sludge treatment and disposal.

Evidence exists to show that the final compost product has a market value. The success of a marketing program is contingent upon the ability of the compost product to enter existing channels of soil conditioner distribution and to establish new and creative channels of distribution based on marketing

TABLE 7-3

PROCESS COST-EFFECTIVENESS ANALYSIS

(Present Worth Dollars)

Unit Process	Conventional	Innovative
Centrifuge Thickening	$1,075,981	$1,075,981
Anaerobic Digestion	1,848,360	---
Belt-press Dewatering	468,157	716,381
Landfill Disposal	224,580	---
Mechanical Composting	---	3,648,810
Operation and Maintenance	2,809,024	1,812,878
Total	$6,426,102	$7,254,050

principles. A comprehensive marketing pilot program is needed to generally introduce compost to the public in a metropolitan area and to further evaluate price elasticity, market potential, seasonality of demand, and marketing strategies.

Based upon the cost-effectiveness analysis presented in Table 7-3, a credit of $17.86 per wet ton of product would be required for the innovative alternative to be cost-effective. Based upon recent market surveys conducted by others, this value of credit for the compost product could be obtained; therefore, the innovative alternative should be given further detailed evaluation if this regional plant proceeds to detailed design. Additionally, if the City cannot obtain an executed agreement for disposal of sludge in the Broward County landfill at a reasonable tipping fee, then the composting facilities would be investigated further. However, due to the uncertainties, the basis of design for this report shall be the conventional sludge treatment and disposal alternative.

7.4 CENTRIFUGE THICKENING

Sludges are thickened primarily to decrease the capital and operating costs of subsequent sludge processing steps by substantially reducing the volume. Thickening from one to two percent solids concentration halves the sludge volume. Further concentration to five percent solids reduces the volume to one-fifth its original volume.

The waste sludge from the primary clarifiers is expected to have a solids concentration of 3-5 percent. Consequently, there is no need to thicken, and this waste primary sludge will be fed directly to the primary digesters. However, the waste-activated sludge is expected to range from 0.5 to 1.0 percent solids concentration. Therefore, the waste-activated sludge will require thickening.

Solid bowl centrifuges shall be the basis of design for the thickening unit process. The operation of a solid bowl centrifuge consists basically of a rotating unit consisting of a bowl and a conveyor joined through a gear system. The bowl and the conveyor are designed to rotate at slightly different speeds, which conveys the solids toward the discharge end of the bowl. The helical conveyor pushes the sludge solids collected at the wall of the centrifuge towards the outlet part of the centrifuge at the conical end. The liquid overflows at the opposite end of the bowl.

Class I reliability criteria require that a sufficient number of centrifuges be installed to enable the design sludge flow to be dewatered with the largest capacity unit out of service. However, if the equipment is sized based on less than 24-hour per-day operation, extension of normal working hours can be used to make up lost capacity.

The basis of design for the thickening centrifuges is the maximum month waste-activated sludge loading. As noted in Section 7.2, the maximum month loading is 7,800 lbs/day of dry solids for the Phase I facilities. Assuming a feed concentration of 0.75 percent, a feed rate of 0.125 MGD results. Based on an operation of 5 days per week and 7 hours per day, a design-hydraulic loading of 417 gpm is utilized for sizing of equipment.

The Sharples "Polymizer" solid bowl centrifuge was utilized as the basis of design. Two (2) models PM-75,000 will operate at 210 gpm and 88 horsepower. The expected performance for thickening of the waste-activated sludge is 4-6 percent solids with a capture rate of 85 percent. This performance can probably be achieved without polymer; however, a standby polymer feed system will be included in the design to improve recovery if necessary.

As indicated in Table 7-2, the average waste-activated sludge loading rate for the Phase I design is 6000 lbs/day. If one of the two centrifuge units is out of service, a resultant operational requirement of approximately 8 hours per day for a full week is required. Since this temporary operational requirement is not excessive, a full standby centrifuge unit will not be installed. However, an additional unit will be required for the Phase II addition. Table 7-4 is a summary of the preliminary design of the centrifuge thickening units.

The thickened waste-activated sludge will be conveyed by gravity to a thickened sludge wet well directly beneath the centrifuge units. Thickened waste-activated sludge feed pumps will feed the thickened sludge to the primary

TABLE 7-4

PRELIMINARY DESIGN OF CENTRIFUGE THICKENING

Number of Units (Phase I)	2
Hydraulic Loading, each	210 gpm
Concentration of Unthickened Sludge	0.5 - 1.0%
Concentration of Thickened Sludge	4 - 6%
Solids Capture, w/o Polymer	85%
Number of Thickened Waste-Activated Sludge Feed Pumps	2
Rating of Feed Pumps, each	16 gpm

digesters. The pumps will be heavy duty, self-priming, positive displacement, peristaltic type rated at 16 gpm. The pumps are based on Model No. SP/40 as manufactured by Waukesha Pump Division.

7.5 ANAEROBIC DIGESTION

Anaerobic digestion will be utilized as the stabilization process unit which is required for sludge to be landfilled. This process offers the following advantages over other sludge stabilization process units:

- o Produces methane gas.
- o Reduces total sludge mass.
- o Yields a product with greatly reduced odor potential.
- o Inactivates pathogens.

A two-stage high rate sludge digestion process will be the basis of design. A high rate process differs from the conventional system in that the solids loading rate is much greater. These higher loading rates can be achieved by improved mixing capabilities and increased heating requirements. The contents of the digesters are heated and consistently maintained at a temperature of 95^{o} F (35^{o} C). The organisms that grow in this temperature range are called mesophilic.

In the two-stage process, the first unit, or the primary digester, is where the digestion of the volatile solids occurs. This process of digestion kinetics is governed by two distinct groups of bacteria: facultative anaerobic acid producers that convert carbohydrates, proteins, and fats into organic acids and alcohols; and, anaerobic methane fermenters that convert the acids and alcohols into methane and carbon dioxide. The second unit or the secondary digester can successfully serve the following functions:

- o Thicken digested sludge.
- o Provide standby digester capacity.
- o Store digested sludge.
- o Assure against short-circuiting.

The key design parameter for anaerobic sludge digestion is the solids retention time (SRT). This parameter is the average time a unit of microbial mass is retained in the system. The basis of design will be a minimum SRT of 10 days. This criteria must be met at the following critical periods:

- o Peak Hydraulic Loading - This design value is based on the maximum month solids production rate, an 85 percent capture rate from the centrifuge thickeners, and a digester feed solids concentration of 4 percent.
- o Grit and Scum Accumulation - The active volume of the digester tanks will be reduced by four feet for grit deposit and two feet for scum blanket.
- o Liquid Level Below Highest Level - Two feet of liquid level variability will be retained to allow for differences in the rate of feeding and withdrawal and to provide reasonable operational flexibility.

Class I reliability requires that at least two digestion tanks be provided. Additionally, if mixing is required as part of the digestion process, then each tank requiring mixing shall have sufficient mixing equipment or flexibility in system design to assure that the total capability for mixing is not lost when any one piece of mechanical mixing equipment is taken out of service.

Based upon the design criteria outlined herein, two (2) primary digesters and one (1) secondary digester with a diameter of 55 feet and a side water depth of 25 feet will be provided for the Phase I design. One (1) additional 55 feet in diameter primary digester will be provided in Phase II. This design is expected to achieve a volatile solids destruction of 50 percent. Additionally, the methane gas provided from this process will be utilized to heat the digester sludge. Moreover, excess methane gas could be scrubbed and stored for utilization as an alternative fuel to power City vehicles.

One centrally located building will be provided to serve the four digester tanks. This building will house three sludge heat exchangers, two sludge recirculation pumps, two heated sludge pumps, and three transfer/digested sludge pumps for the Phase I design. One additional piece of equipment of each type shall be added for the Phase II facilities. The pumps shall be of the recessed impeller, centrifugal, solids-handling type.

Table 7-5 summarizes the preliminary design of the anaerobic digesters. Figure 7-1 illustrates a schematic of the pumps, piping, and appurtenances for the digestion complex.

7.6 BELT FILTER PRESS DEWATERING

Dewatering is the removal of water from wastewater solids to achieve a volume reduction greater than that achieved by thickening. Dewatering sludge from 5 percent solids concentration to a 20 percent solids concentration reduces volume by three-fourths and results in a non-fluid material. This substantial reduction in volume is done primarily to decrease the capital and operating costs of the subsequent sludge disposal process.

The basis of design for the dewatering process unit shall be belt filter presses. This mechanical dewatering unit employs an endless moving belt to dewater sludge continuously. The process consists of three basic operational stages: chemical conditioning of the feed sludge, gravity drainage to a non-fluid consistency, and compaction of the partially dewatered sludge.

The design loading of the belt filter presses for the Phase I design is indirectly based upon the maximum monthly solids loading of 18,200 lbs/day as developed in Table 7-2. However, a 50 percent destruction of volatile solids is expected during the digestion process. Therefore, the solids feed loading rate expected to be dewatered is 12,000 lbs/day. Assuming a 5-day-per-week and 7-hour-per-day operation, a solids loading rate of 2,400 lbs/hr is required. A typical solids handling capacity of 600 lbs/hr.-meter is utilized for design. Therefore, two (2) 2.0-meter belt presses are required to handle the maximum month solids loading for Phase I.

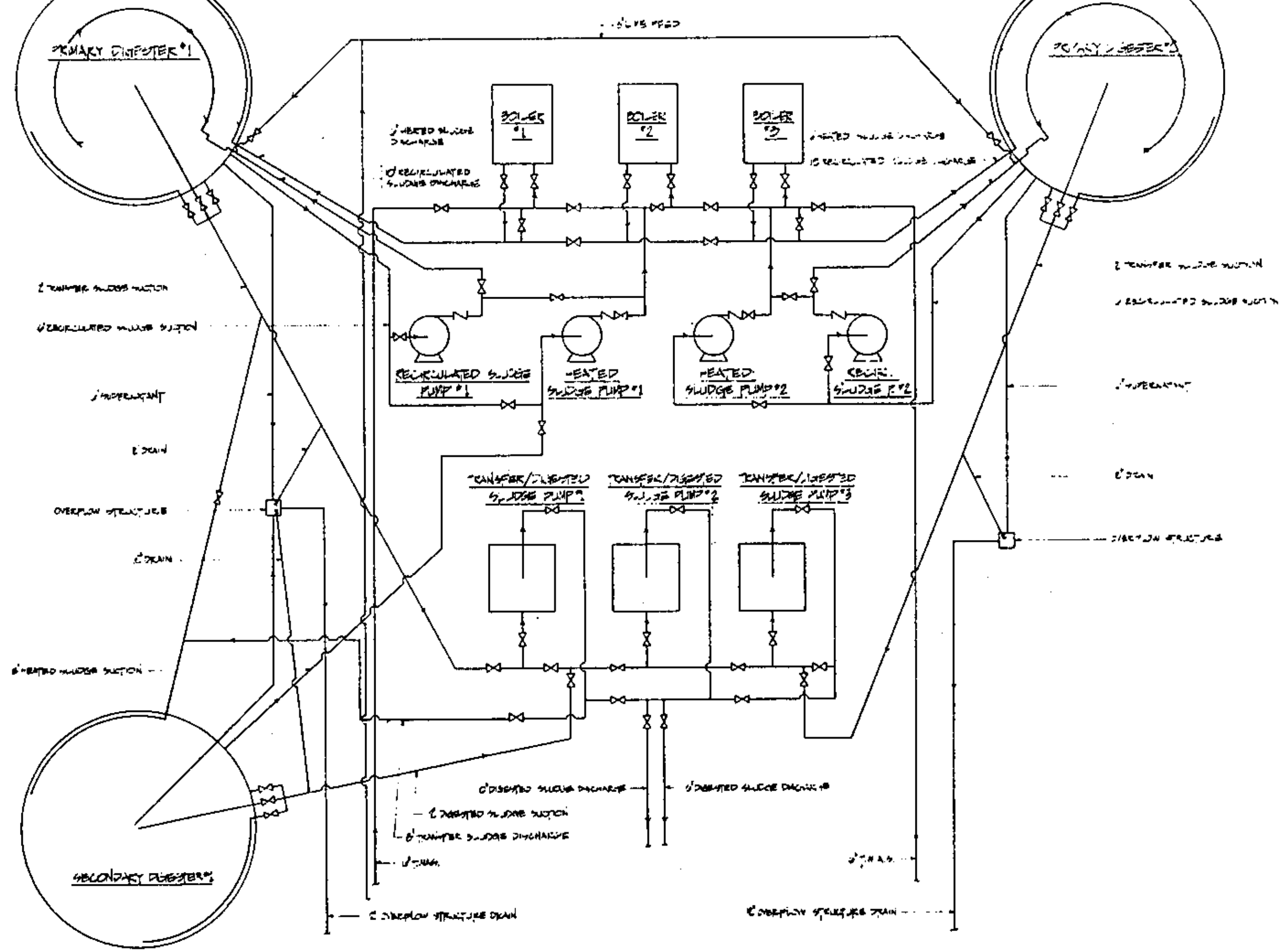
The expected performance of belt filter presses varies because results depend on many factors: type of sludge, quality of conditioning, pressure of rollers, number of rollers, etc. However, with a feed concentration of 3-5 percent of

TABLE 7-5

PRELIMINARY DESIGN OF ANAEROBIC DIGESTERS

PHASE I - 10 MGD

Number of Units (Phase I)	3
Number of Primary Tanks	2
Number of Secondary Tanks	1
Tank Diameter	55
Sidewater Depth (Max.)	25
Cover Type	Floating
Active Volume, 2 Tanks	0.60 MG
Total Volume, 2 Tanks	0.89 MG
Design Solids Loading	17,000 lbs/day
Solids Loading at Peak Conditions with Full Volume	0.11 lb VSS/day/ft^3
Solids Feed Concentration	4%
Detention Time at Peak Conditions with Active Volume	11.9 days
Detention Time at Average Conditions with Full Volume	23.7 days
Detention Time at Peak Conditions with Full Volume	17.4 days
Operating Temperature	95^oF (35^oC)
Volatile Solids Destruction	50%



SLUDGE PIPING SCHEMATIC

CDM

REGIONAL WASTEWATER TREATMENT PLANT
FOR
CITY OF PLANTATION
PRIMARY & SECONDARY DIGESTERS

6009-42

FIGURE NO
7-1

a mixture of digested primary and waste-activated sludge, a cake with a solids concentration of 20-25 percent can be expected. Polymer addition will be required at a rate of 5 to 10 lbs of dry polymer per ton of dry feed solids. An overall solids capture of 95 percent can be expected.

Table 7-6 is a summary of the preliminary design of the belt filter presses. Figure 7-2 is an illustration of a typical belt filter press installation. The dewatering building will include such items as the belt presses, polymer system, cake conveyor system, and necessary appurtenances.

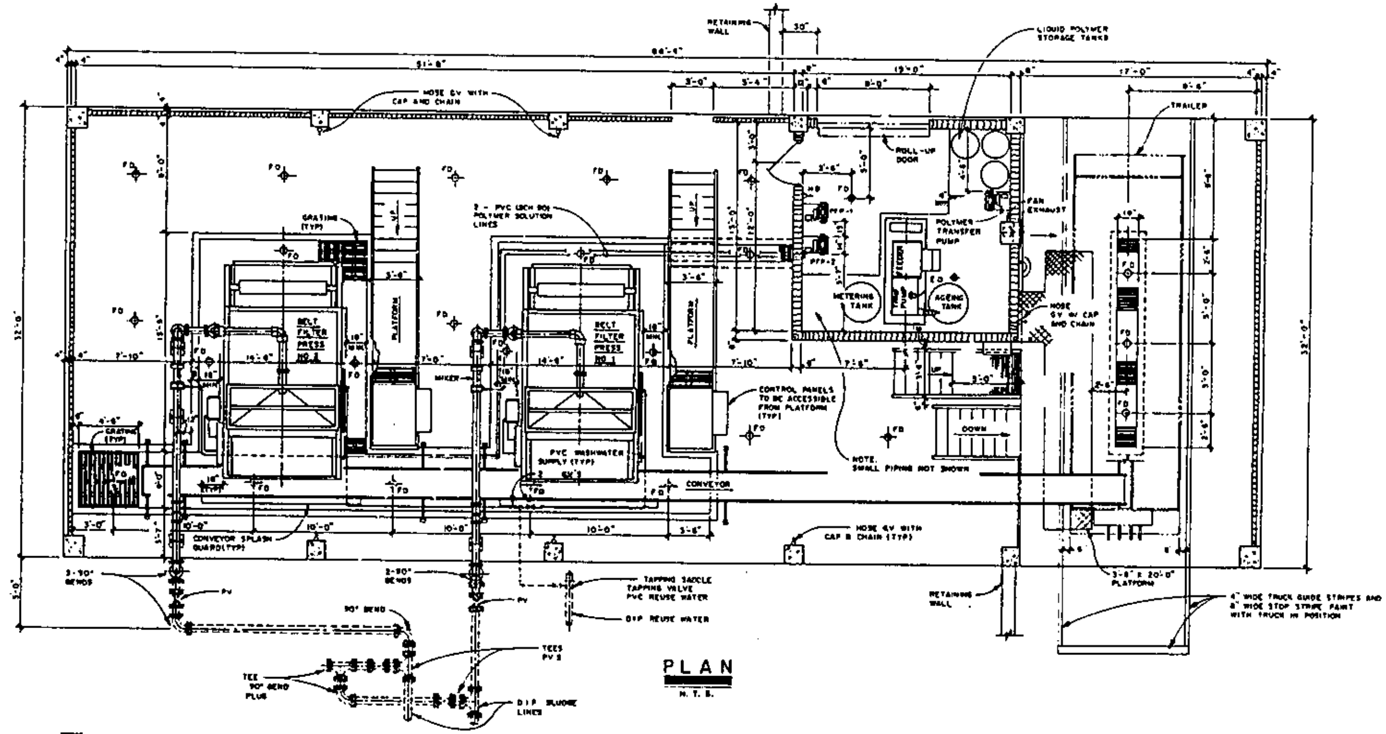
Class I reliability criteria require that a sufficient number of presses be installed to enable the design sludge flow to be dewatered with the largest capacity unit out of service. However, if the equipment is sized on less than 24-hour-per-day operation, extension of normal working hours can be used to make up lost capacity.

The average day solids loading rate for the Phase I design is 8750 lbs/day. If one of the two belt press units is out of service, a resultant operational requirement of approximately 8 hours per day for a full week is required. Since this temporary operational requirement is not excessive, a full standby belt press will not be installed. However, an additional unit will be required for the Phase II addition.

TABLE 7-6

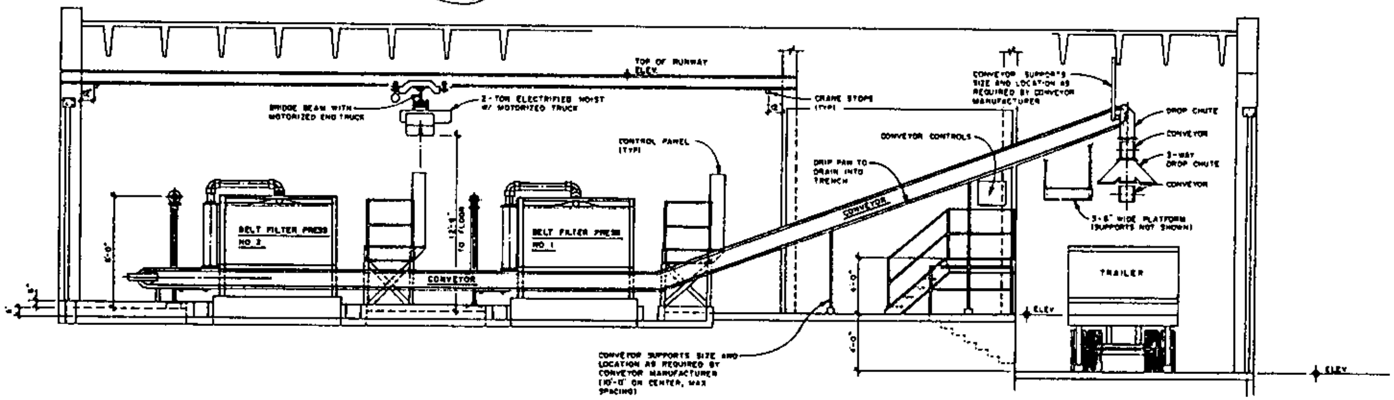
PRELIMINARY DESIGN OF BELT FILTER PRESSES

Number of Units (Phase I)	2
Size, each	2.0 meter
Sludge type	Digested Primary/Secondary
Design Solids Loading, each (2 units)	600 lbs/hr-meter
Operating Period	5 days/week, 7 hours/day
Concentration of Feed Sludge	3 - 5%
Concentration of Dewatered Cake	20 - 25%
Polymer Dose	5 - 10 lbs/ton
Solids Capture	95%



PLAN

N.T.S.



SECTION

N.T.S.

CAMP DRESSER & McKEE INC.

CDM

REGIONAL WASTEWATER TREATMENT PLANT
FOR
CITY OF PLANTATION
SLUDGE DEWATERING - PLAN & SECTION

FT. LAUD OFFICE PROJECT NO.
6009-42

FIGURE NO.
7-2

SECTION 8
DEEP WELL INJECTION

8.1 INTRODUCTION

Deep well injection of treated effluent into disposal we regulatory and technical viewpoints providing that cer ditions exist. A highly permeable, areally extensive ir present so that large quantities of effluent can be disp injection pressures. Secondly, the injection zone mus saline water with a total dissolved solids (TDS) content milligrams per liter (mg/l). Finally, the injection hor by lithologic units having an extremely low vertical p injected fluids will not migrate and contaminate aquif underground sources of drinking water. Aquifers def Department of Environmental Regulation (FDER) as undergro ing water (classifications G-I and G-II) contain water tion of less than 10,000 mg/l. Regulation and permittir are controlled under the FDER Rules, Chapter 17-28.

Geraghty & Miller, Inc. (G&M) was contracted by CDM to investigate the existence in the Plantation area of th conditions outlined above. G&M produced an injection entitled, "Preliminary Design of the Plantation Dis Plantation, Florida", which is included herein as Ap responsible for the preliminary design of the 10 MGD pha monitor system. CDM was responsible for the integratior and 15 MGD phases, cost estimates, and related matters. the G&M report are incorporated herein. This section items:

8.2 Geology
8.3 Injection Well System
8.4 Regulatory Procedures

8.5 Noise Constraints

8.6 Well Operations

8.7 Summary

8.2 GEOLOGY

Peninsular Florida is underlain by a thick sequence of carbonate rocks. The total thickness of these sedimentary rocks is in excess of 10,000 feet in the southern portion of the state. The geologic formations which are significant in the disposal of treated effluent by injection wells in Broward County are approximately 5,000 feet thick. A summary of the geologic formations and their hydrogeologic significance in the design of the injection well system is discussed in the following paragraphs.

General

The uppermost formations consist of alternating layers of sand, shell, clay, coquina, limestone, and silt. These formations comprise the Biscayne aquifer. This water table aquifer is the sole source of potable water in Broward County. The thickness of the aquifer is approximately 200 feet in the City of Plantation area.

Underlying the units which comprise the Biscayne aquifer is the Hawthorn formation. This unit primarily consists of a grey-green clay. The Hawthorn formation is the aquiclude (confining sequence) separating the water table Biscayne aquifer and the artesian Floridan aquifer. The Hawthorn formation is present to a depth of 900 to 1,000 feet in the Plantation area.

The limestones of the Ocala group underlie the Hawthorn formation. The Ocala Group forms the upper part of the Floridan aquifer. The water contained in this aquifer is brackish (greater than 250 mg/l chlorides); however, the aquifer is classified as a G-II groundwater (less than 10,000 mg/l TDS) and must be protected from possible sources of pollution. The Ocala group is anticipated to be approximately 100 feet thick in the central Broward County area.

The Avon Park and the Lake City limestone formations are similar in lithologic characteristics and are discussed together in this report. The distinction between the units is academic. The rock types of these formations are limestone and dolostone (dolomite). The formations consist of permeable and tight (low or non-permeable) zones, which are considered confining beds, particularly in the basal sections of the sequence. This sequence of limestone and dolostone is approximately 1,400 feet thick extending to a depth of approximately 2,400 feet.

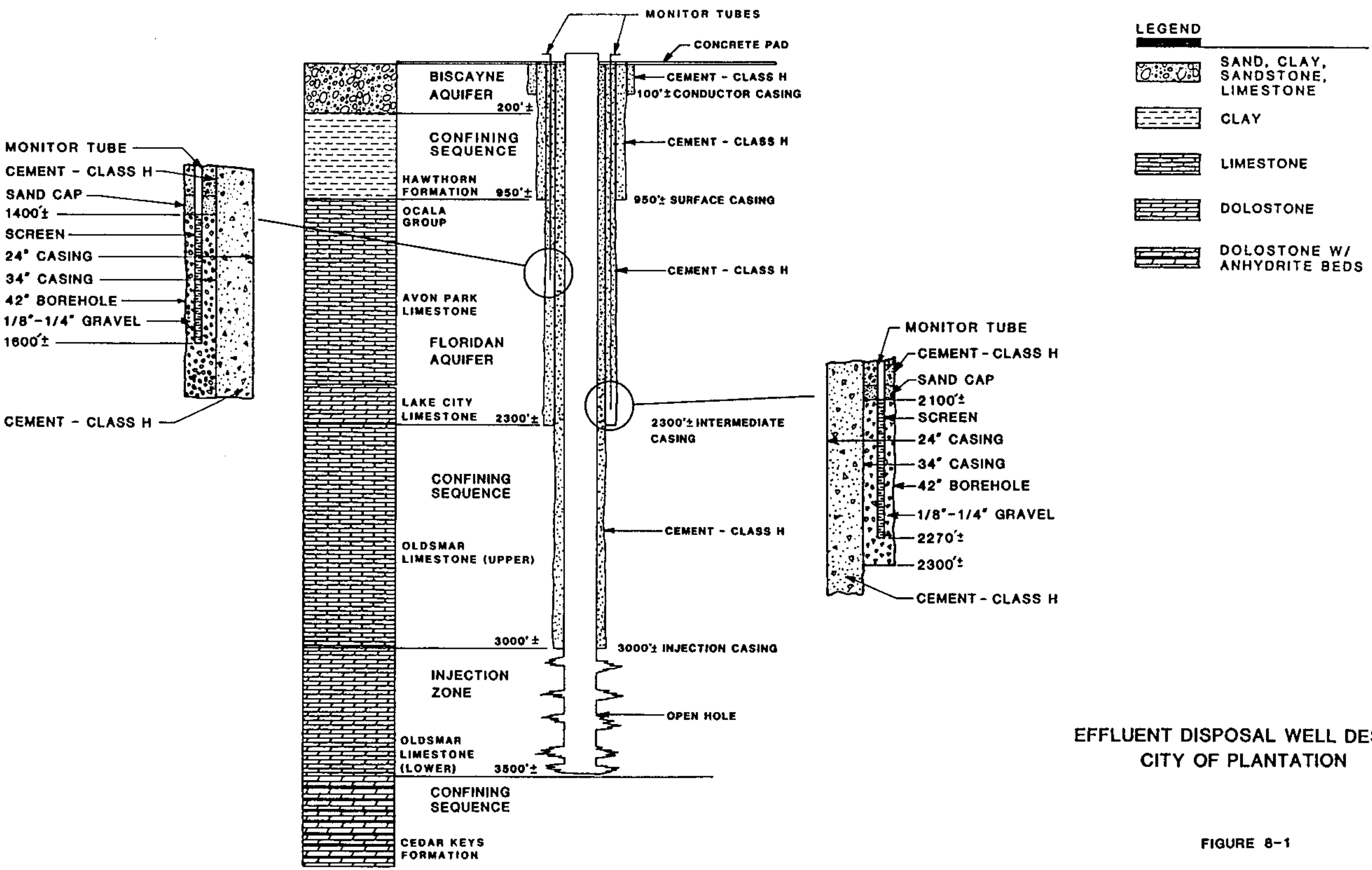
Underlying the Avon Park and Lake City limestone is the Oldsmar limestone. The Oldsmar limestone consists of two distinct rock types. The upper section is a fine grained, chalky, tight limestone. The lower section is composed of a dense, brown, crystalline dolostone, which is highly fractured and cavernous. The importance of the Oldsmar limestone is that the upper limestone unit acts as a confining bed, and the lower dolostone unit acts as the injection horizon (zone) which contains water with TDS concentrations greater than 10,000 mg/l. This injection zone is known as the "boulder zone" because of singular drilling characteristics (as if drilling through boulders).

Underlying the Oldsmar limestone is the Cedar Keys limestone, which acts as the lower confining unit to the Oldsmar injection zone. The Cedar Keys limestone is comprised of dolostone and evaporites (gypsum and anhydrite beds) and is estimated to be 1,000 feet thick.

Local Conditions

Local conditions are defined as those conditions existing in the study area. The geologic units discussed in the previous section have been shown to be areally extensive throughout southeastern Florida, based on previous studies for injection well systems for the City of Fort Lauderdale and the City of Margate. The information presented is considered to be representative of the study area. This information was used as the basis for the injection well design.

Based on the interpretation of the available disposal well data, the anticipated geologic sequence in the study area is presented on Figure 8-1. The



EFFLUENT DISPOSAL WELL DESIG
CITY OF PLANTATION

FIGURE 8-1

CDM

top of the lower unit of the Oldsmar limestone exists at a depth of approximately 3,000 feet. The test and operation data from this well demonstrate that the highly transmissive injection zone exists. Above this zone, the upper unit (confining beds) of the Oldsmar limestone exists to a depth of approximately 2,500 feet. A cavernous dolostone is present from a depth of 2,100 to 2,270 feet. This zone contains saline water and can be utilized for water quality monitoring purposes. Dense to chalky dolomitic limestones occur from 2,100 to 1,400 feet, which function as confining beds separating the saltwater zones and the brackish water zone. The Ocala group is present above 1,400 feet and contains potentially potable water between 950 to 1,150 feet. This constitutes the upper portion of the Floridan aquifer.

The Hawthorn formation exists from 300 to 950 feet as the aquiclude between the Biscayne aquifer and the Floridan aquifer. This unit prevents contamination of the fresh water in the Biscayne aquifer with brackish water from the Floridan aquifer.

8.3 INJECTION WELL SYSTEM

The design of the injection well system is based on a two-well system implemented in a two-phase approach. The 10 MGD phase will consist of one 24-inch diameter injection well in operation by the year 1986, and the 15 MGD phase will consist of a second 24-inch diameter injection well in operation by the year 1991. The design of the injection well system will consider the maximum daily flows for both the 10 MGD and the 15 MGD phases. Equalization will be incorporated into the flow system so that flow in excess of maximum day will be retained. Each phase consists of drilling an injection well to a total depth of approximately 3,500 feet to penetrate most or all of the injection horizon. Both wells will be constructed in a similar manner.

Injection velocities and injection pressures (in feet of head and pounds per square inch, respectively) were calculated for both phases for the average annual, maximum month, and maximum 24-hour flow rates. Injection velocities were calculated using the equation (Darcy-Wiesbach-Colebrook):

$$V = \frac{.4085 \times (\text{gpm})}{d^2} \tag{8.0}$$

where

V = velocity of flow, ft/sec

d = inside diameter of circular pipe, inches

gpm = injection rate

Injection pressures were calculated using the Hazen and Williams equation, rewritten in a convenient form:

$$h_f = \frac{4.727}{D^{4.87}} \times L \times \left(\frac{Q}{C_1}\right)^{1.85} \tag{8.1}$$

where

h_f = friction loss, ft

D = inside diameter of pipe, ft

L = length of pipe, ft

Q = discharge, cfs

C_1 = coefficient, dependent on surface roughness

Conversion of the injection pressures from feet of head to pounds per square inch was performed using the following equation:

$$\text{psi} = \frac{h}{2.23} \tag{8.2}$$

where

psi = pounds per square inch

h = head, ft

Compensation for the density differential of "fresh water" and salt water is given by:

$$\begin{aligned}\text{density differential} &= 3{,}000 \times (1.025 - 1.00) \\ &= 75 \text{ feet of head}\end{aligned} \tag{8.3}$$

The velocity and injection pressures for each phase are shown in Table 8-1. A single, 24-inch diameter well will accomodate the 10 MGD phase design flow of 14.1 MGD. Two 24-inch diameter wells will provide more than enough capacity to accomodate the 15 MGD phase design flow of 21.3 MGD. The well head injection pressure in one well at the rate of 14.1 MGD is estimated to be about 111 feet of head. The total pumping head in each well for the 15 MGD phase is less than the 10 MGD phase assuming equal flow is diverted to each well. The well head injection pressure for each well during the 15 MGD phase is estimated to be about 97 feet of head. The effluent pumping system will be capable of pumping the total 15 MGD phase (21.3 MGD) design flow down a single well to provide emergency discharge in the unlikely eventuality that one well should fail. The well head injection pressure at this rate will be 150 feet of head and the injection velocity will be 11.4 fps (14,792 gpm). This velocity exceeds the recommended velocity of 10 fps for raw wastewater transmission lines. However, treated effluent is similar to fresh water and the 11.4 fps is therefore not considered detrimental from a standpoint of well integrity. The 11.4 fps velocity is a maximum peak day velocity which would occur only over short periods and only in the eventuality that one well should fail.

The injection well system will have back flushing capabilities for each well. The back flushing procedure will be done on a routine basis during low flow periods with the back flow being routed into the storage tank. This procedure will enhance the longevity of the wells and reduce the possibility of the formation becoming plugged.

The wells will be spaced approximately 300 feet apart. This spacing will be more than adequate to provide working area for the second phase of construction. The 300-foot spacing will also be sufficient to negate pressure build up or interference between the wells when both are in operation.

Well Construction

The successful completion of a disposal well is dependent on the reliability of the information collected during the test/production well drilling. The

TABLE 8-1

INJECTION PRESSURES FOR 10 MGD AND 15 MGD PHASES

10 MGD PHASE

Flow	Rate (MGD)	Velocity (fps)	Head[a] (ft)	Head[a] (psi)
Average Annual	8.3	4.5	89.3	38.5
Maximum Month	10.0	5.4	94.7	40.5
Maximum Day	14.1	7.6	111.4	48.0

15 MGD PHASE[b]

Flow	Rate (MGD)	Velocity (fps)	Head[a] (ft)	Head[a] (psi)
Average Annual	6.3	3.4	83.9	36.2
Maximum Month	7.5	4.0	87.0	37.5
Maximum Day	10.6	5.7	97.0	41.8

15 MGD PHASE [c]

Flow	Rate (MGD)	Velocity (fps)	Head[a] (ft)	Head[a] (psi)
Average Annual	12.5	6.7	104.3	45.0
Maximum Month	13.0	8.0	115.7	49.0
Maximum Day	21.3	11.4	150.0	64.6

(a) The heads reflect 75 feet density differential and bottom hole driving pressure.
(b) Assumes equal injection rates for both wells.
(c) Rate assumes one of two wells is inoperable and all fluid is being injected into one well.

test/production well will incorporate the drilling of a test/pilot hole to determine the exact depths of casings, monitor zones, confining beds and injection horizons at the Plantation site.

The drilling of the test/pilot hole will involve two steps. During the first step, the test/pilot hole will be drilled to a depth of 1,000 feet. Formation samples will be collected at 10-foot intervals or at every formation change. The information collected during this operation will define the depth of the 54-inch diameter conductor casing and the 42-inch diameter surface casing.

The second step of the pilot hole drilling will commence after the installation of the 54-inch and the 42-inch diameter casings. Formation samples will be collected as previously stated. Approximately eight formation cores will be taken from 1,500 to 3,000 feet. Vertical and horizontal permeability analyses will be performed on each core. Upon the completion of the test/pilot hole drilling, geophysical logs will be run in the bore hole. The geophysical logging will include but not be limited to: single point electric, gamma ray, temperature, flow meter, caliper, dual induction, neutron porosity, and bore hole compensated logs. The information obtained from the geophysical and geological logs will assist in the design of the straddle packer (a device used to separate hydraulic zones in the bore hole) test program.

The test/pilot hole program will define the hydrogeologic properties of the formations. The final selection of casing depths, monitor zones and injection zones will be dependent on the information collected during the drilling. The data will be collected and analyzed, and the construction monitored by a trained hydrogeologic and engineering staff to ensure the proper construction and testing of the well. These activities will require full-time, on-site inspection by hydrogeologists during all phases of well drilling.

The test/production well for each of the 10 MGD and 15 MGD phases will be drilled by conventional mud rotary methods through the Hawthorn formation. Below the Hawthorn formation, reverse circulation drilling methods will be used. Below the bottom of the 24-inch diameter casing, only reverse air circulation will be permitted. Reverse air circulation is the most effective way of succesfully completing (developing) the injection horizon.

A straight bore hole is essential in the completion of any large diameter deep injection well. A straight hole is defined as a hole that does not deviate from vertical or contain any abrupt changes in hole direction (dog legs). To ensure that these requirements are met, the specifications will contain stringent straight hole testing requirements including: inclination surveys as test/pilot hole drilling is in progress, gyroscopic surveys at the completion of each phase of test/pilot hole drilling at approximate depths of 1,000 and 3,000 feet, and directional surveys in all reamed holes following pilot holes below 1,000 feet. The contractor will not be allowed to deviate more than 0.5 degrees for each 60 feet drilled and not be allowed to exceed one degree of inclination during any drilling operation. The requirement of directional surveys for the reamed holes is essential for two reasons: first, to ensure that the hole is straight and no dog legs exist; and second, to ensure the integrity of the confining beds by verifying that the reamed hole is tracked (followed) by the pilot hole.

Protection from the possibility of contamination of the potable and potentially potable water supplies of the Biscayne and Floridan aquifers will be achieved by the construction of the disposal wells with four strings of casing of varying diameters. The casings depths are selected to protect the potable groundwater sources or for construction purposes. The inner string of 24-inch diameter casing (injection casing) will be set to a depth of approximately 3,000 feet below land surface (bls). The 34-inch diameter intermediate casing will be set to a depth of 2,300 feet bls. The surface string of 42-inch diameter casing will be set to a depth of approximately 1,000 feet bls, and the conductor string of 54-inch diameter casing will be set to a depth of 100 feet bls. Each casing string will be completely cemented in place (with the exception of the intermediate casing zone from approximately 2,100 to 2,270 feet bls and 1,400 to 1,600 feet bls), using an API Class H cement and lost circulation additives to ensure that a good bond is achieved between the casing and bore hole. The amount of time required to construct one 24-inch injection well is approximately 180 days. A summary of the information presented above is given in Table 8-2.

TABLE 8-2

CASING STRINGS

Casing	Diameter (inches)	Wall Thickness (inches)	Setting (feet below grade)	Bit Size (inches)
Conductor	54	0.375	100	62
Surface	42	0.375	1,000	52
Intermediate	34	0.375	2,300	40.5
Inner (Injection)	24	0.500	3,000	32
Open Hole	--	---	---	22

Each injection well will be equipped with two steel monitor tubes set in the annular space opposite the intermediate casing (34-inch diameter) to monitor the saline water zones between 2,100 and 2,270 feet, and 1,400 to 1,600 feet, respectively. The monitor tubes will be screened and the screened intervals will be gravel packed. A sand cap will be placed on top of the gravel packs and the remainder of the annulus will be cemented. The monitor tube from 1400 to 1600 feet could be cemented in place and directionally perforated, rather than screened and gravel packed. This type of construction accomplishes the monitoring of the saline water zones, while eliminating the need to cement a cavernous zone (2,100-2,300 feet) that may be impossible or extremely expensive to seal using cement grout.

A second method of monitoring is to drill a separate monitor well. This method is objectionable from a cost standpoint and does not enhance the monitoring abilities over the system discussed above. The monitor tubes act as an "early warning system" for the protection of present and potential drinking water sources. Although the possibility is remote that the treated effluent would migrate upward through 700 feet of dense limestone, the monitor tubes would detect possible occurrence of this event. Early detection of contamination would enable remedial action to be taken before potable sources of water are contaminated. The constituent monitoring requirements for each system are dependent on specific FDER requirements. As an example, the City of West Palm Beach is required to monitor for fecal coliform, biological oxygen demand, and chlorides for their injection well system.

Safety Precautions

The last step in the construction of an injection well is the testing of the ability of the well to accept fluids. This test usually can be performed by injecting into the well or by pumping out of the well. However, there cannot be any pump out test at the Plantation site since the boulder zone contains saline water with no ready source of saline water disposal. The test will therefore have to be conducted as an injection test. During the injection testing, careful monitoring of the Floridan aquifer monitor wells will be conducted to ensure that there is no hydraulic connection between the injection horizon and the monitor zone, thereby verifying the integrity of the injection horizon. Permits will be obtained for use of water from the East Holloway Canal or the use of treated effluent for the injection well testing.

The disposal wells will be constructed using cost-effective and environmentally safe methods. The possibility of contaminating both surface and groundwaters will be minimized during the construction of the injection wells. This will be accomplished by the use of concrete drilling pads, steel-lined tanks, and sealed sump pits. Drill cuttings, drilling mud, other fluids, and potential contaminants will be properly disposed of in approved areas. Shallow groundwater monitor wells will be located at critical points around each pad and monitored on a weekly basis for evidence of contamination.

Special precautions against contamination will be incorporated in the construction of the 15 MGD phase well. These precautions are necessary because the saline water in the boulder zone will have been displaced by six years of treated effluent disposal in the 10 MGD phase well. The second well will have a tendency to back flow when drilling in the boulder zone in the same manner as used in the back flushing operation of the wells (described in Section 8.6). During the drilling of this well in the injection horizon, the contractor will be required to continually suppress the "fresh water" head on the well.

8.4 REGULATORY PROCEDURES

The Rules of the FDER, for underground injection control, Chapter 17-28, provide for the formation of a Tactical Advisory Committee (TAC) to evaluate each injection well system permit application. The TAC will be responsible for the evaluation of the adequacy of the test/injection well design and testing program in the issuance of a construction permit. The review of the design includes the injection wells and the entire injection well system. The system includes the effluent quality, pumps, lines, surge protection (water hammer), back flow system, monitor wells, and instrumentation. The TAC will evaluate the redundancy in the system and emergency back up systems.

During the actual construction of the injection wells, the TAC will be kept advised during all phases of construction. TAC concurrence with major construction decisions (casing depths, monitor zones, etc.) will be required. Upon completion of construction and testing, all results will be presented to the TAC. TAC approval of the constructed system is required prior to the issuance of a FDER operational permit.

The proposed effluent disposal system will ultimately consist of two deep wells. Redundancy will be provided since either well will be capable of receiving 100 percent of the plant effluent if necessary. However, during the first phase of operation, 1986 through 1991, only one well will be constructed. Back-up for this well will be provided by the adjacent canal during this interim period. A position paper issued by the FDER in Tallahassee in 1981 and included as Appendix C herein states that, "The back-up system is not required to be another injection well unless there is no other acceptable option."

However, recent practice of the FDER in West Palm Beach has been to require a second standby well for facilities designed to treat 7 MGD or more. Special circumstances at the proposed facility may allow the local FDER to waive the requirement of a second deep well during the interim period. These mitigating factors include: (1) an acceptable standby emergency discharge exists, which is the canal adjacent to the plant site, and (2) it is unlikely that the well will become inoperative during the first five years of service due to the well's design and the nature of the boulder zone. The back-up system does not require a separate operating permit. However, the injection well permit should adequately address the back-up disposal system.

8.5 NOISE CONSTRAINTS

The most efficient and cost-effective well construction operation is 24 hours per day, seven days per week. Drilling operations are carried out on a continuous basis because of reduced costs, and because stopping certain procedures could jeopardize the operation and the integrity of the well. The costs of well construction will increase by 40 to 50 percent by restricting the working hours. Additional engineering costs will be incurred from the increased time of inspection. However, this type of operation may cause objectionable noise levels which could violate City of Plantation noise ordinances. In addition, the proposed injection well sites are located in the northwest quadrant of the treatment plant property, approximately 600 feet from a hospital and 750 feet from residential neighborhoods.

Certain drilling operations present more problems with noise levels than other drilling operations. The most objectionable noises occur during laying down of drill pipe, brake noise during casing setting, and drilling in the injection zone. Additional noises are created by the rig, pump, and air compressor motors and cementing operations. The City of Plantation noise ordinance states that the operation of "noisy" businesses is unlawful except between the hours of 8:00 a.m. and 6:00 p.m. on weekdays from December 1st to April 1st of each year, and between the hours of 7:00 a.m. and 6:00 p.m. on weekdays, the remainder of the year. The ordinance provides for a variance to be granted upon application to the City Council. The Council's approval is dependent on the results of their investigations as to the need for the variance.

The noise problems will be mitigated to some degree by stringent noise specifications, which will require the Contractor to take precautions against objectionable noises. The precautions against noise will include muffling all engines, and covering with wood (where practical) all metal surfaces that could come in contact with drill pipe, tools, casings, etc. In addition, a noise barrier will be required (this method was successful at the Fort Lauderdale site). Although these precautions will help reduce the noise level, application to the Council for a variance will still be necessary. The application for variance will be supported by evidence on the adequacy of noise reduction procedures and support of an injection well expert justifying the need for continuous operations during well construction and testing. In addition, the City should develop and implement a public relations program to educate the public as to the need for these operations.

8.6 WELL OPERATION

Based on a growing body of operational data generated over the past few years, the level of knowledge and expertise regarding the installation and operation of injection wells has improved significantly. Confidence in such systems has improved, and operators and regulators alike have become aware that not only must the injection wells be carefully designed, constructed, and operated, but other portions of the effluent disposal system must be given the same attention. Experience gained from current disposal operations indicate several matters that must be addressed in any injection well system design.

Adequate surge or water hammer protection must be incorporated in the design of the effluent pumping system. Pressure surges associated with water hammer have been observed at several older and inadequately designed facilities. The theoretical collapse strength of the 24-inch casing (0.5-inch wall thickness) is 442 psi. The potential water hammer pressures at the peak injection rates were analyzed for both the 10 MGD and 15 MGD phases (water hammer-instantaneous closure, no friction loss) using the following equation: (Daugherty & Franzini):

$$P_h = \frac{(e \times Cp \times V)}{144} \qquad (8.4)$$

where

e = density of water, 1.94 lbm/ft^3

v = velocity, fps

P_h = water hammer pressure, psi

Cp = pressure wave velocity, fps

and

$$Cp = 4720 \times \sqrt{\frac{1}{(1 + (\frac{D \times Ev}{t \times E}))}}$$

D = inside diameter of pipe, inches

t = wall thickness of pipe, inches

Ev = volume modulus of the medium

E = modulus of elasticity of pipe material

The calculations are extremely conservative. At the maximun day flow rate (15 MGD phase, one well operational) and associated 11.42 fps velocity, an estimated maximum water hammer pressure of 601 psi could occur, clearly indicating the need for surge protection. The water hammer pressure will be less for the 10 MGD phase (one well) and the 15 MGD phase with two wells operating. The calculated pressures are 398 psi and 301 psi, respectively.

The injection well system must have redundancy in the operational features of the system to minimize potential operating difficulties and the possibility of injection system failure. This must include surge protection, back flow capabilities, dual instrumentation for data collection, chlorination system, and screening for removal of solids. Incorporating these considerations in the system design will enhance the efficient operation and longevity of the system. Several additional matters which must be addressed are listed below.

- The potential for plugging of the injection zone by solids will be mitigated by the inclusion of screens and back flushing capability in the effluent disposal system design.
- Although FDER has relaxed the requirement for chlorination of effluent disposed in injection wells, facilities for chlorination will be included in the event some form of disposal other than the injection wells are required in an emergency situation. These facilities are discussed in Section 6.
- Some form of data collection must be included with back-up. Information on injection pressures, rates, and total or cumulative volumes must be collected from each operating well. Continuous records of injection rates and pressures must be maintained.

8.7 SUMMARY

The possibility of favorable hydrogeologic conditions existing for an injection well system in the Plantation area is excellent. The preliminary design of the injection well system is envisioned as a two-well system implemented in two phases. The first well will be completed for the 10 MGD phase and will be operational by 1986. The second well will become operational for the 15 MGD phase in 1991. Each well will be constructed to a depth of 3,500 feet using four strings of casing with 54-inch, 42-inch, 34-inch, 24-inch diameters set at 100 feet, 1,000, 2,300 feet, and 3,000 feet, respectively, and an open hole from 3,000 to 3,500 feet. All casing will be cemented in place using a cement formulated specifically to formation and fluid conditions. Two monitor tubes will be set in the annulus between the 34-inch diameter casing and the 40-inch diameter bore hole. The depths of the monitor tubes will be approximately 2,170 to 2,100 feet and approximately 1,600 to 1,400 feet.

Certain problems exist in the construction of any injection well system. For the Plantation system, concerns over emergency discharge and back-up during the five years of the 10 MGD phase will be addressed with the regulatory agencies (FDER and BCEQCB). Another major concern is noise abatement during injection well construction for both phases. A variance will be required from the City Council concerning noise requirements and working hours, or objectionable costs will be incurred. Environmentally safe methods will be employed during all well construction activities to minimize the possibility of contamination of potable water sources. This will include special precautions employed during the drilling of the second well (15 MGD phase), to abate back flow problems.

SECTION 9
ELECTRICAL AND INSTRUMENTATION SYSTEMS

9.1 INTRODUCTION

In this section the electrical and instrumentation systems are evaluated and designed for the wastewater treatment plant. This section is subdivided into the following subsections:

9.2 Existing Electrical System
9.3 Proposed Electrical System
9.4 Instrumentation

9.2 EXISTING ELECTRICAL SYSTEM

The existing plant is served by 480 volt, 3 phase, 4-wire power from an onsite transformer. A 600 amp, 480 volt, 3 phase motor control center is located in an electrical room adjacent to the transformer vault. All 3 phase motors are operated at 460 volts from the motor control center. Stepdown transformers provide 120 volts for lighting and miscellaneous power.

A 200 kw, 480 volt generator provides a limited amount of backup power to the plant site. The generator operation is manual.

9.3 PROPOSED ELECTRICAL SYSTEM

The present Florida Power & Light Company transformer vault is not large enough to serve the new plant. A new vault, electrical room, and standby generator will be located in the Administration Building as detailed in Section 10. The electrical service will be 480 volt, 3 phase, 4-wire power with dual mains, each sized to carry the entire plant load. Stepdown transformers will be used to provide 120 volt power for lighting and miscellaneous power.

A generator room will be located adjacent to the electrical room to house one generator for Phase I with provisions for a second generator for Phase II. The generators will be automatic in operation with remote control circuits to allow the operator to use either generator or both for peak shaving.

Table 9-1 is a summary of the expected electrical demands for the Phase I and Phase II facilities. As noted in the table, the two generators will each need to be rated at 1000 kw. The basis of design is Caterpillor Model No. 3512. Figure 9-1 is an illustration of a typical generator installation.

9.4 INSTRUMENTATION

Instrumentation will be designed to facilitate a centralized mode of operation from a Control Room located in the Administration Building. The on-duty plant operator will be able to monitor the operation of all equipment from the Control Room. The instrumentation system will consist primarily of the following functions:

- Alarm Annunciator
- Status Light
- Meter System
- Radio Telemetry System
- Effluent Disposal Wells

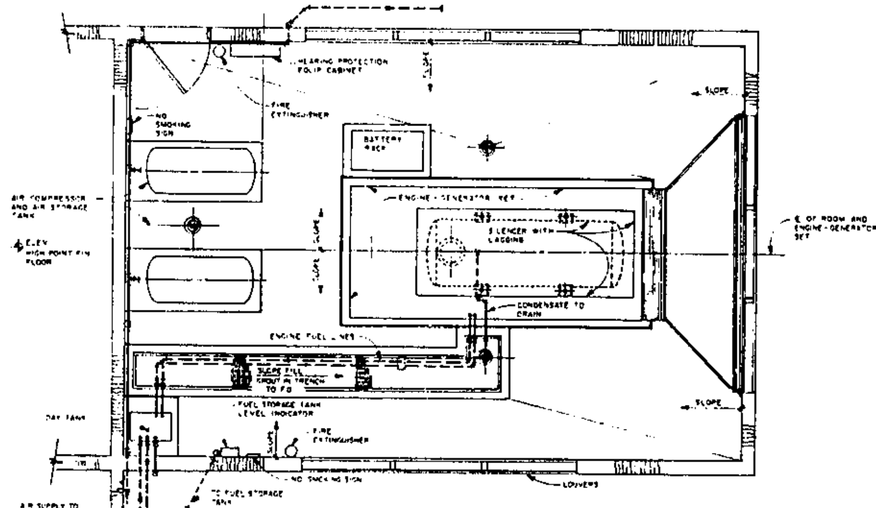
Alarm Annunciator

An alarm annunciator shall be provided to monitor the condition of equipment where failure could result in an effluent violation. Alarm annunciators shall also be provided to monitor conditions which could result in damage to vital equipment or hazards to personnel. The alarms shall sound in areas normally operated and also in areas near the equipment. The alarm annunciators shall be such that such announced condition is uniquely identified. Test circuits shall be provided to enable the alarm annunciators to be tested and verified to be in working order.

TABLE 9-1

ELECTRICAL DEMANDS

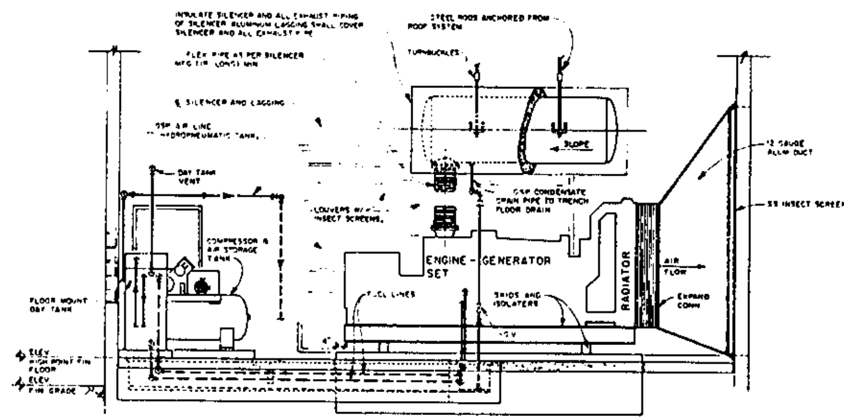
Components	Phase I	Phase II Additional
Screens	2 @ 1 = 2	3 @ 1 = 3
Grit Collectors	2 @ 6.5 = 13	3 @ 6.5 = 20
Grit Pumps	2 @ 5 = 10	3 @ 5 = 15
Primary Clarifiers	2 @ 1 = 2	3 @ 1 = 3
Waste Feed Pumps	2 @ 15 = 30	3 @ 15 = 45
Aerators	2 @ 325 = 650	3 @ 325 = 975
Secondary Clarifiers	2 @ 1 = 2	3 @ 1 = 3
Recirculation Pumps	2 @ 20 = 40	3 @ 20 = 60
Transfer Pumps	2 @ 25 = 50	3 @ 25 = 75
Effluent Pumps	4 @ 150 = 600	5 @ 150 = 750
On-site Pump Station	1 @ 20 = 20	1 @ 20 = 20
Centrifuges	2 @ 150 = 300	3 @ 150 = 450
Sludge Feed Pumps	2 @ 15 = 30	3 @ 15 = 45
Digester Complex	150	200
Dewatering Building	20	30
Control Building	5	5
Chlorine Building	10	10
Maintenance Building	5	5
Total	1939 HP = 1446 kw	2714 HP = 2024 kw



FLOOR PLAN

GENERATOR ROOM

NOT TO SCALE



SECTIONAL ELEVATION

NOT TO SCALE

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REGIONAL WASTEWATER TREATMENT PLANT
FOR
CITY OF PLANTATION
GENERATOR ROOM

6009-42

9-1

Status Lights

Running-off status indication lights will be located on the instrumentation panel for all major pieces of equipment. Identification name plates of each component shall be permanently affixed to the panel for all status-indicating lights. Control of equipment shall be local with hand-off-automatic switches.

Meter System

The metering system will consist of the following functions:

- Influent Flow Meter
- Recirculation Sludge Flow Meter
- Primary Waste Sludge Flow Meter
- Waste-activated Sludge Flow Meter
- Effluent Reuse Flow Meter
- Deep Well Disposal Flow Meter

The influent, recirculation sludge, primary waste sludge, the waste-activated sludge, and the deep well disposal flow meters shall be of the ultra-sonic type. The flow meters will consist of a cast iron spool piece, a pair of dual-path transducer probes integrally cast, and transmitting electronics. The electronics shall produce a 4-20 mA output which is linear with respect to flow. Receivers, which will be mounted in the common instrument panel, shall be activated by a 4-20 mA DC signal. The charts for the receivers will be 12 inches in diameter and be graduated on a basis of one-day increments. Additionally, an indicator and totalizer will be provided.

The effluent reuse flow meter shall be of the propeller type. The meter will have an extended meterhead register with a six-digit direct-reading totalizer. A remote signal to the instrumentation panel will not be provided.

Radio Telemetry System

A radio transmitter, receiver, and a central terminal unit will be located in the Administration Building. The radio system will monitor the new wastewater pumping stations with possible remote control for the pumps. Also, some consideration will be given to monitoring some critical functions on key pumping stations now in operation.

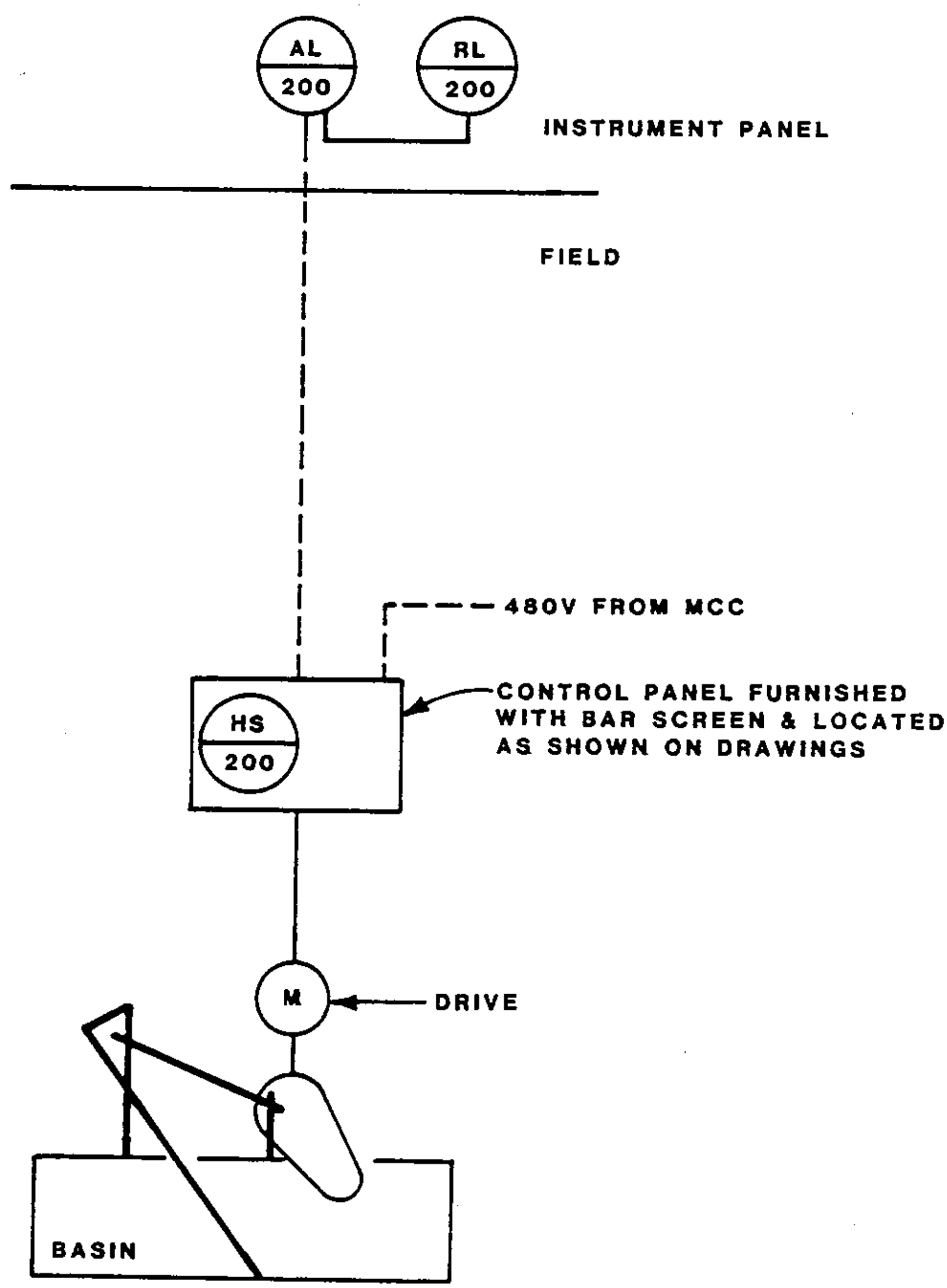
Effluent Disposal Wells

As noted in the metering subsection, a continuous recording of the injection rates will be obtained. Additionally, recorders shall be provided for data collection on the injection pressures.

Loop Diagrams

Typical instrumentation loop diagrams are illustrated in Figures 9-2 and 9-3. Figure 9-2 is a typical diagram for a mechanical piece of equipment. The example shown is for the bar screen equipment. Process control is incorporated at the local control panel with a hand-off-remote selector switch. A run indicating light and an alarm light signal are transmitted to the instrument panel located in the Control Room. Figure 9-3 is a typical diagram for a meter system. The example shown is for the plant influent flow meter. A 4-20 ma signal is transmitted to a flow-indicating recorder located in the instrumentation panel. Additionally, a 4-20 ma signal is transmitted to the chlorinators for the purpose of pacing chlorination with influent wastewater flow.

ITEM	DESCRIPTION	TYPE	CRITERIA	NAME PLATE/SERVICE	QTY.
HS 200	HAND-OFF-REMOTE SELECTOR SWITCH	3-POSITION	LOCATED IN CONTROL PANEL	BAR SCREENS	2
RL 200	RUN INDICATING LIGHT	RED	INSTRUMENT PANEL	BAR SCREENS	2
AL 200	ALARM LIGHT	ANNUNCIATOR	INSTRUMENT PANEL	BAR SCREEN EQUIPMENT FAILURE	2



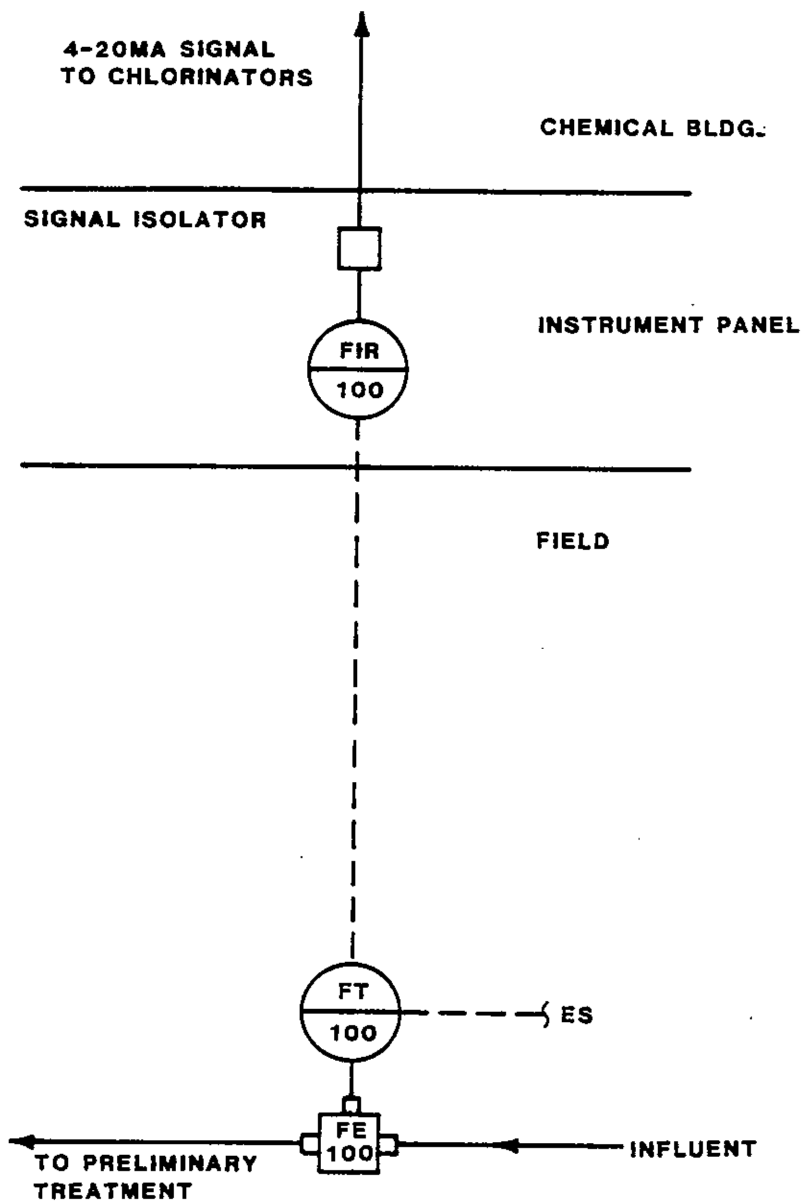
INSTRUMENTATION LOOP DIAGRAM–
MECHANICAL EQUIPMENT

FIGURE 9-2

CDM

AF 28917A

ITEM	DESCRIPTION	TYPE	CRITERIA	NAME PLATE/SERVICE	QTY.
FE 100	FLOW SENSOR	SONIC	IN METER PIT	INFLUENT	1
FT 100	FLOW TRANSMTTER	ELECTRONIC, 4-20MA OUTPUT	IN METER PIT	INFLUENT	1
FIR 100	FLOW INDICATING RECORDER	CIRCULAR CHART RANGE 0-7000 GPM	MOUNTED IN INSTRUMENT PANEL	PLANT INFLUENT	1



INSTRUMENTATION LOOP DIAGRAM–
METER SYSTEM

FIGURE 9-3

CDM

SECTION 10
ADMINISTRATION BUILDING

10.1 INTRODUCTION

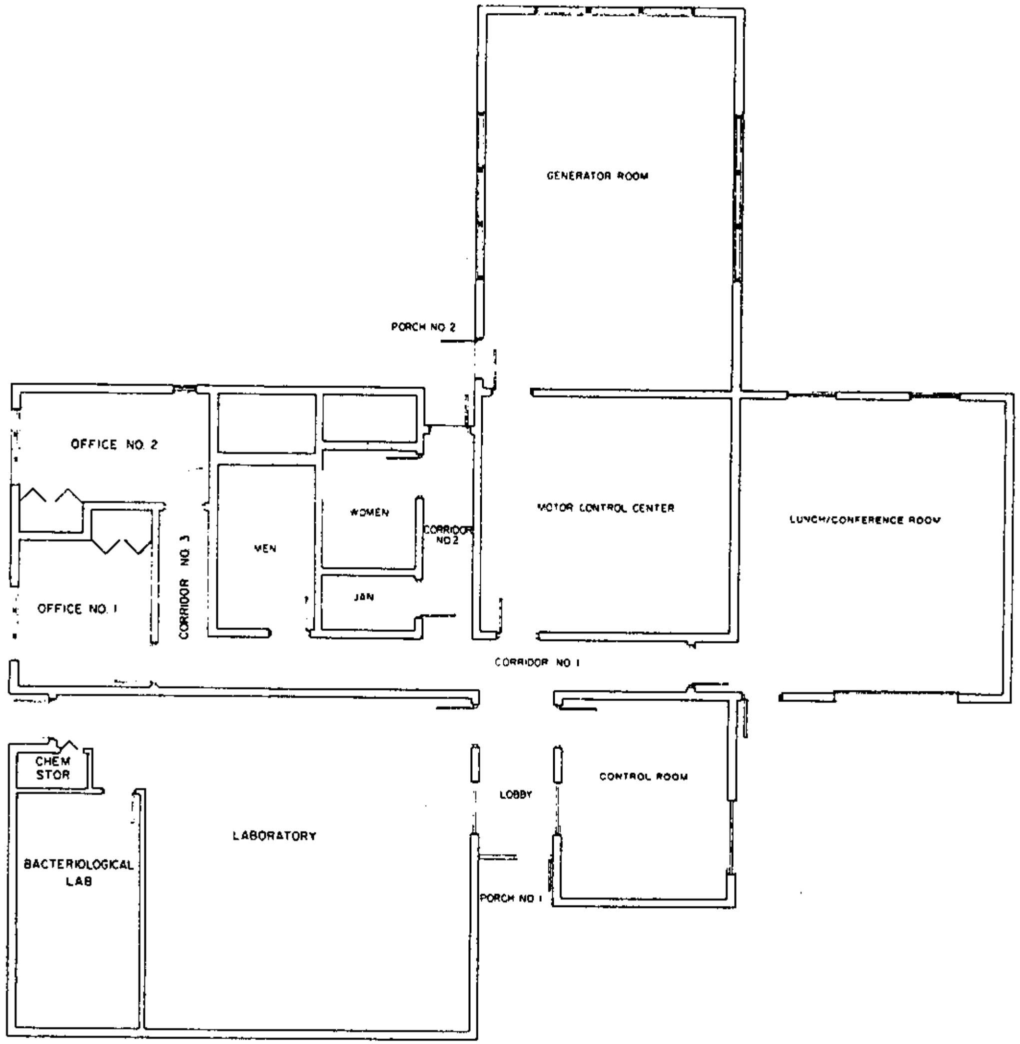
A wastewater treatment plant administration building is a multi-purpose facility that is designed to accommodate the administrative, laboratory testing, plant control, storage, and other support functions of the plant. The following functions of an administration building are described in Section 10:

10.2 Laboratory Facilities
10.3 Offices and Control Room
10.4 Sanitary Facilities
10.5 Lunch/Conference Room
10.6 Motor Control Center and Generator Room
10.7 Maintenance Building

Figure 10-1 is the proposed layout for these facilities.

10.2 LABORATORY FACILITIES

There are essentially four reasons for laboratory testing at wastewater treatment facilities: process control, cost control, historical data, and the requirements of regulatory agencies. Process control testing ensures that a given unit process is operating properly. Cost control testing may be used to reduce plant operating expenses. Historical testing provides a backlog of data concerning wastewater characteristics or process flow features which can be used as a basis for design of future expansions and as a record in the event of investigations of facility operation. Testing required by regulatory agencies is determined by applicable permit requirements and local conditions, such as the characteristics of the receiving water body.


GENERATOR ROOM
PORCH NO 2
OFFICE NO. 2
WOMEN
CORRIDOR NO 2
MOTOR CONTROL CENTER
LUNCH/CONFERENCE ROOM
MEN
CORRIDOR NO. 3
OFFICE NO. 1
JAN
CORRIDOR NO. 1
CHEM STOR
CONTROL ROOM
LOBBY
LABORATORY
BACTERIOLOGICAL LAB
PORCH NO 1

PLAN

N.T.S.

Equipment and Supplies

The laboratory facilities should have the capability to run the tests listed in Table 10-1. In order to run these tests, the laboratory must be provided with the necessary equipment, supplies, and reagents. Discharge permit process control, and industrial waste monitoring requirements should be considered when specifying equipment needs. References such as Standard Methods and the EPA Analytical Procedures Manual should be consulted prior to specifying equipment items. In addition, the cost-effectiveness of optional equipment should be evaluated based on the following criteria:

- Frequency of tests.
- Sophistication of unit processes.
- Trade-offs between more or better-trained lab staff versus optimum equipment.
- Trade-offs between duplicating minimum equipment items versus replacement with fewer optimum equipment items.
- Flexibility to cope with unforeseen increases in testing requirements if optimum equipment is used.

Space Requirements

The size of the laboratory has been related to the capacity of the plant in curves developed by the EPA. For the initial phase of 10.0 MGD, the recommended laboratory size is 740 square feet. For the second phase capacity of 15.0 MGD, a size of 940 square feet is recommended. Due to the small increment of additional laboratory space required by the expansion, it is practical to provide sufficient area for the 15.0 MGD phase initially.

Similar curves were developed to estimate the required bench surface area and cabinet volume. For a 10.0 MGD facility, 30 percent of the lab floor space was recommended to be utilized as bench area. For a 15.0 MGD plant, the proportion was reduced to 27 percent. For cabinet volume, 5300 cubic feet were recommended for a 10.0 MGD facility and 6200 cubic feet for a 15.0 MGD plant.

TABLE 10-1

PROCESS SAMPLING AND TESTING NEEDS[a]

Test	Frequency	Method of Sample	Reason for Test
Temperature	1/day	Grab	Historical
pH	1/day	Grab	Historical
BOD	2/week	24-hr. Composite	Process
DO	3/week	Grab	Process
SS	3/week	24-hr. Composite	Process
Flow	Continuous	Continuous	Process
Total Solids	2/week	24-hr. Composite	Historical
Volatile Solids	2/week	24-hr. Composite	Historical
Dissolved Solids	2/week	24-hr. Composite	Historical
Alkalinity	2/week	24-hr. Composite	Historical
COD	2/week	24-hr. Composite	Historical
Heavy Metals (opt.)	2/month	24-hr. Composite	Historical
Fecal Coliform	Quarterly	----------------	Historical/ Regulatory
Specific Conductance	Continuous	Continuous	Historical/ Regulatory
Chlorides	Quarterly	----------------	Historical/ Regulatory

[a] Information derived from EPA Manual, Estimating Laboratory Needs for Municipal Wastewater Treatment Facilities.

In summary, the laboratory should be designed to accommodate the needs of the 15.0 MGD phase. A total laboratory area of 900-1000 square feet is recommended. As seen in Figure 10-1, this area is divided among the main laboratory, the bacteriological lab, and the chemical storage room. Bench surface area will constitute 250-300 square feet of the total laboratory area. A cabinet volume of about 6200 cubic feet is recommended.

Efficient Layout

Efficient laboratory operation depends largely on the physical layout of the laboratory. The physical includes such items as working area arrangement, the number and location of sinks and electrical outlets, the arrangement of laboratory equipment, and the materials of construction and lighting. The layout details can affect the accuracy of the laboratory tests. For example, tests that include identification of some colorimetric end point, such as heavy metals determinations, can be significantly impacted by the type of lighting and the finishes on laboratory facilities.

An excellent discussion of criteria for laboratory layout has been developed by the Michigan Water Pollution Control Association. These criteria have been included with the 1971 edition of the Ten States Standards and are summarized below:

- A northern exposure is preferred because it provides more uniform lighting.
- Adequate lighting should be provided. Color corrected fluorescent lighting is suggested.
- Wall and floor finishes should be nonglare and light in color. Flat finish type wall paint is suggested. Floor finishes should be of a single color for ease of locating small items that have been dropped.
- Floor covering, in addition to being nonglare, should be easy to clean and comfortable.
- Aisle width between work benches should be at least 4 feet. Also, adequate spacing should be provided around floor-standing equipment, workbenches, or file cabinets to facilitate cleaning.

- Storage space for reagent stock should be under workbenches. Reagent containers removed from storage areas under workbenches are less likely to be dropped than reagent containers removed from storage in the inconvenient and hard-to-reach areas above the work bench areas. Only those items that are infrequently used, or chemicals of a nondangerous nature, should be stored above workbenches. Strong acids or bases should never be stored out of the convenient reach of the laboratory personnel.
- One sink, large enough to wash laboratory equipment, should be provided for every 25 to 30 feet of bench length. One sink should be sufficient when total bench length is less than 25 feet. The minimum size of this sink should be 21-1/2 inches by 15-1/2 inches by 8 inches and it should be made of chemical resistant material. Cup sinks also should be provided at strategic locations on the bench surface to facilitate laboratory testing. They too should be made of chemical resistant material. The number of cup sinks depends largely on the type of tests that will be run. The general rule, however, is one cup sink for every 25 to 30 feet of bench length. These cup sinks should be alternated with the wash sinks at 12- to 15-foot intervals. Where workbench assemblies are provided in the center of the laboratory, a trough type sink down the center of the workbench may be provided in lieu of cup sinks. A hot and cold water tap should be placed at approximately 5 to 10 feet along the trough.
- Electrical receptacles should be provided at strategic points for convenient and efficient operation of the laboratory. Duplex type receptacles should be spaced at appropriate intervals along benches used for laboratory tests. Strip molding receptacles may be used.
- Gas fixtures also should be provided at convenient locations on the bench used for laboratory tests. One gas fixture should be provided for every 15-foot length of bench.
- Bench tops should be suitable for heavy duty work and resistant to chemical attack. Resin impregnated natural stone provides such a surface.
- Bench surfaces should be 36 inches high and 30 inches high, respectively, for work done from standing and sitting positions.

- Bench surfaces should be at least 30 inches wide.
- Equipment arrangement should be given special consideration in laying out the laboratory facility. Pieces of equipment used for making common tests should be in close proximity. For example, the drying oven used in making total, suspended and dissolved solids tests should be close to the muffle furnace for determining total volatile solids and volatile suspended solids from the samples dried in the drying oven. The drying oven and the muffle furnace should be near the balance table, because the balance is used in the weight determinations for the various solids tests.
- Safety is a prime consideration of a laboratory. The first aid kit, fire extinguisher, eye wash and emergency shower should be near the main working area of the laboratory. If the safety shower is not provided in a separate shower stall, a floor drain should be nearby.
- The analytical balance should be on a separate table. This table should be at least 30 inches in length by 24 inches deep. It should not transmit vibrations that would adversely affect the operation of the balance.
- A separate table should be provided for the microscope. This table should be about 30 inches long by 24 inches deep and 27 inches high.
- Fume hoods should be near the area where most laboratory tests are made.

10.3 OFFICES AND CONTROL ROOM

A wastewater plant office and a control room are required for monitoring plant functions and for storing plant records. The control room should be easily accessible to the operator. It should be located away from vibrating machinery or equipment which could have an adverse effect on recording equipment.

As seen in Figure 10-1, the control room is centrally located in the administration building near the main entrance. Two separate offices are also provided. The total area of the offices and the control room is approximately 400-500 square feet.

10.4 SANITARY FACILITIES

Toilet, shower, lavatory, and locker facilities should be provided for the projected number of operators. Two lockers per employee are preferred, one for street clothes and one for work clothes. The facilities should be furnished with hot and cold water, soap, and towels. Slop sinks and cleaning materials should be provided for general cleaning. Lavatories are frequently the long, shallow, wall-mounted type or the industrial circular type located in the center of the floor.

Figure 10-1 illustrates the sanitary facilities provided for men and women. The total area occupied by sanitary facilities, including the janitor's area, is approximately 300 square feet.

10.5 LUNCH/CONFERENCE ROOM

A combination lunch and conference room is recommended for the convenience of operating personnel. The eating room should be separated from other facilities. Basic kitchen equipment may be installed if desired. At a minimum, a refrigerator, a microwave, and sink should be provided. The 500 square foot lunch/conference room is located on Figure 10-1.

10.6 MOTOR CONTROL CENTER AND GENERATOR ROOM

A motor control center and generator room will be housed in the administration building, as seen in Figure 10-1. A detailed description of these facilities appears in Section 9. A total area of 1200 square feet is provided for these facilities.

10.7 MAINTENANCE BUILDING

A maintenance/storage area is often included as part of an administration building. However, there is an existing maintenance building on the plant site that will be used for the new treatment facility. The 1250 square-foot building was constructed in 1982. It may be utilized as a maintenance building, a machine shop, and a store room for spare parts, materials, and operating supplies. Four overhead doors provide access for large machinery and equipment. The existing maintenance building is expected to be adequate for the ultimate capacity of the plant.

SECTION 11
PLANT SITE LAYOUT AND MISCELLANEOUS IMPROVEMENTS

11.1 INTRODUCTION

Section 11 includes a site layout and a hydraulic profile of the plant, as well as a summary of the various miscellaneous improvements required. The section is organized as follows:

11.2 SITE LAYOUT

For purposes of this report, it has been assumed that the regional facility will be constructed at the site of the existing North Plant, for the following reasons:

- The North Plant site is located in an existing industrial/institutional land use area.
- The existing force main network for the East System indicates that the North Plant site is the logical location for the proposed facilities.
- Although the Gulfstream Plant site is more centrally located, the City of Plantation does not own that property or utility.

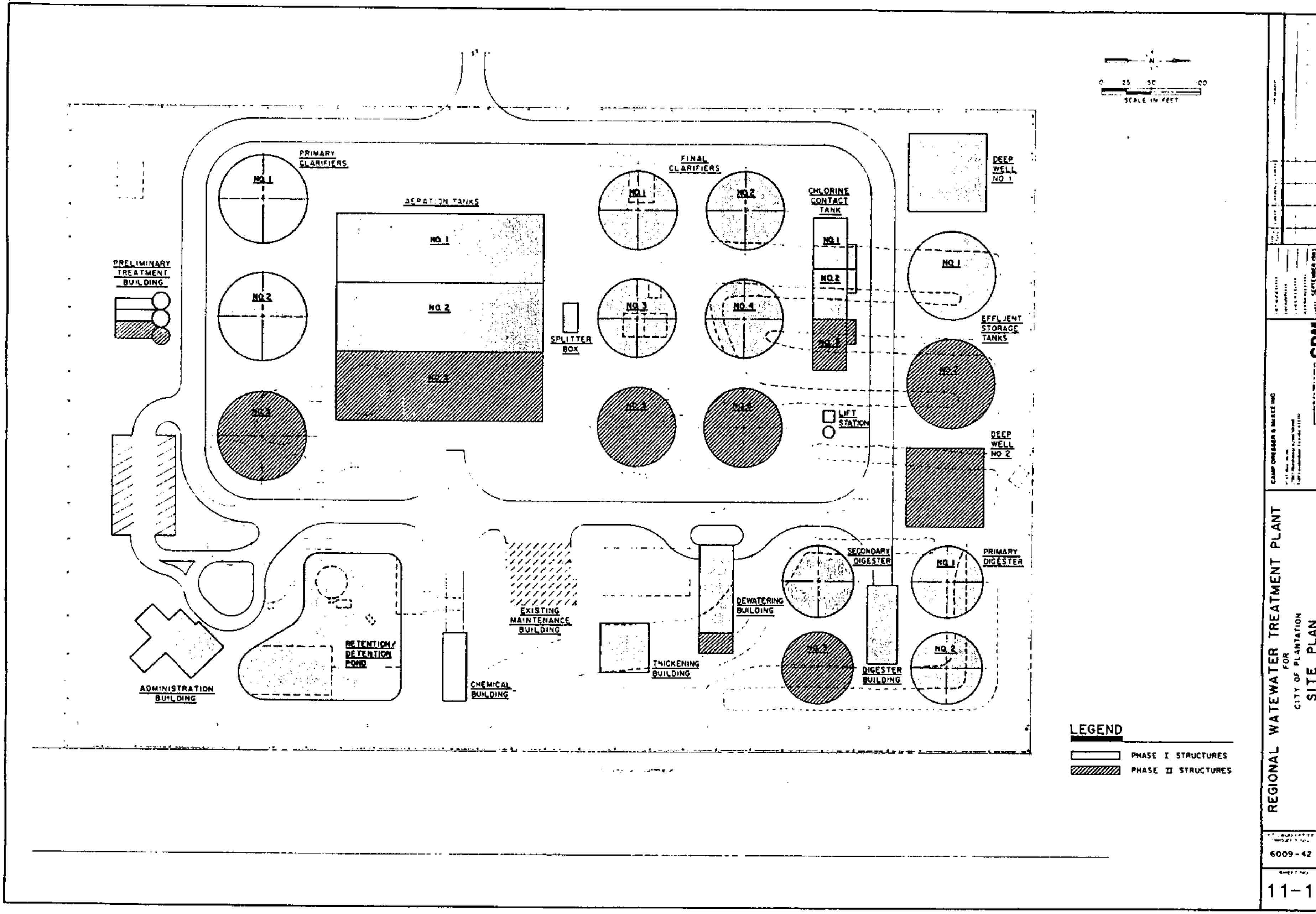
Site layout refers to the spatial arrangement of the physical facilities re quired to achieve a given treatment objective. The overall plant layout must incorporate administrative and maintenance buildings as well as the process units.

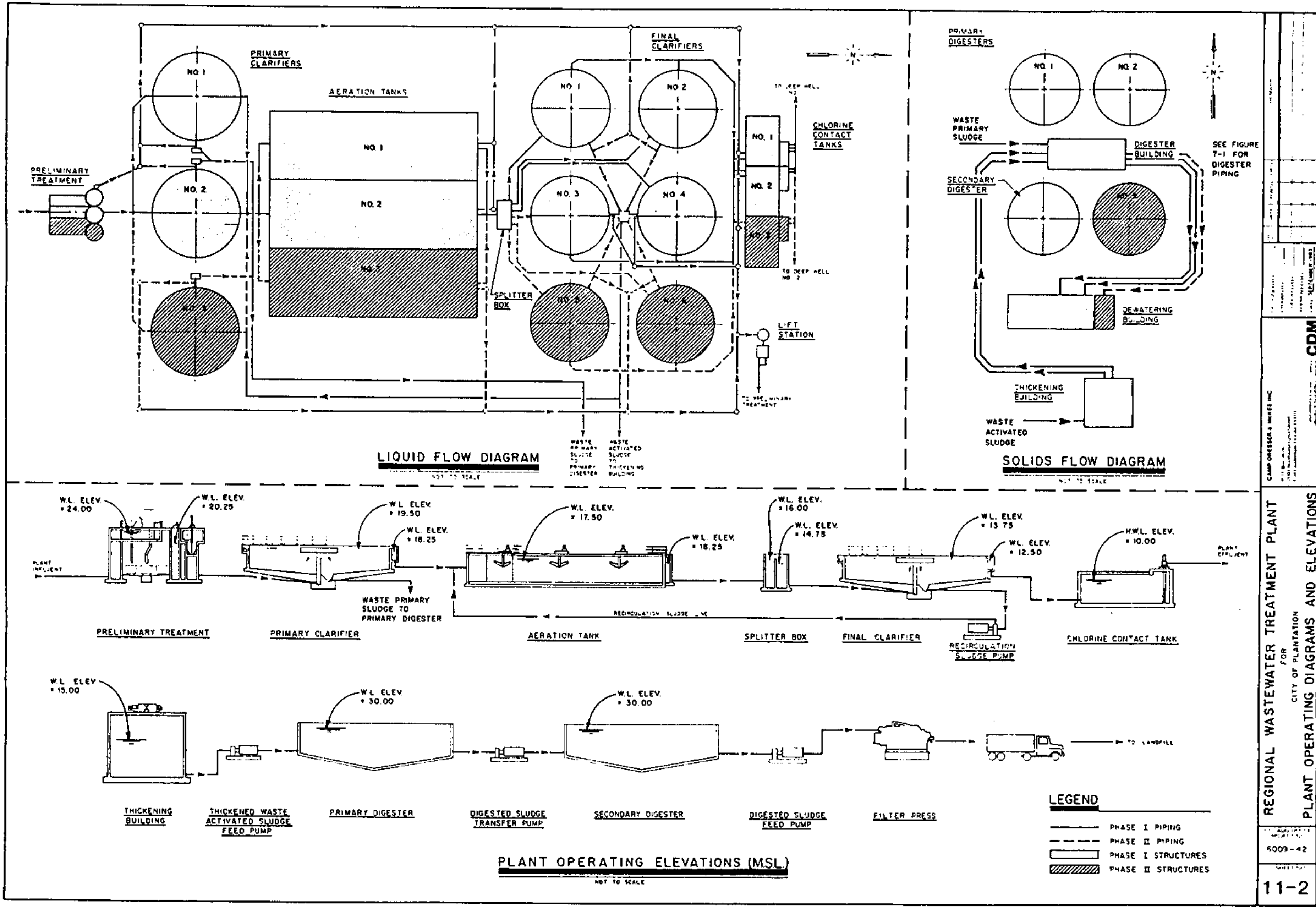
Among the factors that must be considered when laying out a treatment plant are the following:

- o Geometry of the plant site.
- o Topography.
- o Soil and foundation conditions.
- o Location of the influent force main.
- o Location of the point of discharge.
- o Transportation access.
- o Types of processes.
- o Effects of the length of process piping on treatment.
- o Process performance and efficiency.
- o Reliability, ease, and economy of operation.
- o Aesthetics.
- o Environmental control.
- o Additional area for future expansion.

In addition to these factors, the site layout was constrained by the existing plant, which must remain in operation during construction of the initial phase.

Figure 11-1 illustrates the proposed site layout for the three process trains. During construction of Phase I, which will consist of two process trains, the existing plant will remain operational. The polishing pond is the only part of the existing plant that must be taken out of service during Phase I construction. For Phase II, the existing plant will be removed, with the exception of the maintenance building.


SCALE IN FEET
PRIMARY CLARIFIERS
NO. 1
NO. 2
NO. 3
AERATION TANKS
NO. 1
NO. 2
NO. 3
FINAL CLARIFIERS
NO. 1
NO. 2
NO. 3
NO. 4
CHLORINE CONTACT TANK
NO. 1
NO. 2
DEEP WELL NO. 1
EFFLUENT STORAGE TANKS
NO. 1
NO. 2
DEEP WELL NO. 2
PRELIMINARY TREATMENT BUILDING
SPLITTER BOX
LIFT STATION
SECONDARY DIGESTER
PRIMARY DIGESTER
NO. 1
NO. 2
DEWATERING BUILDING
EXISTING MAINTENANCE BUILDING
THICKENING BUILDING
DIGESTER BUILDING
RETENTION/ DETENTION POND
CHEMICAL BUILDING
ADMINISTRATION BUILDING
LEGEND
PHASE I STRUCTURES
PHASE II STRUCTURES
CDM
REGIONAL WATEWATER TREATMENT PLANT
FOR
CITY OF PLANTATION
SITE PLAN
6009-42
11-1


LIQUID FLOW DIAGRAM
NOT TO SCALE
PRELIMINARY TREATMENT
PRIMARY CLARIFIERS
NO. 1
NO. 2
NO. 3
AERATION TANKS
NO. 1
NO. 2
NO. 3
SPLITTER BOX
FINAL CLARIFIERS
NO. 1
NO. 2
NO. 3
NO. 4
NO. 5
NO. 6
CHLORINE CONTACT TANKS
NO. 1
NO. 2
NO. 3
TO DEEP WELL NO. 2
LIFT STATION
TO PRELIMINARY TREATMENT
WASTE PRIMARY SLUDGE TO PRIMARY DIGESTER
WASTE ACTIVATED SLUDGE TO THICKENING BUILDING
N
SOLIDS FLOW DIAGRAM
NOT TO SCALE
PRIMARY DIGESTERS
NO. 1
NO. 2
WASTE PRIMARY SLUDGE
DIGESTER BUILDING
SEE FIGURE 7-1 FOR DIGESTER PIPING
SECONDARY DIGESTER
DEWATERING BUILDING
THICKENING BUILDING
WASTE ACTIVATED SLUDGE
PLANT OPERATING ELEVATIONS (MSL)
NOT TO SCALE
W.L. ELEV. + 24.00
W.L. ELEV. + 20.25
W.L. ELEV. + 19.50
W.L. ELEV. + 18.25
W.L. ELEV. + 17.50
W.L. ELEV. + 16.25
W.L. ELEV. + 16.00
W.L. ELEV. + 14.75
W.L. ELEV. + 13.75
W.L. ELEV. + 12.50
H.W.L. ELEV. + 10.00
PLANT INFLUENT
PLANT EFFLUENT
WASTE PRIMARY SLUDGE TO PRIMARY DIGESTER
RECIRCULATION SLUDGE LINE
PRELIMINARY TREATMENT
PRIMARY CLARIFIER
AERATION TANK
SPLITTER BOX
FINAL CLARIFIER
RECIRCULATION SLUDGE PUMP
CHLORINE CONTACT TANK
W.L. ELEV. + 15.00
W.L. ELEV. + 30.00
W.L. ELEV. + 30.00
TO LANDFILL
THICKENING BUILDING
THICKENED WASTE ACTIVATED SLUDGE FEED PUMP
PRIMARY DIGESTER
DIGESTED SLUDGE TRANSFER PUMP
SECONDARY DIGESTER
DIGESTED SLUDGE FEED PUMP
FILTER PRESS
LEGEND
PHASE I PIPING
PHASE II PIPING
PHASE I STRUCTURES
PHASE II STRUCTURES
CAMP DRESSER & McKEE INC.
CDM
REGIONAL WASTEWATER TREATMENT PLANT
FOR
CITY OF PLANTATION
PLANT OPERATING DIAGRAMS AND ELEVATIONS
5009-42
11-2

The advantages of ductile iron pipe include tight joints, ability to withstand relatively high internal design pressures and external loads, ductility, machinability, and toughness. The disadvantages are that it may be subject to corrosion by acid or highly septic wastewater and by corrosive soils. A cement lining with a bituminous seal coating is usually specified on the interior of the pipe.

If ductile iron pipe is used, it could be Class 51 thickness for the anticipated burial depth range, according to AWWA Standard C151. City standards require Class 51 pipe for normal use and Class 52 pipe under roadways. Since the plant construction will require the movement of heavy trucks and equipment, the use of Class 52 pipe may be required. The small difference in cost between Class 52 and 51 pipe may be justified due to the possible high live loads in the installation area.

11.5 ON-SITE GRAVITY COLLECTION SYSTEM

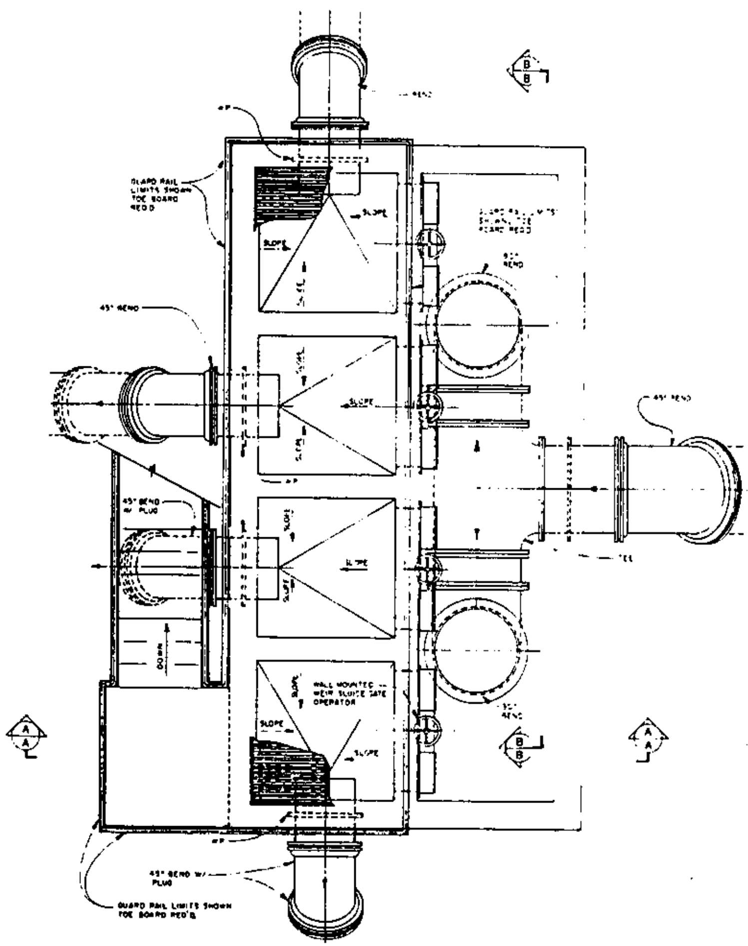
The gravity collection system for the site pump station will consist of ductile iron pipe with a minimum diameter of 12 inches. The site lift station will contain two submersible pumps rated at 700 gallons per minute (gpm) at 60 feet of head. The pump motors will be rated at 1750 rpm at 20 horsepower.

11.6 ON-SITE EFFLUENT REUSE SYSTEM

Plant effluent may be used for the following purposes:

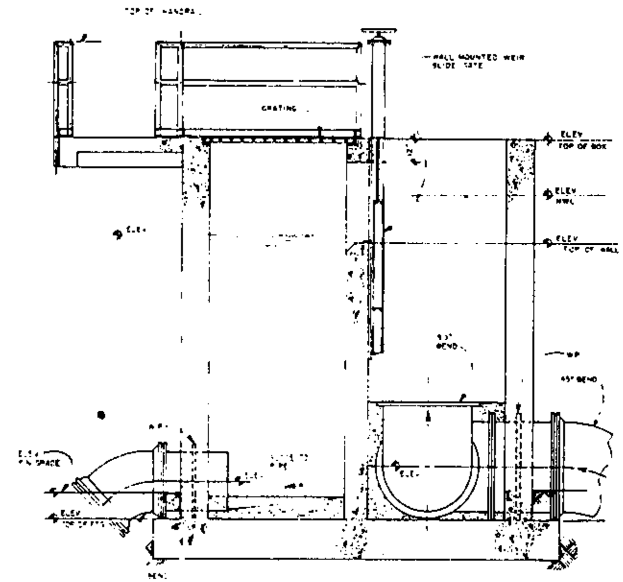
- Irrigation of landscaping.
- Seal water for pumping units.
- Washwater for belt filter presses.
- Flushing water for scum boxes.
- Hose bib connections for general maintenance.
- Chlorine injection water.

The effluent must first be filtered before it is used, and horizontal multiplex filters are recommended for this application. The filter consists of a horizontal vessel divided into several individual filter sections by vertical



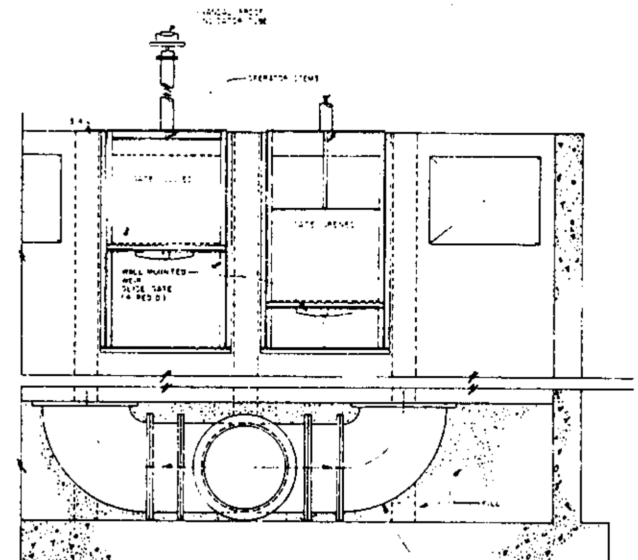
PLAN – SPLITTER BOX

NTS



SECTION A/A

NTS



SECTION B/B

NTS

CDM

REGIONAL WASTEWATER TREATMENT PLANT

FOR

CITY OF PLANTATION

SPLITTER BOX

6009-42

11-3

Figure 11-1 indicates the layout of the plant site road. The loop configuration promotes a smooth traffic flow. Access drives will be provided to the digester building, the dewatering building, and the chemical building to enable mobile cranes and other service vehicles to get close to these facilities for maintenance and removal or replacement of mechanical equipment. The road will be constructed with a 12-inch limerock subgrade and a 1.5-inch thick paved asphalt surface.

Parking facilities have been provided on the site for employees and visitors. The parking lot is located adjacent to the administration building for the convenience and control of visitors and delivery people.

11.10 SIDEWALKS AND WALKWAYS

Sidewalks will be provided for buildings and structures where a significant amount of pedestrian traffic is anticipated. All sidewalks will be installed at a grade which will minimize flooding problems during heavy rainstorms.

Elevated walkways will be provided above and between the treatment units, including suitably located access stairs, for the convenience of operating personnel.

11.11 DRAINAGE

In general, the finished grading will allow the rainstorm runoff to drain away from all buildings, structures, roads, and sidewalks. Culverts and drainage swales will divert excess rainfall to the retention/detention pond. The pond will be designed such that the runoff associated with the first inch of rainfall, which will be the most contaminated runoff, is completely retained in the pond. The runoff associated with the 25 year, 3-day rainfall event will be temporarily detained by the control structure located within the pond and released to the Holloway Canal.

11.12 LANDSCAPING AND IRRIGATION

A unified landscaping plan will be provided for the entire plant site, including trees, shrubs, and lawn area. The plan will be coordinated with the City's landscaping staff. Maximum use will be made of existing trees and shrubs at the plant site.

A sprinkling system will be provided for irrigation of the entire plant grounds. The irrigation network will be divided into suitable zones and will be furnished with an adjustable automatic timing control system. The irrigation water will be filtered effluent.

11.13 ODOR CONTROL

Odor control equipment will be provided for the following process units:

- Bar Screens
- Grit Collectors
- Primary Clarifiers
- Thickening Building

The basis of design is the two-stage wet scrubber system which is manufactured by PEPCON. This system is capable of reducing the hydrogen sulfide to less than one part per million. The system utilizes two different chemicals. Low level chlorine solution (sodium hypochlorite) is generated by passing a brine solution through electrolytic cells. Sodium hypochlorite solution is then dropped down through the first stage tower packing and interacts with the odorous air flowing in the opposite direction. The air then travels through a second scrubber which utilizes sodium hydroxide as a scrubbant. Before being exhausted, the air travels through a mist eliminator. For certain unit processes which produce highly malodorous off-gases, it may be necessary to provide activated carbon facilities for odor control.

SECTION 12
CONSTRUCTION COST ESTIMATE

12.1 INTRODUCTION

A construction cost estimate is developed herein for the 10 MGD phase of the proposed facilities. This estimate may be used to determine the cost-effectiveness of constructing the plant versus the other wastewater treatment and disposal options available to the City and Gulfstream. This section includes a summary of the facilities to be constructed and is organized as follows:

12.2 Scope of Work for 10 MGD Phase
12.3 Scope of Work for 15 MGD Phase
12.4 Cost Estimate

12.2 SCOPE OF WORK FOR 10 MGD PHASE

Sections 5 through 11 of this design report describe all of the facilities proposed for the 10 MGD phase of the treatment plant. The principal components of the proposed transmission, treatment, and disposal facilities are outlined below:

A. Transmission Facilities
   1. Gulfstream Force Main - 14,500 linear feet of 30-inch force main.
   2. Gulfstream Pumping Station - 16.8 MGD station with three Model 18530 pumps manufactured by the Clow Corporation, with variable-speed drives.
   3. South Pumping Station - 3.5 MGD station with two Model 8518 pumps manufactured by the Clow Corporation, with variable-speed drives.

B. Treatment Facilities

1. Liquid Treatment
   a. Preliminary Treatment Building - Two 6 mm bar/filter screens, two 20.8 MGD Pista grit collectors.
   b. Primary Clarifiers - Two 80' diameter, scraper-bottom clarifiers.
   c. Aeration Tanks - Two conventional activated sludge tanks with 325 horsepower installed in each tank.
   d. Flow Splitter Box - Divided into six compartments with an inverted slide gate in each.
   e. Secondary Clarifiers - Four 85' diameter rapid sludge withdrawal clarifiers.
   f. Chlorine Contact Tank - Two "around-the-bend" type tanks.
   g. Transfer Pumps - Three Model 12 HQR-H IMP. vertical turbine pumps.
   h. Effluent Pumps - Five Model 17 HQH vertical turbine pumps.

2. Solids Treatment
   a. Thickening - Two 210 gpm centrifuge thickeners.
   b. Digestion - Two 55' diameter primary digesters, one 55' diameter secondary digester, one digester building with associated pumping, heating, and other appurtenances.
   c. Dewatering - Two 2.0 meter belt filter presses, one dewatering building with associated polymer feed system and conveyance system.
   d. Chemical Building - Building to house evaporators, chlorinators, and other miscellaneous appurtenances.
   e. Administration Building - Building to house a laboratory, offices, a control room, sanitary facilities, a lunch/conference room, the motor control center, and the generator room.

3. Miscellaneous - Includes plant piping, the on-site gravity collection system, on-site effluent reuse system, flow

splitter boxes, scum handling system, access road and parking, sidewalks and walkways, drainage, and landscaping and irrigation.

C. Disposal Facilities

1. Equalization Tank - One 1.0 MG prestressed concrete tank.
2. Deep well - One 24" injection well.

12.3 SCOPE OF WORK FOR 15 MGD PHASE

Sections 5 through 11 indicate the additional facilities that will be required to expand the treatment plant from 10 MGD to 15 MGD. The principal components of the transmission, treatment, and disposal facilities are outlined below. An allowance for demolition of the existing facilities must also be included.

A. Transmission Facilities - Impellers and motors in the Gulfstream Pumping Station will be replaced with larger units.

B. Treatment Facilities

1. Liquid Treatment
   a. Preliminary Treatment Building - One additional 6 mm bar/filter screen, one additional Pista grit collector.
   b. Primary Clarifier - One additional 80' diameter, scraper-bottom clarifier.
   c. Aeration Tank - One additional conventional activated sludge tank with 325 horsepower installed.
   d. Flow splitter box - No additions required.
   e. Secondary Clarifiers - Two 85' diameter rapid sludge withdrawal clarifiers.
   f. Chlorine Contact Tank - One additional "around-the-bend" type tank.
   g. Transfer Pumps - One additional Model 12 HQR-H IMP. vertical turbine.
   h. Effluent Pumps - One additional Model 17 HQH vertical turbine pump.

2. Solids Treatment
   a. Thickening - One additional 210-gpm centrifuge thickener.
   b. Digestion - One additional 55' diameter primary digester.
   c. Dewatering - One additional 2.0 meter belt filter press.
   d. Chemical Building - No additions required.
   e. Administration Building - No additions required.

3. Miscellaneous - Additions to some of these facilities will be required.

C. Disposal Facilities
   1. Equalization Tank - One additional 1.0 MG prestressed concrete tank.
   2. Deep well - One additional 24" injection well.

12.4 COST ESTIMATE

A construction cost estimate has been developed for transmission, treatment, and disposal facilities for the 10 MGD phase. These costs are listed in Table 12-1. An allowance for contingencies of 10 percent has been added to the construction costs. The cost of related services, such as engineering, administration, and legal costs, has been estimated at 15 percent. An allowance for inflation has been added at the rate of 1/2 percent per month until the projected bid date in the last quarter of 1984.

A detailed cost estimate has not been prepared for the Phase II improvements due to several factors. A realistic cost estimate projected to a bid date in 1990 would appear to be unreasonable in comparison with present construction costs, due to the effect of inflation for approximately seven years. In addition, the costs associated with Phase II should not be distributed only to present customers. The allocation of Phase II costs should be among the increased customer base that will be available at the time of Phase II construction. It should be noted that the incremental cost for the Phase II expansion will be at a lower unit cost than Phase I. An estimated project cost of approximately 40 percent of the Phase I costs could be expected.

TABLE 12-1

PROJECT COST ESTIMATE FOR 10 MGD PHASE

Item	Cost
A. Transmission Facilities	
1. Gulfstream Force Main	$ 1,015,000
2. Gulfstream Pumping Station	1,650,000
3. South Pumping Station	935,000
SUBTOTAL	$ 3,600,000
B. Treatment Facilities	
1. Liquid Treatment	
a. Preliminary Treatment Building	$ 600,000
b. Primary Clarifiers	700,000
c. Aeration Tanks	1,290,000
d. Flow Splitter Box	80,000
e. Secondary Clarifiers	1,380,000
f. Chlorine Contact Tank	565,000
g. Transfer Pumps	25,000
h. Effluent Pumps	575,000
2. Solids Treatment	
a. Thickening Building	975,000
b. Digesters	1,080,000
c. Digester Building	515,000
d. Dewatering Building	700,000
3. Miscellaneous	
a. On-site Lift Station	95,000
b. Chemical Building	45,000
c. Administration Building	475,000
d. Retention/Detention Pond	25,000
e. Yard Piping	600,000
f. Paving	150,000
g. Landscaping	85,000
h. Effluent Reuse and Irrigation	115,000
i. Electrical	900,000
j. Instrumentation	200,000
k. Miscellaneous (sitework, taxes, etc.)	815,000
l. Demolition	150,000
m. Recirculation Sludge Pump Station	100,000
4. Overhead and Profit @ 15%	1,836,000
SUBTOTAL	$14,076,000
C. Disposal Facilities	
1. Equalization Tank	$ 325,000
2. Deep Well	2,600,000
3. Miscellaneous	25,000
SUBTOTAL	$ 2,950,000

TABLE 12-1
(Continued)

Item	Cost
Contingencies (10%)	$ 2,062,600
Related services - engineering, administration, legal (15%)	3,093,900
Inflation - (7%, equal to 1/2% per month until bid date)	1,443,800
GRAND TOTAL PROJECT COST	$27,226,300

SECTION 13
FINANCIAL ASPECTS

13.1 INTRODUCTION

This section examines the financial aspects of the recommended wastewater plan. The following specific issues are addressed herein:

13.2 Financing options, including revenue bonds, general obligation bonds, the state bond loan program and others.
13.3 Impact of the plan on sewer rates and cost to a typical customer.
13.4 Typical large user charges for Gulfstream Utility Company.
13.5 Summary.

13.2 FINANCING OPTIONS

There are several financing options available to the City. These options are discussed below.

Water and Sewer Revenue Bonds

Long-term municipal revenue bonds can be sold to raise the required capital funds. Such debt would be backed by the pledge of system revenues, including a rate covenant in the bond resolution that requires user charges to be sufficient to generate the requisite cash needs. In addition, when the implementing entity possesses taxing powers, such as in the case here, the bonds may be further backed by a pledge of excise taxes, but still using net revenues as the source of funds to make debt service payments.

The City of Plantation presently has outstanding the "Water and Sewer Refunding and Improvement Revenue Bonds, Series 1978", which were issued in the principal amount of $18,475,000. These bonds were issued to defease previously issued revenue bonds. Generally, it is considered that the use of such bonds is the most practical and realistic way in which to fund the construction of utility projects such as is recommended here.

Certain pari passu requirements must be fulfilled before the City may issue additional revenue bonds on a par with the outstanding revenue bonds. The bond covenants state that the annual average pledged revenues for the full fiscal year of 12 months out of the preceding 24 months immediately preceding the date of sale of the proposed additional bonds must equal at least 1.00 times the maximum bond service requirement on all 1978 Bonds outstanding and the proposed additional bonds. Furthermore, the average annual pledged revenues plus the excise taxes (if still pledged) for the above described 12 month period must equal 1.25 times the maximum bond service requirement on all 1978 Bonds outstanding and the proposed additional bonds.

Municipal General Obligation (G.O.) Bonds

The G.O. bonds sold by the implementing entity are typically secured by payments from ad valorem taxes imposed to generate the necessary monies in addition to the revenues generated from the use of the system. However, the ultimate guarantee of the repayment of the debt is from the full faith and credit pledge of the implementing municipality.

The use of G.O. bonds may not be considered as desirable an alternative as revenue bonds. Municipalities tend to utilize revenue bond financing when there is an assured stream of revenues that can be pledged to the debt repayment. In addition, there are referendum requirements and debt ceilings associated with proposed G.O. debt issues.

In some cases, G.O. bond issues may be established with the actual source of funds for debt service payments being from both net revenues and from capacity charges. This alternative permits the good interest rates of G.O. bonds to be obtained and provides the security of pledged ad valorem taxes to the bondholder, but capacity charges and rates would be set so that the City would not have to fund debt service payments from taxes.

State of Florida Pollution Control Bond Program

State of Florida Full Faith and Credit Pollution Control Bonds can be utilized for funding the capital costs of eligible pollution control facilities. Municipal wastewater treatment facilities are eligible projects.

From the municipality's perspective, the debt obligation would resemble a revenue bond. The municipality must pledge revenues from rates and charges from system operations for the repayment of the debt, and it must generate revenues sufficient to support a 1.33 coverage of the debt service each year. Although coverage requirements for State Pollution Control Bonds are greater than for the City's revenue bonds, lower interest rates are typically available for State Pollution Control Bonds. At such time as the City is preparing to issue additional debt, the relative merits of revenue bonds and State Pollution Control Bonds should be examined more closely with the assistance of a fiscal advisor to establish a financial package.

Federal Grant Funds

The EPA Federal Construction Grants program provides grant contribution to defray a major portion of the capital costs for wastewater treatment plants. However, for the foreseeable future, it is not anticipated that grant funds will be available for the recommended plan. The reason is that existing grant allocations are oriented more toward wastewater systems currently causing adverse environmental impacts. Although it is assumed herein that all capital costs must be borne by the City, the City should make every effort to pursue federal grant funds.

Lease Financing

As a result of the two most recent comprehensive tax laws, there has been a significant increase in the interest in alternative sources of capital funds to pay for capital intensive municipal projects. One such source is the use of leasing financing, where a lease structure is used for paying for new facilities because other traditional capital sources are not available to the municipality. In certain leasing situations, where the full tax advantages of private party ownership can be utilized, leasing can result in a reduction of costs below those typically enjoyed through municipal (tax-exempt) financing.

The use of lease financing would require the resolution of significant legal issues, such as whether the lease commitment is to be recognized as long term debt of the municipality and, thus, subject to its debt limits. A more extensive feasibility study would be required to determine if lease financing is possible. Thus, at this time, lease financing is not considered practical.

Private Equity/Industrial Development Bonds

It is possible that a private party would implement, own and operate the desired facilities. In fact, as a result of the reduction in Federal grant support, there has been a corresponding increase in interest in the private ownership of wastewater treatment plants. In Florida, private ownership of facilities is well established; many small treatment facilities are owned and operated by private parties.

In the case of private ownership, the financing would be effected through an equity contribution by the private party and, presumably, through the sale of tax exempt industrial development bonds. However, the practicality of such an alternative would require a much more extensive feasibility study. Thus, for this project, private ownership is viewed to be impractical.

Capital Contribution/User Fees

Theoretically, capital funds can be acquired through accumulation of additional charges on existing user fees and through capacity charges to be paid by developers or new residents. In the case of an existing wastewater treatment service, user charges could be increased and the resulting funds could be accumulated to be utilized for the construction of the proposed additional facilities. In addition, the proceeds of capacity charges could be accumulated in the wastewater capacity fund.

Some qualifications are necessary regarding the accumulation of funds to pay for capital costs of the Phase I facilities. Because of the time required to accumulate funds, such funds will not pay for a significant amount of the required capital costs; thus, they would have to be utilized in combination with other means of acquiring capital funds. Furthermore, it is expected that rates for the City service area will be increased in order to generate the necessary level of net revenues in advance of additional long-term capital financing.

Based upon the capital cost estimates for the Phase I facilities, wastewater system capacity charges can be calculated to recover the cost for each new equivalent residential unit (ERU). The 10 MGD capacity of the Phase I facilities is expressed on the basis of average daily flow in the maximum month. The 10 MGD is equivalent to 8.33 MGD of annual average daily flow. Thus, the total capacity cost per gallon per day of flow is $3.27 (= $27,226,300/8.33 MGD). Since 270 gallons per day, annual average daily flow, is attributable to each ERU, the wastewater system capacity charge of $882.90 per ERU (= $3.27 x 270), say $880 per ERU, is calculated. It will be assumed that wastewater capacity charges of $880 per ERU could be accumulated beginning January 1, 1984 to be used to help defray the total capital cost of Phase I facilities in April 1985.

In addition to the accumulation of wastewater capacity charges from new connections, a surcharge of perhaps 10 percent, for example, could be placed upon the monthly wastewater bills of East System customers beginning in January 1984. These surcharges could then be accumulated for the purpose of paying a portion of the capital cost of the Phase I facilities. The preliminary Utilities Department budget for Fiscal Year 1984 projects $1,720,000 in wastewater revenues. Thus, a 10 percent surcharge would generate approximately $172,000 in 1984. The amount of $172,000 for surcharges is assumed to be received also in 1985.

There are some cases in which revenue bond issues are secured by a lien upon capacity charges received by the system. The City of Orlando is contemplating a large bond issue that would be serviced by pledged revenues comprised of capacity charges, utility taxes and net revenues. Such an approach may be available to the City if a refunding bond issue were used, but is not ble for additional parity bonds under the bond covenants of the 1978 Bonds.

State Wastewater Grant Program

The State of Florida recently enacted the Water Quality Assurance Ac Section IX of this act established a Wastewater Grant Program. program is initially being funded at a level of $100,000,000. Of funds available, 45 percent have been earmarked for the Small Commu Grant Program intended for communities with populations of less t persons.

Communities desiring to obtain funding from this program must subm application in order to be placed on the priority list. Since m this grant program are available, the City should make every effort this source of funding.

Utilities Department Funds On-Hand

The Utilities Department at present has approximately $5,000,000 in it considers available for expenditure on the wastewater plan. Alth diture of these monies may reduce the overall impact of the cost o the resulting loss of interest income may necessitate an increas water system rates and charges.

Bond Anticipation Notes (BAN's)

Bond Anticipation Notes can provide an important interim source o construction of the proposed wastewater facilities.

BAN's have several advantages as indicated below:

1. BAN's may need to be used initially as an interim measur of the above long-term financing methods could be used.

2. BAN's permit deferring the assumption of long-term debt until the actual total cost of the project is known.

The BAN's could be issued by the start of construction in April 1985, and all or a portion of the interest capitalized until final completion of the first phase of construction in October 1986. The BAN's and associated capitalized interest could then be paid off by debt issued in November 1986. An increase in wastewater rates could then conceivably be deferred until November 1985. The period from November 1985 to November 1986 would be the qualifying period for meeting the 125 percent coverage on existing plus proposed bonds.

Summary

The foregoing subsections discuss alternative financing methods. For the purposes of this report, it is assumed that the major source of financing would be from additional revenue bonds issued on a par with existing revenue bonds, with assistance from the accumulation of wastewater system capacity charges and perhaps other revenues from surcharges. It is assumed that no grant funds would be received. If, however, grant funds do become available, these funds would reduce the charges shown subsequently in this section.

13.3 IMPACT OF THE PLAN ON SEWER RATES AND COST TO A TYPICAL CUSTOMER

This subsection presents the impact of the plan on sewer rates, based upon estimates for accumulated funds, capital costs with related debt service, and operation and maintenance costs. The effect on rates is then translated into the cost to a typical customer.

Sources and Uses of Funds

Possible sources of funds projected to be accumulated and on-hand are shown in Table 13-1. By April 1985, approximately $500,000 could have been accumulated, consisting of $483,400 of capacity charges and wastewater surcharges, plus interest earnings on these monies. The City may also elect to use

TABLE 13-1

SOURCES OF ACCUMULATED FUNDS FROM PLANTATION EAST SYSTEM

TO DEFRAY PORTION OF PHASE I CAPITAL COSTS

Year	New Wastewater ERU's (1)	Capacity Charges (2)	Revenues from 10% Surcharge	Total Revenues
1984	244	$ 214,720	$ 172,000	$ 386,720
1985	244	53,680 (3)	43,000 (3)	96,680
TOTAL	488	$ 268,400	$ 215,000	$ 483,400

(1) Based upon population projections from Table 3-1.

(2) Using $880.00 per new wastewater ERU.

(3) Funds accumulated from January through March 1985 prior to beginning construction in April 1985.

$5,000,000 of investments currently on-hand, which, together with the $500,000 of accumulated funds, would provide $5,500,000 of "up-front" funds to defray a portion of the total Phase I capital costs.

It is conceivable that Gulfstream might wish to front-end a portion of the capital costs of Phase I facilities. As discussed subsequently in Section 13.4, Gulfstream would be allocated 46.9 percent of the capacity of the Phase I wastewater facilities, using the reserved-capacity basis for cost allocation. If it assumed that Gulfstream would match the up-front funds provided by Plantation under the cost allocation percentages of 46.9 percent for Gulfstream and 53.1 percent for Plantation, Gulfstream's matching funds can be calculated for various assumed levels of up-front funding by Plantation. For the case in which Plantation front ends $500,000 of monies, Gulfstream's matching amount would be $441,620. If Plantation contributes $5,500,000 of up-front monies, Gulfstream's matching share would be $4,857,815.

The other source of funds to meet the remaining amount of total monies required for the capital cost of Phase I facilities is assumed to be from the proceeds of another revenue bond issue.

Uses of funds would be solely to pay the total capital costs of Phase I facilities. Monies generated from possible surcharges and wastewater system capacity charges beyond April 1985 could be accumulated to help defray the cost of the Phase II facilities.

Cost Estimates

Capital Costs. Phase I construction of the 10 MGD wastewater treatment plant with deep well, pumping stations and force mains is assumed to take place from April 1985 to October 1986. The total capital cost of Phase I is estimated as $27,226,300.

Operation and Maintenance (O&M) Costs. Under the current configuration of wastewater treatment and disposal, there are three separate facilities providing these services. With the recommended plan, there would be only one wastewater treatment and disposal facility.

Economies of scale in operation would be achieved at a single facility as compared with three separate facilities, primarily as a result of the reduction of fixed costs related to lower overall staffing levels. From recent work done by CDM concerning economies of scale in O&M expenses, these economies of scale can be roughly estimated. Currently the average O&M cost for each of the three treatment and disposal facilities, with related major transmission facilities, is estimated as $0.59 per 1,000 gallons. For a single large facility treating and disposing of the aggregate wastewater processed by the three separate facilities, the current O&M cost is estimated as $0.55 per thousand gallons. Applying estimates of inflation to these unit costs, the average O&M cost for three separate facilities is estimated as $0.68 per thousand gallons in 1985; whereas, for the single facility, 1985 O&M costs are estimated at $0.63 per thousand gallons. Thus, a $0.05 per thousand gallon savings in 1985 is estimated by utilizing the single larger facility.

It is important to note that as a result of inflation, O&M costs will increase in the future. By continuing to operate three separate facilities, O&M expenses are estimated to increase by $0.09 per thousand gallons, from $0.59 to $0.68. If, however, the recommended facilities are constructed, the O&M cost would increase only by $0.04 per thousand gallons, from $0.59 to $0.63. Thus, more than half the cost of inflation is estimated to be offset by the effects of economies of scale.

Debt Service Costs. In order to estimate the impact of the recommended plan upon sewer rates, it is necessary to assume a method for financing these improvements. For the purposes of this report, it is assumed that additional revenue bonds of a parity with the outstanding Series 1978 Bonds would be the major source of financing for the recommended wastewater plan. Other sources of funding to be considered are investments of the water and wastewater system currently on-hand and accumulated reserves.

For the case in which the $27,226,300 is financed totally from the issuance of additional revenue bonds, the total increase in annual principal and interest cost would be $3,013,449. Using estimated interest earnings on the debt service reserve fund to help pay for principal and interest, the additional annual amount required from operating revenues equals $2,742,239. Table 13-2

TABLE 13-2

ANNUAL PRINCIPAL AND INTEREST COST FOR ADDITIONAL BONDS TO FINANCE PHASE I CONSTRUCTION: TOTAL COST OF $27,226,300

Assume:

- Revenue bonds for 30 years at 10% interest (Capital Recovery Factor = 0.1061).
- $27,226,300 of total capital costs including bond issuance costs, to be funded from bond proceeds.
- No capitalized interest during construction.
- A debt service reserve fund to be funded from bond proceeds equal to one annual principal and interest payment.
- Interest income on the average balance in the construction fund from bond proceeds at 9% over the 18 months of construction.
- Interest income of 9% earned on the debt service reserve fund is used to offset annual principal and interest payments.

Let P = Principal amount of additional bonds required.

$\therefore$ - Debt service reserve = 0.1061P

- Interest income on construction fund = 1.5 (0.09)($27,226,300/2)

$\therefore$ P = $27,226,300 + 0.1061P - 1.5 (0.09)($27,226,300/2)

0.8939P = $25,388,525

P = $28,401,974

With principal amount of bonds = $28,401,974, annual Principal and Interest (P&I) related to these additional bonds is calculated:

Total P&I = 0.1061 X P = 0.1061 X $28,401,974 = $3,013,449

Since estimated income earned on the debt service reserve can be used to pay for P&I, annual P&I required from operating revenues is calculated:

P&I from Operating Revenues = 0.1061P - (0.09)(0.1061P)

= $3,013,449 - (0.09)($3,013,449)

= $2,742,239

shows the assumptions and calculations used to determine this annual cost. The covenants of the Series 1978 Bonds require that revenues, in conjunction with excise taxes, be sufficient to generate a 25 percent coverage on debt service. Since excise taxes are expected to be sufficient in themselves to meet the 25 percent coverage requirement on existing plus additional debt, revenues need only be adequate to pay 100 percent of debt service from the standpoint of the rate covenant. Because it is necessary to have monies available for renewal and replacement requirements related to the recommended wastewater facilities, however, the rates should generate revenues in excess of 100 percent of debt service. Bond covenants also require that 5 percent of gross revenues of the previous fiscal year be deposited each year into the Renewal and Replacement (R&R) Fund. Thus, the $3,013,449 of total additional revenues for principal and interest required would also result in the need for $150,672 (= 5% x $3,013,449) to be generated from revenues for deposit in the R&R Fund. The total increase in wastewater revenues therefore must be $2,892,911 (=$2,742,239 + $150,672). Although R&R needs related to the new facilities would likely be minimal during the first several years of service, the monies generated for R&R can be accumulated and used at such time as it becomes necessary.

It is possible that up-front monies may be received to reduce the amount of the bond issue required, as discussed previously in this section. Table 13-3 presents examples of various amounts of up-front monies that might be available to reduce additional debt requirements.

The above discussion of debt service costs assumed that interest on the bonds would not be capitalized during the 18 month construction period. This assumption implies that debt service payments on the bonds would commence shortly after issuance of the bonds from net revenues generated by existing customers. Thus, the necessary increases in the wastewater rates to support additional debt would be required as early as one year prior to issuance of the bonds in order to accrue the monies to make the first debt payment. The alternative would be to capitalize interest during the construction period. Capitalization of interest would defer the timing of the needed rate increase until shortly before issuance of the bonds. By capitalizing interest, however, the principal amount of the bonds would be increased. For the case in

TABLE 13-3

ADDITIONAL DEBT REQUIREMENTS AND DEBT SERVICE COST FOR VARIOUS LEVELS OF "UP-FRONT" MONIES USED FOR PHASE I CAPITAL COSTS

Plantation "Up-Front" Monies										
Capacity Charges	10% Wastewater Bill Surcharge	Investments On Hand	Gulfstream "Up-Front" Monies	Total "Up-Front" Monies	Proceeds Required From Additional Bonds(1)	Principal Amount of Additional Bonds(2)	Total Additional Annual Principal & Interest Requirements(3)	Additional Annual R&R Requirements(4)	Net Additional Annual Principal & Interest Requirements(5)	Sum of Net Additional P&I Plus R&R Requirements(6)
$ -0-	$ -0-	$ -0-	$ -0-	$ -0-	$27,226,300	$28,401,974	$3,013,449	$150,672	$2,742,239	$2,892,911
268,400	215,000	-0-	-0-	483,400 (say $500,000)	26,726,300	27,880,383	2,958,109	147,905	2,691,879	2,839,784
268,400	215,000	5,000,000	-0-	5,483,400 (say $5,500,000)	21,726,300	22,664,876	2,404,743	120,237	2,188,316	2,308,553
268,400	215,000	-0-	441,620	925,020 (say $941,620)	26,284,680	27,420,178	2,909,281	145,464	2,647,446	2,792,910
268,400	215,000	5,000,000	4,857,815	10,341,215 (say $10,357,815)	16,868,485	17,597,204	1,867,063	93,353	1,699,028	1,792,381

(1) Equals $27,226,300 minus "Total 'Up-Front' Monies."

(2) Calculated similarly as shown in Table 13-2.

(3) Calculated similarly as shown in Table 13-2.

(4) Equals 5% of "Total Additional Annual Principal and Interest Requirements."

(5) Equals "Total Additional Annual Principal and Interest Requirements" less estimated interest income on debt service reserve fund.

(6) Equals "Net Additional Annual Principal and Interest Requirements" plus "Additional Annual R&R Requirements."

which the $27,226,300 is financed totally from the issuance of additional revenue bonds, the principal amount of the additional revenue bonds is estimated as $33,671,784. Total annual principal and interest requirements related to this additional debt is approximated as $3,572,576. Projected annual interest earnings on the debt service reserve would reduce the amount required from wastewater revenues to $3,251,044. A 5 percent allowance for R&R deposit requirement would equal $178,628, bringing the total annual debt service plus R&R requirements to $3,429,673 (= $3,251,044 + $178,628).

Impact of the Plan on Sewer Rates

Table 13-3 presented examples of the additional annual principal and interest plus R&R requirements that would be needed. It is assumed that these would be recovered from increases in the wastewater service rates. Based upon population projections developed in Section 3, in 1985 there are projected to be 2,419,950 thousand gallons (TG) of wastewater from both the East System and Gulfstream's System. Thus, for the case in which an additional $2,892,911 must be recovered for principal and interest plus R&R deposits, the increase in cost per 1,000 gallons of wastewater is $1.20 ($2,892,911/2,419,950 TG). Because of the economies of scale achieved in operations, O&M cost would increase only by $0.04 per thousand gallons, yielding an overall increase of $1.24 per thousand gallons. With a typical wastewater service customer billed for 8,000 gallons per month, the increase in the monthly wastewater bill would be $9.92. Table 13-4 presents typical increases in costs per 1,000 gallons for the examples of other levels of debt service plus R&R deposit requirements given in Table 13-3. Also shown are the increases in the average monthly bill of a wastewater service customer billed for 8,000 gallons, including the increased O&M.

As discussed previously, for the case in which interest is capitalized during construction and there are no up-front monies provided, additional annual principal and interest plus R&R deposit requirements would equal $3,429,673. The increased cost per 1,000 gallons of wastewater thus equals $1.42 ($3,429,673/2,419,950 TG). Adding the $0.04 per 1,000 gallons increase in O&M

costs, the increase in the wastewater bill to a customer billed for 8,000 gallons per month would equal $11.68. This amount is comparable to monthly increase of $9.92 for the case in which interest is not capitalized.

In two of the cases presented in Tables 13-3 and 13-4, it was assumed that $5,000,000 of investments on-hand would be used as a portion of up-front monies. Using these investments would result in a loss of interest income. Assuming a 9 percent interest rate on these investments, the annual loss of interest income would equal $450,000. Wastewater rates would probably need to be increased to compensate for all or a portion of this revenue loss. Thus, up to an additional $0.19 per thousand gallons (= $450,000/2,419,950 TG) would be required in the wastewater rate.

The values discussed above are calculated for 1985, when Phase I of the facilities would be placed into service. Greater revenues for wastewater capacity charges are projected beginning in 1986. Also, billable wastewater gallonage would increase annually after 1985. Thus, with a fixed annual debt service cost, the debt related cost per thousand gallons beyond 1985 would decrease. Practically, however, the charge per thousand gallons to the wastewater service customer in the years following 1985 may not be reduced because of the effects of inflation on operating costs.

13.4 TYPICAL LARGE USER CHARGES FOR GULFSTREAM UTILITY COMPANY

Basis for Calculating Large User Charges

There are two different bases considered here for calculating typical large user charges for Gulfstream. These bases are discussed below.

Reserved Capacity Basis. The reserve capacity basis for calculating large user charges is similar to that used by Broward County in allocating cost to large users of the North District Regional Wastewater system. Gulfstream and Plantation would each be allocated a portion of the capacity of the recommended transmission, treatment and disposal facilities. The allocation would be based upon the two utilities' respective percentages of total projected wastewater flow in some future year. The year 1991 would be a desirable year

TABLE 13-4

ADDITIONAL COST PER THOUSAND GALLONS OF WASTEWATER

FOR EXAMPLES GIVEN IN TABLE 13-3

Sum of Net Additional P&I Plus R&R Requirements(1)	Debt Plus R&R Related Increase In Cost Per 1,000 Gallons (2)	O&M Cost Increase Per 1,000 Gallons	Increase In The Bill of A Typical Customer Billed For 8,000 Gallons(3)
$2,892,911	$1.20	$0.04	$9.92
2,839,784	1.17	0.04	9.68
2,308,553	0.95	0.04	7.92
2,792,910	1.15	0.04	9.52
1,792,381	0.74	0.04	6.24

(1) Taken from Table 13-3.
(2) Equals "Additional P&I Plus R&R Requirements" divided by 2,419,950 thousand gallons. Does not include the cost of O&M expenses.
(3) Equals "Debt Plus R&R Related Increase In Cost Per 1,000 Gallons" plus $0.04 per 1,000 gallons for O&M cost increases, all multiplied by 8,000 gallons.

for allocation of reserve capacity ("reserve capacity year"), since it marks the end of the period for which the capacity of the Phase I facilities is adequate.

Equal-Cost-Per-Customer Basis. The philosophy underlying this basis for cost allocation considers that all customers should incur the same cost per thousand gallons of flow regardless of whether they live in Gulfstream's service area or in the East System. This basis would thus treat all customers within the City equally.

Example Large User Charges Using the Reserved Capacity Basis

Based upon the population projections given in Table 3-1, wastewater flow from the Gulfstream service area would equal 46.9 percent of the total in 1991. Gulfstream would therefore be allocated 46.9 percent of the debt service and R&R costs related to the Phase I facilities.

Although it is possible that Gulfstream might have a large user charge for debt service and R&R related costs levied on a fixed monthly charge basis, large user charges are expressed on a gallonage basis herein for ease of comparison.

For the case in which $27,226,300 are required from bond proceeds for construction of Phase I facilities, total annual principal and interest plus R&R deposit requirements are $2,892,911. Allocating 46.9 percent of this amount to Gulfstream yields $1,356,775 to be recovered annually from Gulfstream. Assuming that all wastewater generated within the Gulfstream service area is measured by a master meter, the total amount of wastewater attributable to Gulfstream in 1985 would be 952,650 thousand gallons. The large user charge to recover the debt and R&R capital related cost would be $1.42 per thousand gallons ($1,356,775/952,650 TG). In addition to the $1.42, the cost of O&M would be recovered through the gallonage charge. The O&M cost is estimated as $0.63 per thousand gallons in 1985, which would bring the total large user charge to $2.05 per thousand gallons. Offsetting the $0.63 per thousand gallon increase for O&M in the large user charge would be an estimated reduction in Gulfstream's O&M costs by $0.68 per thousand gallons. Thus, the net increase to Gulfstream would be $1.37 per thousand gallons in 1985.

Large user charges using the reserved capacity basis can be calculated for the other examples of various up-front funding levels presented in Table 13-3. These large user charges are calculated to give credit to Plantation and Gulfstream for their respective amounts of up-front monies provided. Table 13-5 presents large user charges corresponding to the up-front funding levels shown in Table 13-3.

Typical Large User Charges Using the Equal-Cost-Per Customer Basis

In this subsection, the large user charge to Gulfstream is calculated using the equal-cost-per-customer basis. Again it is assumed that Gulfstream would be billed on the total gallonage it generates as measured by a master meter.

For the equal-cost-per-customer basis, only one example large user charge is calculated. This charge is calculated for the case in which no up-front monies are contributed by either Plantation or Gulfstream. Total wastewater flow from both Plantation and Gulfstream in 1985 is projected as 2,419,950 TG. With the sum of net additional debt service and R&R deposit requirements of $2,892,911 in 1985, the gallonage charge of $1.20 per TG is calculated. This charge does not include the cost of operation and maintenance, estimated as $0.63 per thousand gallons, which would bring the total large user charge up to $1.83 per 1,000 gallons.

The equal-cost-per-customer basis could also be applied to examples in which various levels of up-front funding are used. Such a basis would require negotiations between Plantation and Gulfstream concerning the amount each entity would put up in order that the customers of one system do not subsidize those of the other.

TABLE 13-5

EXAMPLES OF LARGE USER CHARGES TO GULFSTREAM IN 1985

UNDER THE RESERVED CAPACITY BASIS

Total Up-Front Monies From Plantation	Total Up-Front Monies From Gulfstream	Annual P&I Plus R&R Requirements Allocated to Gulfstream Total (1)	$/TG	O&M Cost ($/TG)	Total Gulfstream Large User Charge ($/TG) (2)(3)
$ -0-	$ -0-	$1,356,775	1.42	0.63	2.05
500,000	-0-	1,356,775	1.42	0.63	2.05
5,500,000	-0-	1,356,775	1.42	0.63	2.05
500,000	441,620	1,309,875	1.37	0.63	2.00
5,500,000	4,857,815	840,627	0.88	0.63	1.51

(1) Allocated to Gulfstream on the basis of 46.9 percent of "Sum of Additional P&I plus R&R Requirements" from Table 13-3, adjusted for respective amounts of up-front monies contributed by Plantation and Gulfstream.

(2) Equals "Annual P&I plus R&R Requirements Allocated to Gulfstream" plus "O&M Cost."

(3) Gulfstream would experience an O&M cost savings from discontinuation of its facilities estimated as $0.68 per 1,000 gallons. Thus, net increase to Gulfstream would be $0.05 per 1,000 gallons less than the unit "Annual P&I Plus R&R Requirements Allocated to Gulfstream" shown.

13.5 SUMMARY

There are several different alternatives for obtaining funds to finance the recommended plan. While it may be difficult for the City to obtain grant funding, this source of external financing should be pursued vigorously by the City. It is likely that the major portion of funding for the recommended facilities would be from issuance of additional debt. Such new debt would be supported by net revenues generated from the system, and would require an increase in wastewater rates. Accumulation of funds, such as from capacity charges and possibly from a wastewater bill surcharge, could lessen the amount of additional debt required. Also, the use of investments on-hand by Plantation and perhaps Gulfstream, could decrease the size of an additional debt issue.

Depending upon the amount of additional debt needed to be incurred, the impact of the recommended plan on sewer rates could vary from an increase of $0.74 per 1,000 gallons to $1.20 per 1,000 gallons for principal and interest plus R&R deposit requirements. By implementing the recommended plan, economies of scale in operations would result, and the O&M cost is estimated to increase only by $0.04 per 1,000 gallons between now and 1985. Alternatively, the full effect of inflation would cause an estimated $0.09 per 1,000 gallon increase in O&M costs by 1985 if three treatment and disposal facilities were retained. Thus, the overall net increase in debt and O&M costs could range from $0.78 per 1,000 gallons to $1.24 per 1,000 gallons. For a wastewater customer billed for 8,000 gallons per month, the increase could vary between $6.24 and $9.92 per month.

Two bases have been considered to develop the large user charge for Gulfstream. Under the reserved capacity basis, the charge to Gulfstream related to principal, interest and R&R deposits could vary from $0.88 per 1,000 gallons to $1.42 per thousand gallons. To these charges must be added the cost of O&M, estimated as $0.63 per 1,000 gallons. Offsetting the large user charge, however, is an estimated reduction of $0.68 per 1,000 gallons in Gulfstream's O&M costs.

The costs discussed in this section reflect combined regional facilities serving all of the City of Plantation, including the area served by Gulfstream Utility Company. An alternative discussed by Gulfstream is for that utility to proceed with a separate system improvement. It is important, therefore, that there be a valid "oranges and oranges" comparison of the costs, financing alternatives, cost impacts and institutional aspects of the plan detailed in this design report with those of the Gulfstream "private" program. Only in this way can the question of Gulfstream proceeding jointly with the City or on its own be addressed.

SECTION 14
SUMMARY AND RECOMMENDATIONS

14.1 INTRODUCTION

This preliminary design report has been prepared in order to provide a basis for comparing the alternative of constructing a joint treatment facility for Plantation and Gulfstream versus the other available wastewater treatment and disposal options. This section summarizes the major conclusions of the report and sets forth recommendations for future action. Organization of the section is as follows:

- 14.2 Implementation Schedule
- 14.3 Summary
- 14.4 Recommendations

14.2 IMPLEMENTATION SCHEDULE

The timing for construction of this facility is critical, due to the requirement of no discharge to surface waters by mid-1986. If the decision is made to serve Plantation and Gulfstream with this new facility, the implementation schedule found in Figure 14-1 must be followed in order to meet the no discharge requirement.

The implementation schedule allows a month for client review and a month for regulatory agency review of this report. Preliminary design is scheduled to begin November 1, 1983. Bids will be received in the last quarter of 1984, with a Notice to Proceed to be issued in March 1985. The construction period is 18 months, with beneficial occupancy in July 1986 and final completion in September 1986.

14.3 SUMMARY

This design report addresses the preliminary design of a wastewater treatment facility to jointly serve Plantation and Gulfstream. The major conclusions of this report are summarized as follows:

CITY OF PLANTATION

DESIGN AND CONSTRUCTION SCHEDULE FOR
TRANSMISSION AND TREATMENT FACILITIES

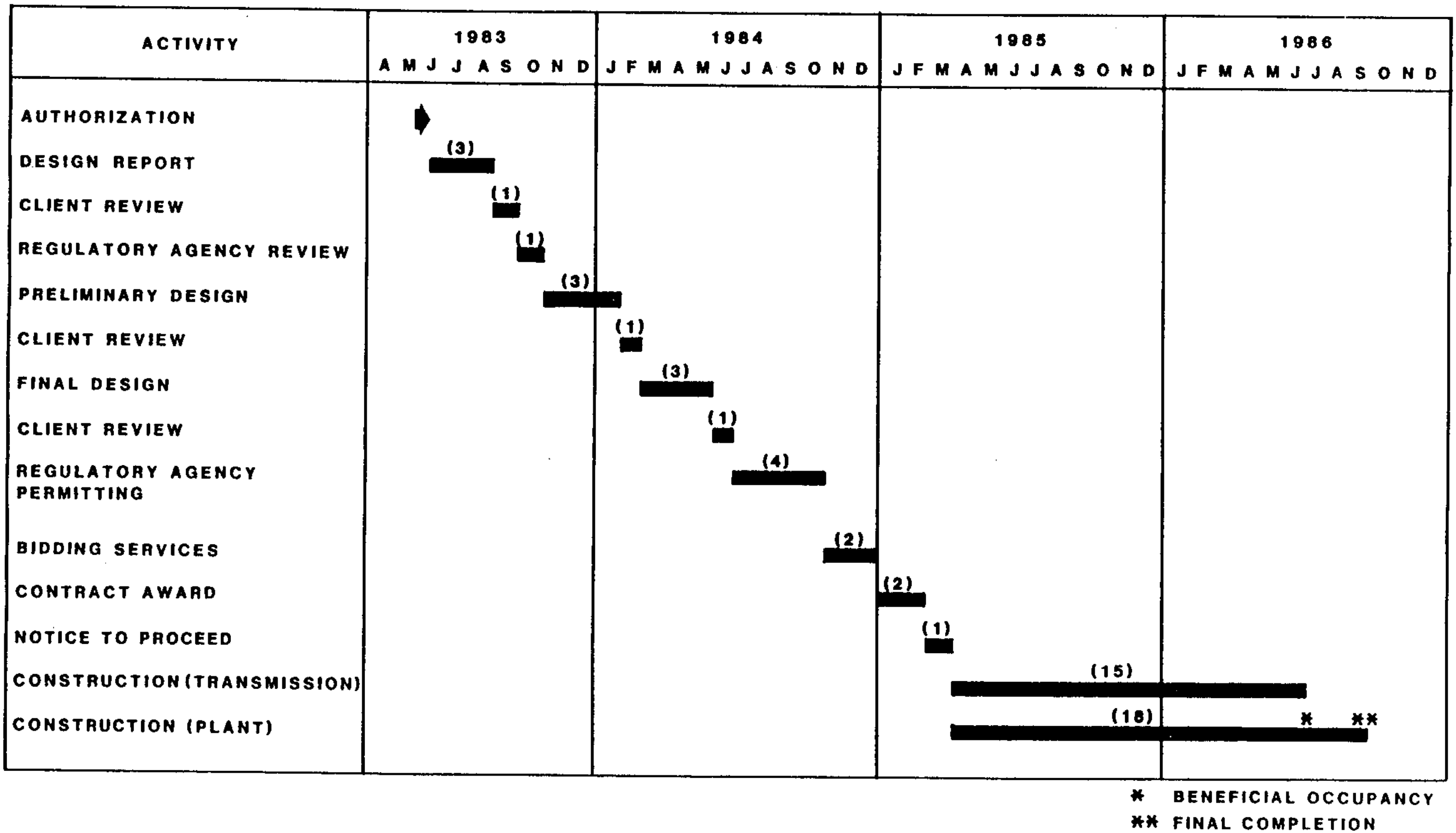


FIGURE 14-1

CDM

- Projected wastewater flows for the study area were 7.9, 11.4, and 14.9 MGD for the years 1985, 1995, and 2005, respectively.
- The recommended staging program is to build a 10 MGD facility in 1986, which will be expanded to 15 MGD in 1991.
- The total rated capacity of the existing facilities is 8.5 MGD, and the projected flow in 1986 is 8.2 MGD. Therefore, the available treatment capacity will be sufficient during the interim period.
- Gulfstream and South treatment plants will be abandoned. A 16.8 MGD pumping station will be constructed at Gulfstream to pump wastewater through a proposed 30-inch force main. A 3.5 MGD pumping station will be constructed at the South site. An existing 16/20-inch force main is sufficient to transport flows from the South service area.
- The conventional activated sludge treatment process was found to be more cost-effective for the proposed facility than the extended aeration process.
- Conventional sludge treatment and landfill disposal was found to be more cost-effective than the "innovative" alternative of sludge composting.
- A two-well system for deep well disposal is recommended, with one well to be constructed in each phase.
- A total construction cost of $27,226,300 has been estimated for the 10 MGD phase of the proposed facility.
- An additional user fee in the range of $6.24 to $9.92 per month was determined to be the impact on a typical residential customer.

14.4 RECOMMENDATIONS

From this design report, several recommendations have been developed, as follows:

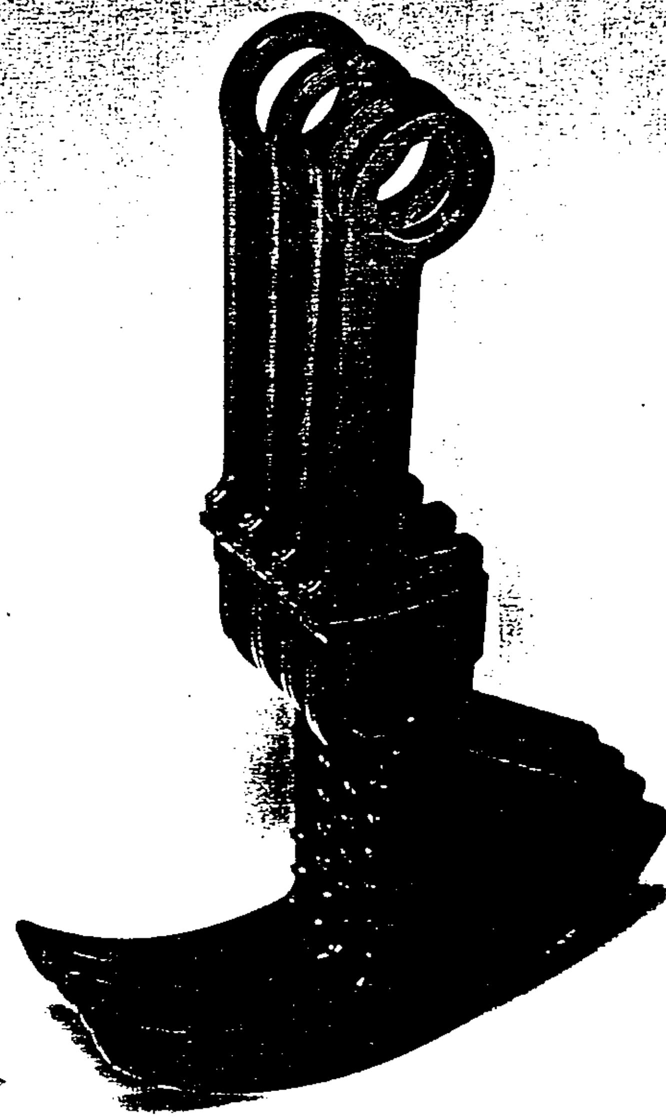
- The City Council may utilize the findings of this report to compare the option of a joint treatment facility with the other wastewater treatment and disposal options available.
- The City should continue to pursue a definite course of action for eliminating existing surface water discharges presently occurring within the City limits.

- The City should file the appropriate applications for state and federal funding of the facilities.
- A detailed staffing analysis should be conducted for the proposed facility to determine operating staff requirements.

Appendices

APPENDIX A

AQUA GUARD SCREEN



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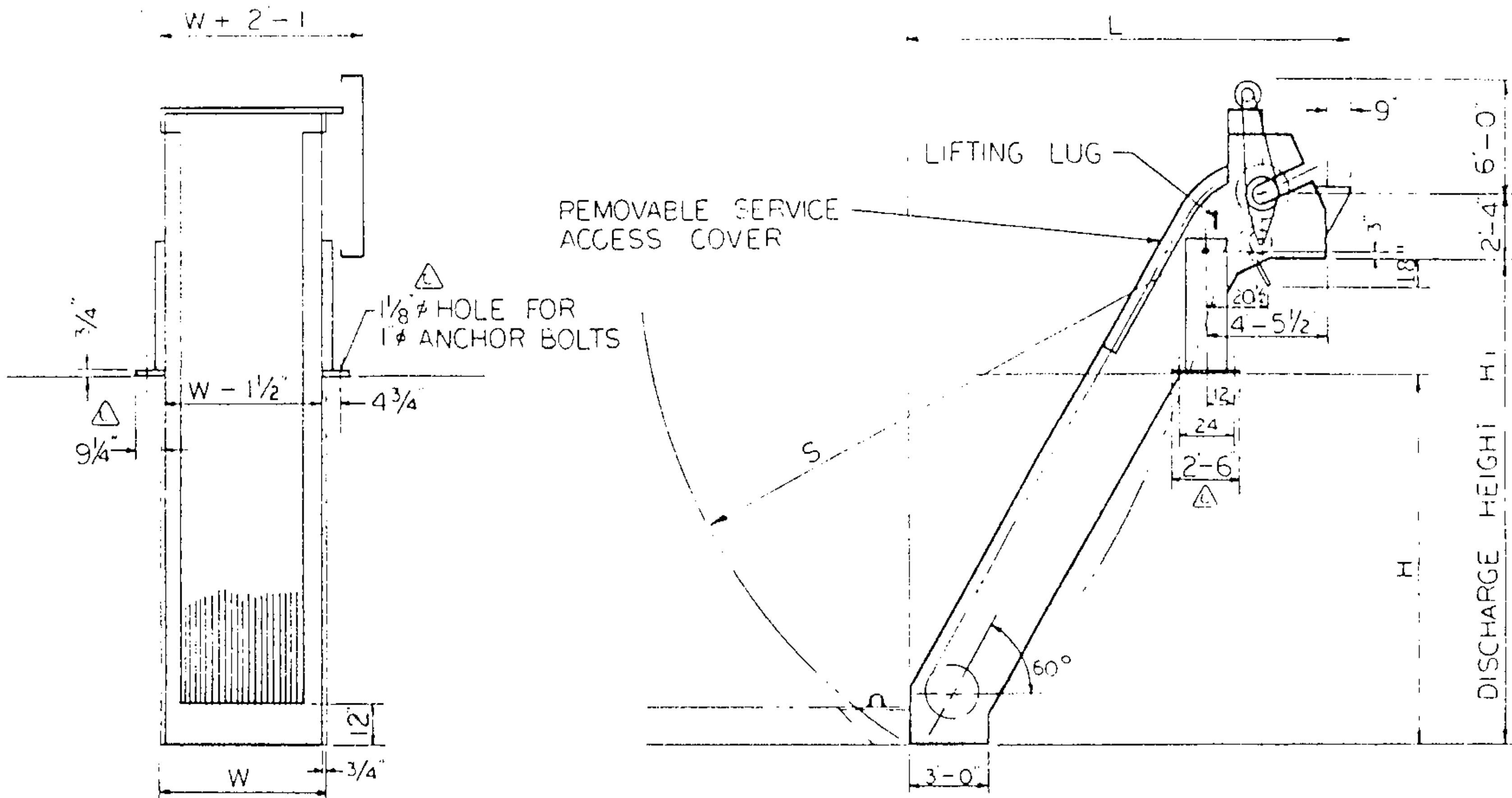

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QUOTATION *

W	4'0"
H1	12'0"
H	6'0"
L	13'1"
S	14'7"

FOR REFERENCE ONLY – NOT FOR CONSTRUCTION

Rev	Ind.	Revision/Change Note No.	Date	Rev. by	Zone
D		IN TITLE BLOCK "S" WAS "3500"	REV 11-17-82		
C		ADDED DIM. & NOTE	3-11-82		
B		REVISED L	2-19-82		1C
A		ADDED DISCHARGE PAN	12-1-81		1C

Item No	Qty	Description	Reference	Material	Remarks	Std. Ref.	Zone

Drawn 1-12-81 | Checked | Std | Approved | Scale | Preceding Drw | Superceding Drw

Parkson CORPORATION

Title: AQUA - GUARD AG-S-A

Reg | Micro | Date | Code

Drw No IA - 409 | Rev D

DCI-7508

BRUNING 44 [illegible]

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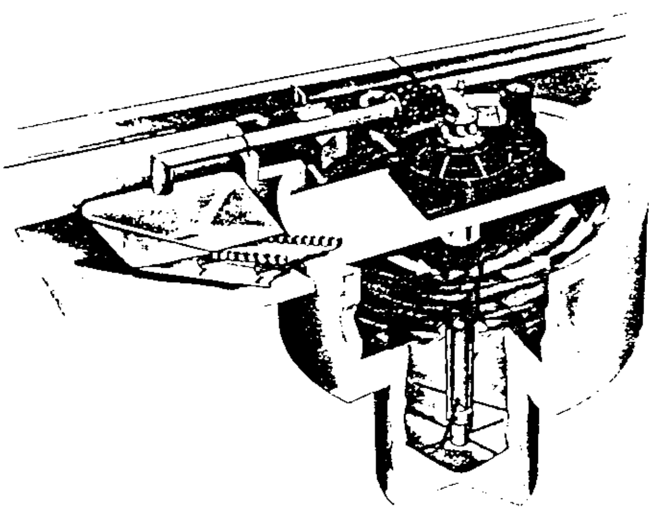
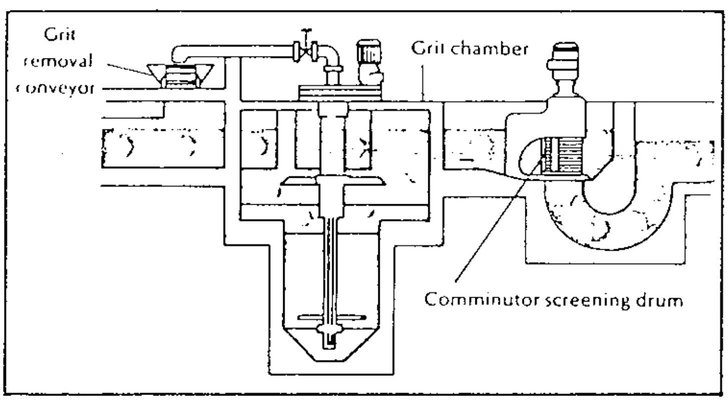
Smith & Loveless, Inc.

S-101

Pista® Grit Chamber





Optional Features

Grit handling equipment can be a screw conveyor or classification chamber, depending upon transportation and disposal facilities available at the treatment system.

Bridge can be either concrete or steel. Chamber is available in steel as a completely factory-built unit, up to 12 MGD. Larger sizes are usually concrete.

S&L Grit Chamber Design Criteria

Model Number	Maximum Flow (MGD)	Paddle Motor Horsepower	Blower HP
2.5	to 2.5	0.5	2
4	2.5-4.0	0.5	2
7	4.0-7.0	0.75	3
12	7.0-12.0	1.0	3
20	12.0-20.0	1.5	5
30	20.0-30.0	2.0	5
50	30.0-50.0	3.0	5

Advantages of a Grit Chamber

Grit chambers are installed because grit, due to its settling characteristics, will settle in aeration tanks and clarifiers, clogging hoppers and sludge outlets. The presence of excess grit makes sludge handling difficult, as well as decreasing the effective size of the treatment compartment where it settles. In addition, the presence of grit in the wastewater flow can drastically reduce the service life of the pumps, comminutors and valves, as well as increase maintenance time and cost.

A grit chamber should be designed to settle out grit particles down to approximately 100-mesh size. Generally grit smaller than 60 to 70 mesh will not materially affect machinery life, so a grit removal system must be designed for removal to 100-mesh size to maintain an acceptable overall efficiency.

Advantages of Pista Grit Chamber

The Pista Grit Chamber, combining the design features of a gravity separator and an aerated grit chamber, removes grit from the waste flow without loss of head, using a minimum of space and without being affected by variable hydraulic loading. The special Pista design provides the required retention time for grit to settle, yet maintains a constant velocity of flow through the chamber to keep the organic particles in suspension. The design also has no chains, buckets, bearings or drive components submerged in the chamber where abrasion and corrosion could cause rapid wear.

Operation of the Pista Grit Chamber

The waste stream enters the unit, is circulated around the chamber by the specially designed paddles, and exits into the downstream channel. The paddles are attached to a drive torque tube which is driven by a clarifier-type bull gear and helical-gear drive unit. The paddles maintain a controlled hydraulic regime in the settling chamber independent of the hydraulic flow to the grit chamber. Grit settles out of the waste flow, is moved towards the center of the unit, and settles into the lower collection hopper. At intervals, which must be determined for each particular application, the collected grit is air-scoured to remove any settled organics and air-lifted out of the hopper to the discharge system. Grit discharge can be manual or automatic based on time or flow to the unit.

TYPICAL INSTALLATIONS

FIGURE 2

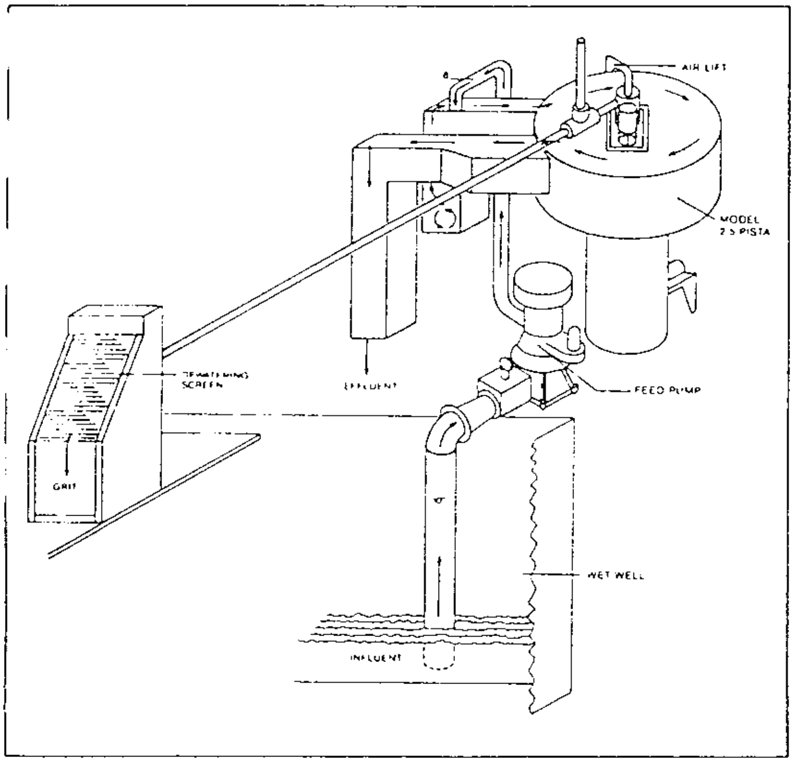


FIGURE 3

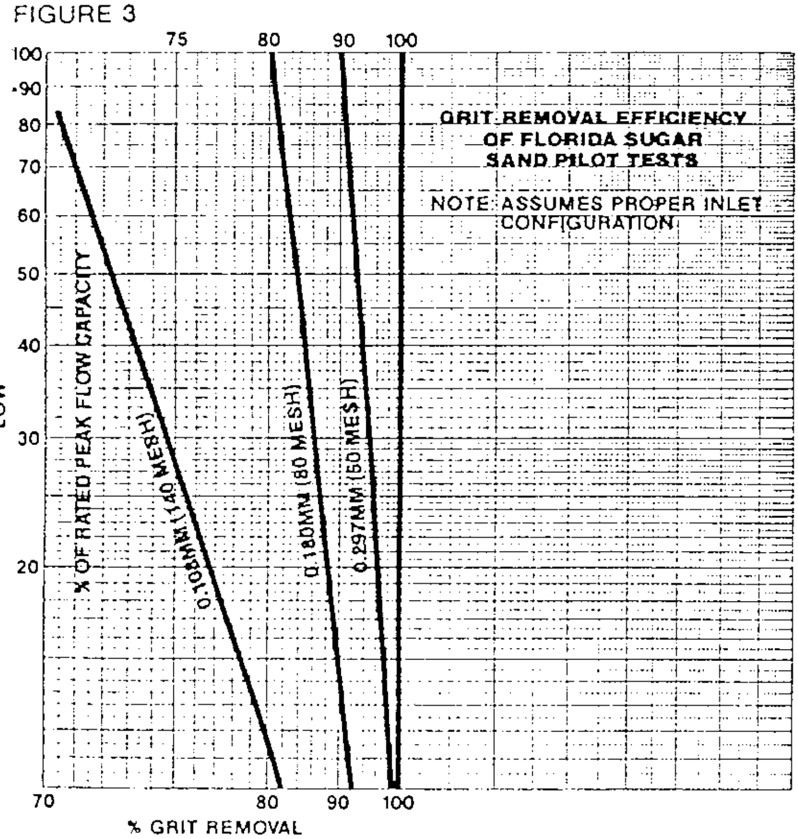


FIGURE 4

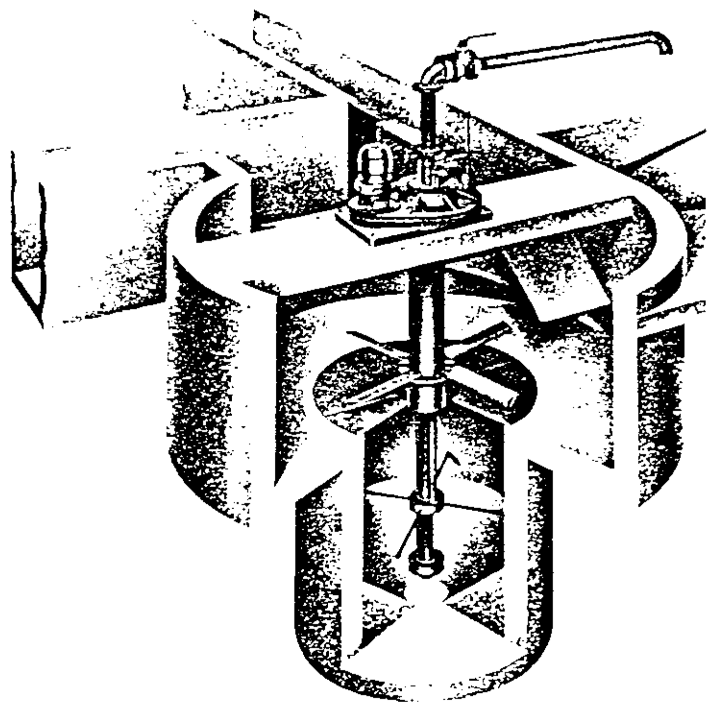
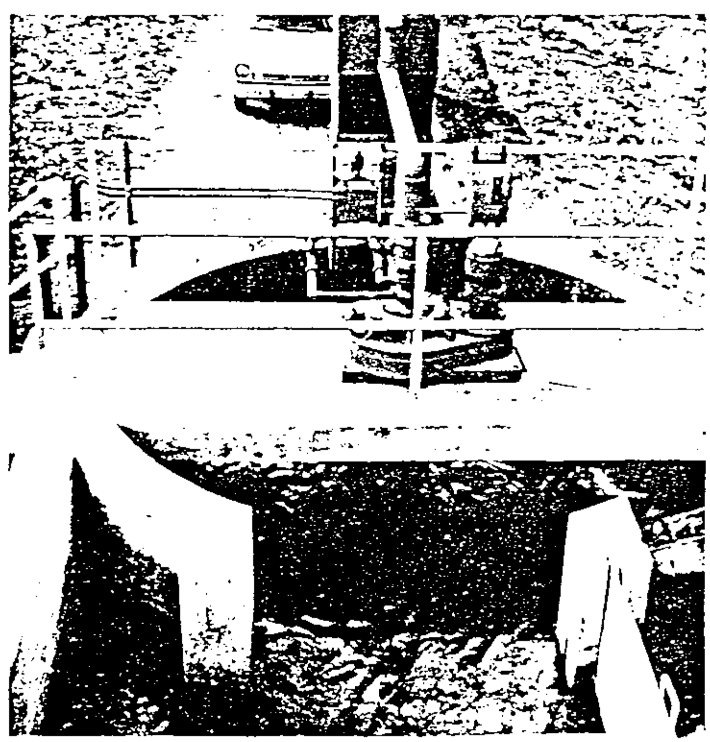


FIGURE 5



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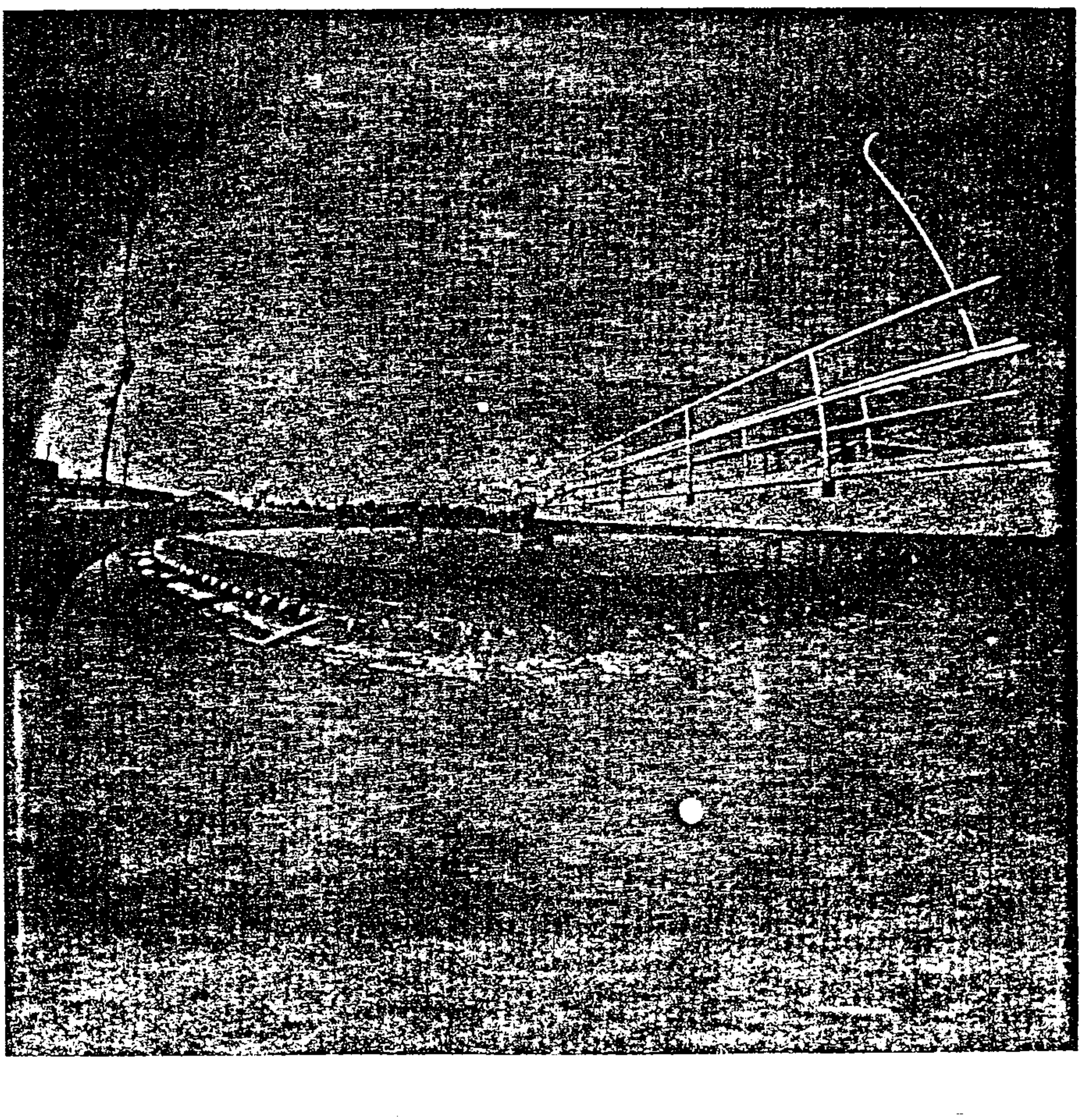
MUNICIPAL EQUIPMENT DIVISION

Clarifiers From EIMCO



C2

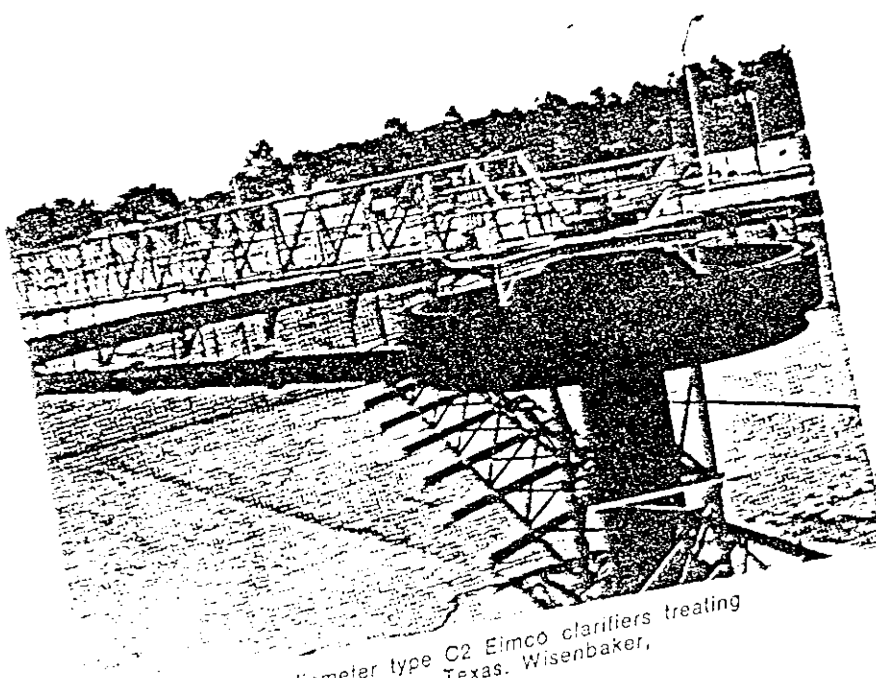
FOR LARGER TANKS: THE TYPE C-2 MECHANISM —

Suitable for tanks up to 20 meter, the type C2 clarifier me supported by a center column the full operating torque. A stal way extending from the side to the center column provide inspection and maintenance of

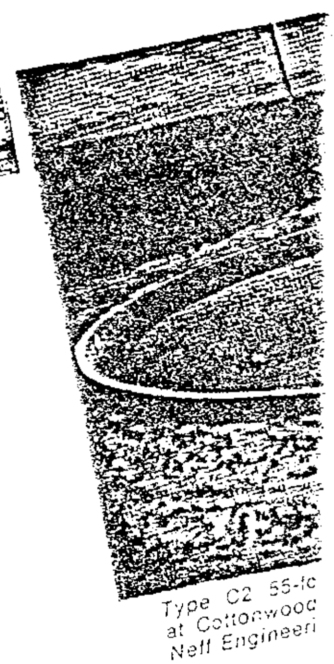
Influent flows through the column into the feedwell. T moval arms, with blades and attached to a center cage th from the rotating main gear. either one or two skimmer ported from the arms can b floating scum into the scur

Two types of drive ur C-Standard for smaller di the C-Balanced for large units offer a number of enclosed housings are chined seats throughout. ings run in oil, and al and oil level indicators a Hardened steel worms gears assure dependab forged alloy steel and cision-machined from steels or high-test cas

The heavy-duty large-diameter main friction ball, roller c on all rotating comp stability and high l protection is provic mechanical device provides alarm an able load points.



Two 150-foot diameter type C2 Eimco clarifiers treating municipal sewage at Tyler, Texas. Wisenbaker, Fix and Assoc., consulting engineers.



Type C2 55-fo at Cottonwood Neff Engineeri

C2D

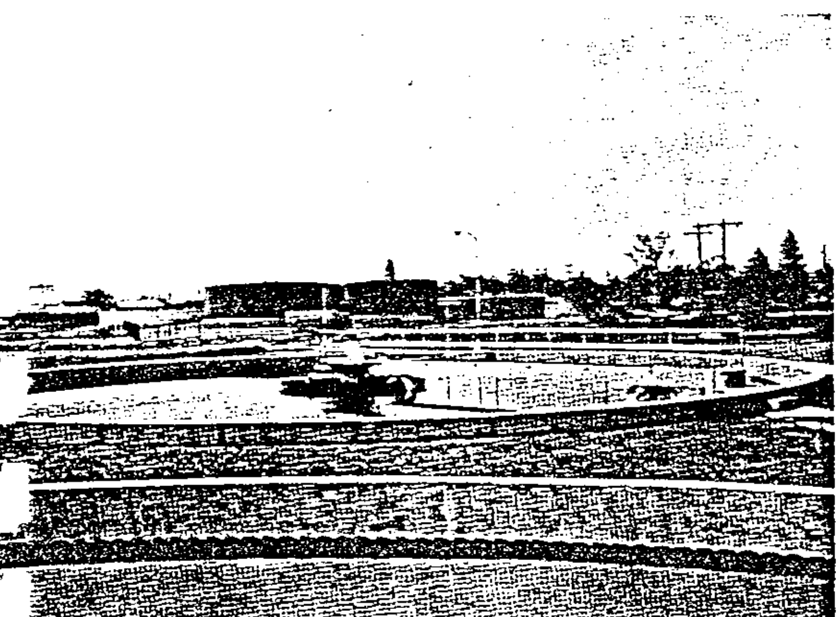
FOR ACTIVATED SLUDGE PLANTS: THE TYPE C2D MECHANISM —

The Eimco Type C2D activated sludge return clarifier mechanism is specifically designed for application in activated sludge plants. The mechanism features controlled, continuous, high-volume sludge removal, thus permitting the return of a biologically active sludge to the aeration basins. This assures maximum control and efficiency of the activated sludge process.

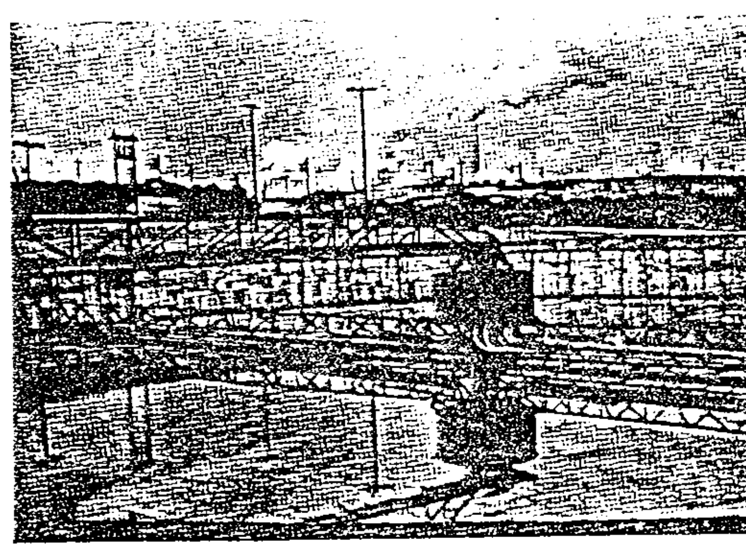
As the activated sludge separates from the mixed liquor introduced into the clarifier, it is collected through orifices into rectangular ducts rotating just above the clarifier floor. Each duct section is provided with its individual sludge drawoff pipe extending to a collection drum surrounding the center column. Rate of withdrawal from each sludge drawoff pipe is regulated by an adjustable control gate, using an operating wrench from the operating platform. This unique system of controlled sludge withdrawal permits the operator to easily obtain the optimum sludge return for the process without the need to either dewater the clarifier or fuss with the cumbersome, fixed discharge rate provided by overflow rings that are subject to fouling.

The C2D clarifier is made in sizes from 30 to 200 feet diameter, with or without surface skimming.

The Eimco Type C2DC is available with sludge drawoff pipes for square units.



Two 85-foot diameter type C2D clarifiers at Greeley, Colorado, treating municipal waste. Parker and Underwood, consulting engineers.



One of two 185-foot-diameter type C2D clarifiers treating waste from Southland Paper Mills, Inc. plant at Sheldon, Texas. Brown & Root, Inc., consulting engineers.

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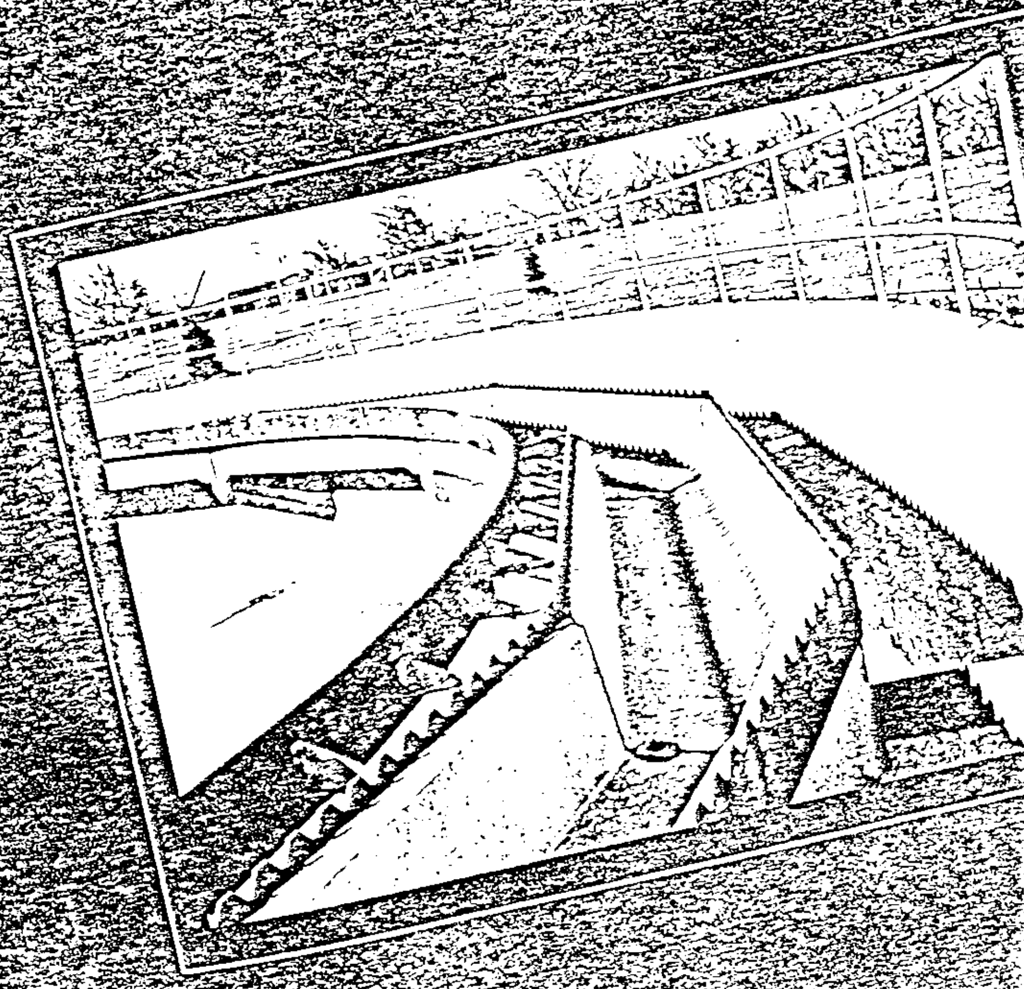

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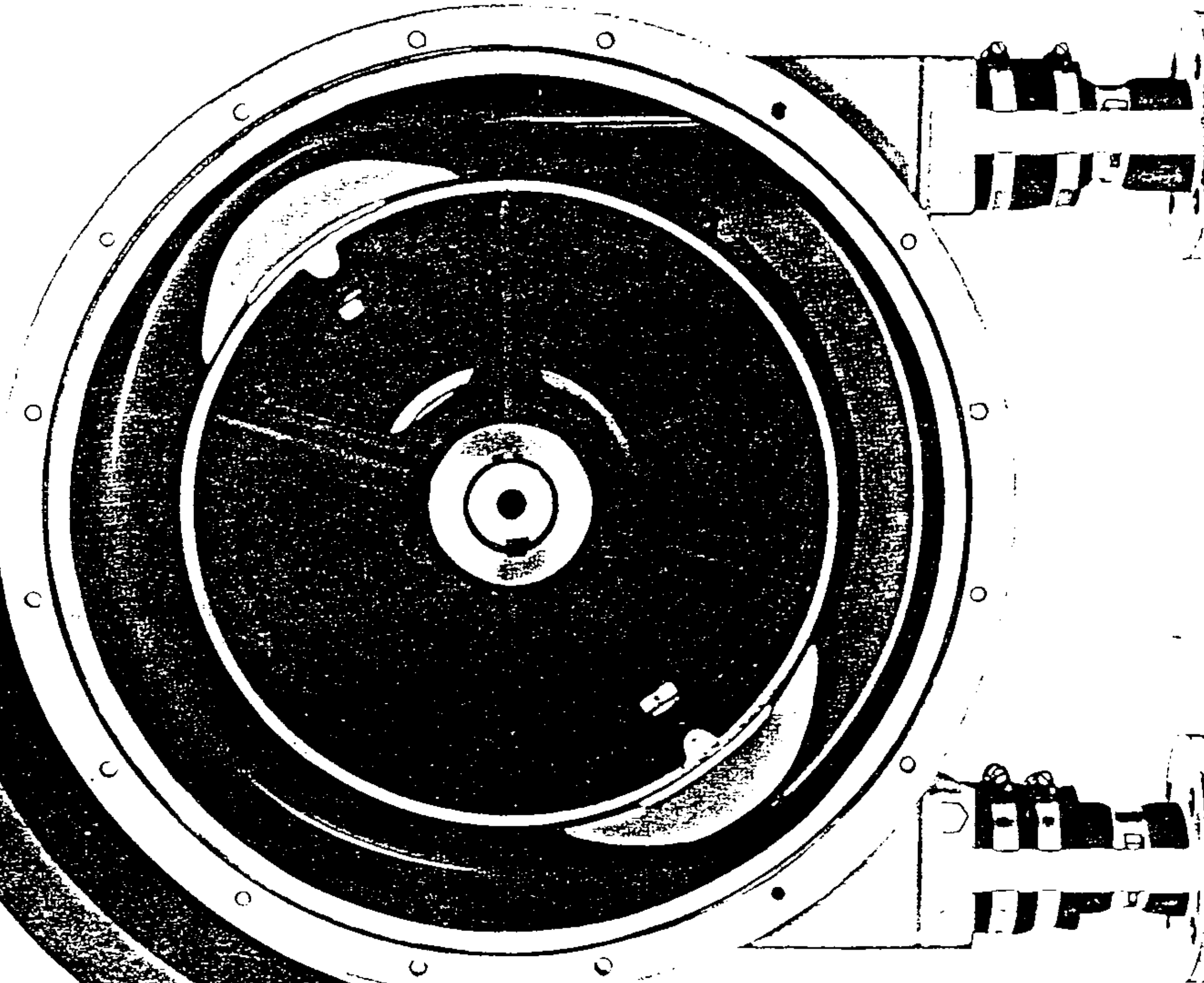

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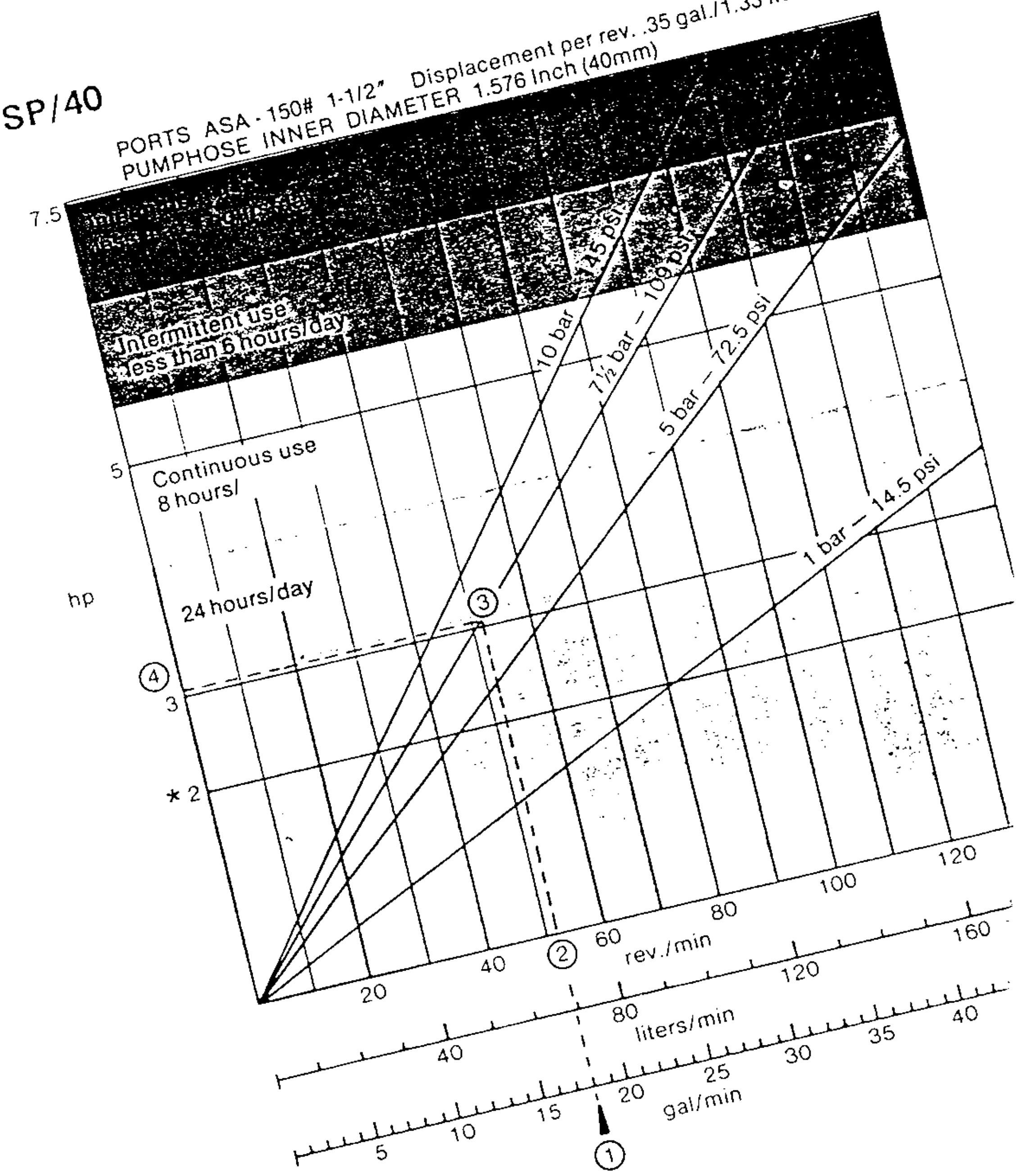
How To Calculate Speed/Horsep

1. Flow Required
2. Required Speed
3. Calculated Pressure
4. Horsepower Required

TYPE SP/40

PORTS ASA - 150# 1-1/2" Displacement per rev. .35 gal./1.33 liter
PUMPHOSE INNER DIAMETER 1.576 Inch (40mm)



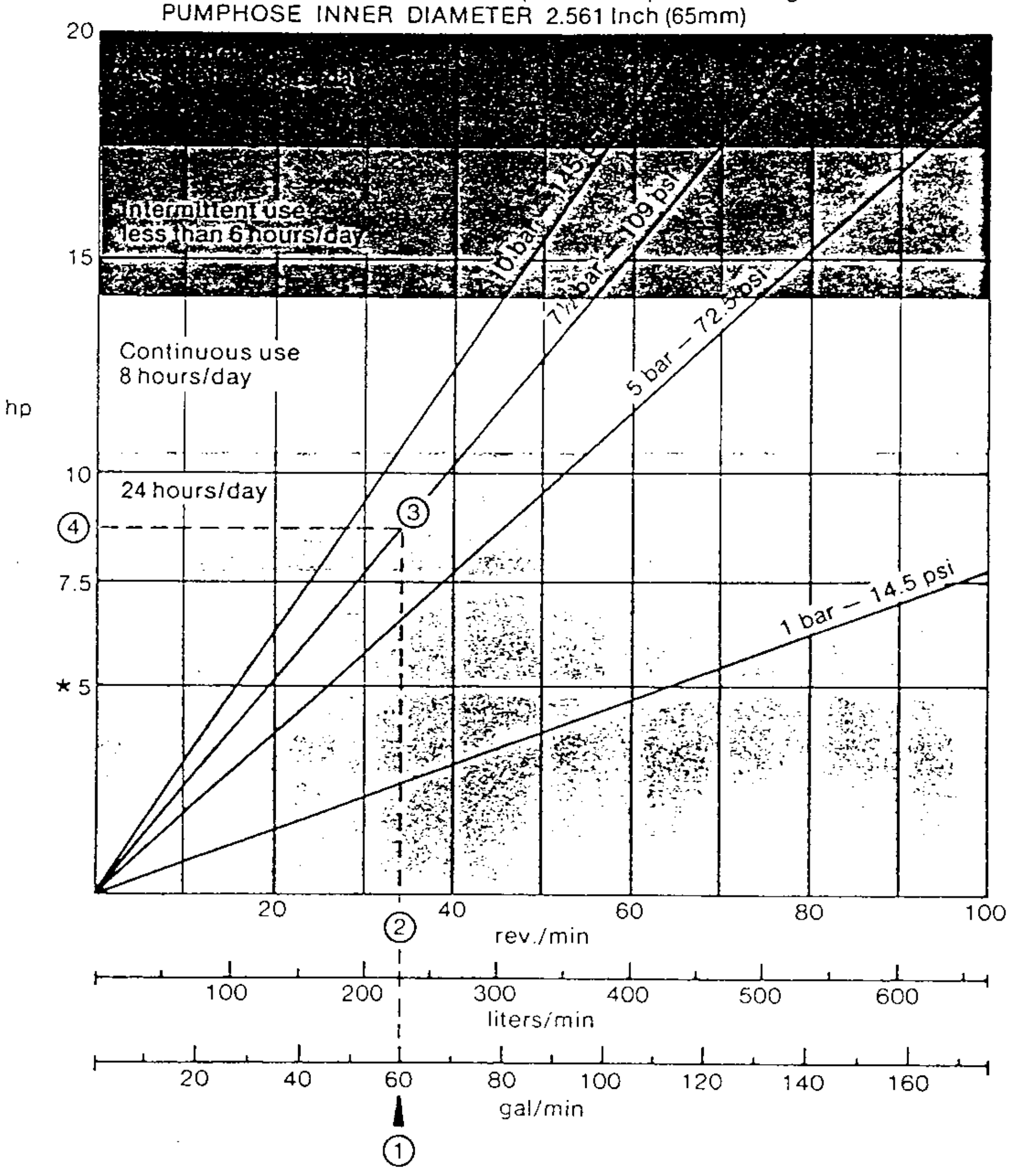
PERFORMANCE CHART

How To Calculate Speed/Horsepower

1. Flow Required
2. Required Speed
3. Calculated Pressure
4. Horsepower Required

TYPE SP/65

PORTS ASA - 150# 2-1/2" Displacement per rev. 1.75 gal./6.62 liter
PUMPHOSE INNER DIAMETER 2.561 Inch (65mm)



NOTES:
*Minimum running drive torque requirement below this hp is 4,100 in. lbs. Depending on operating environment, starting torque can be two to three times running torque.

For temperatures higher than 170°F, consult factory.

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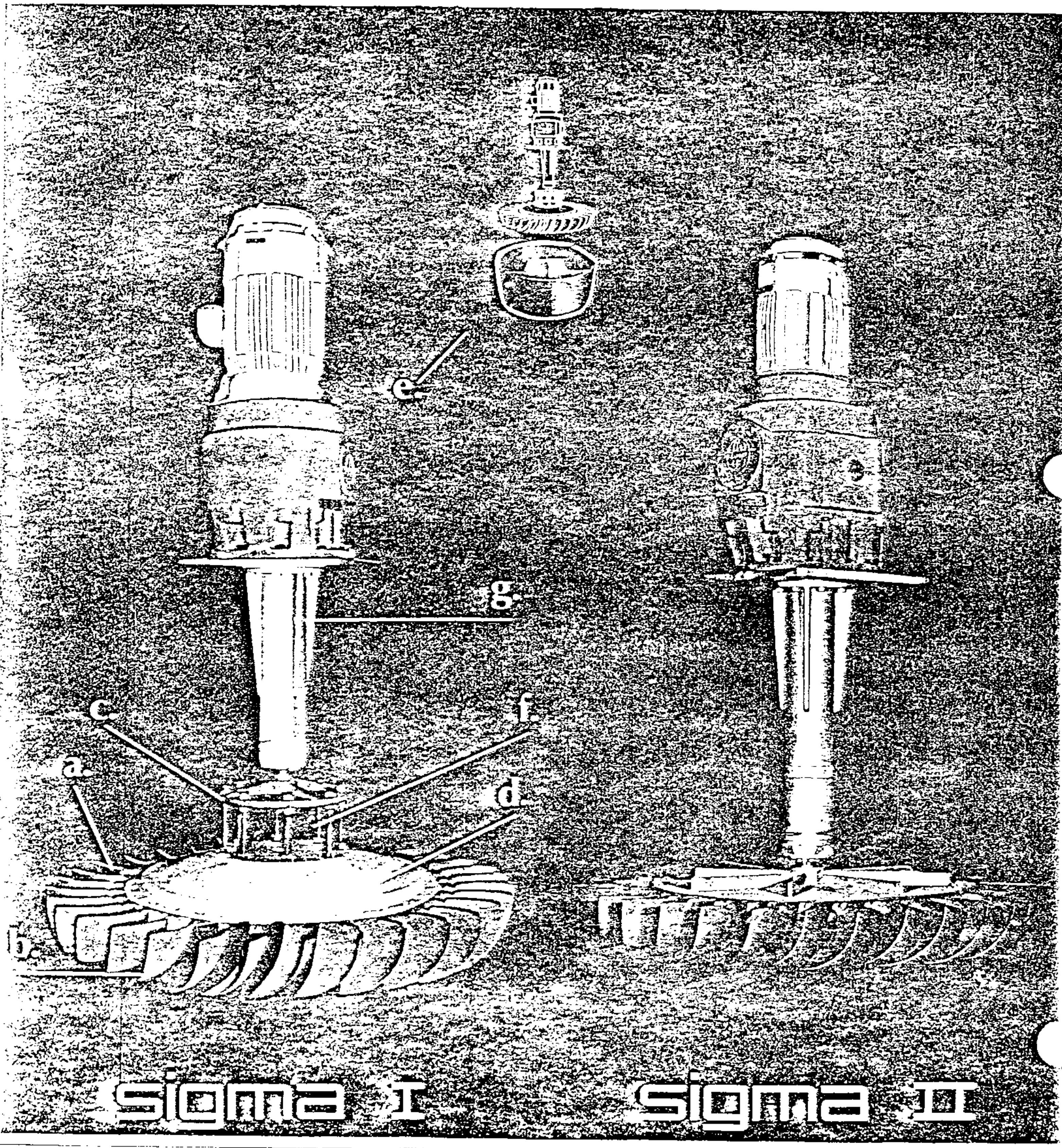
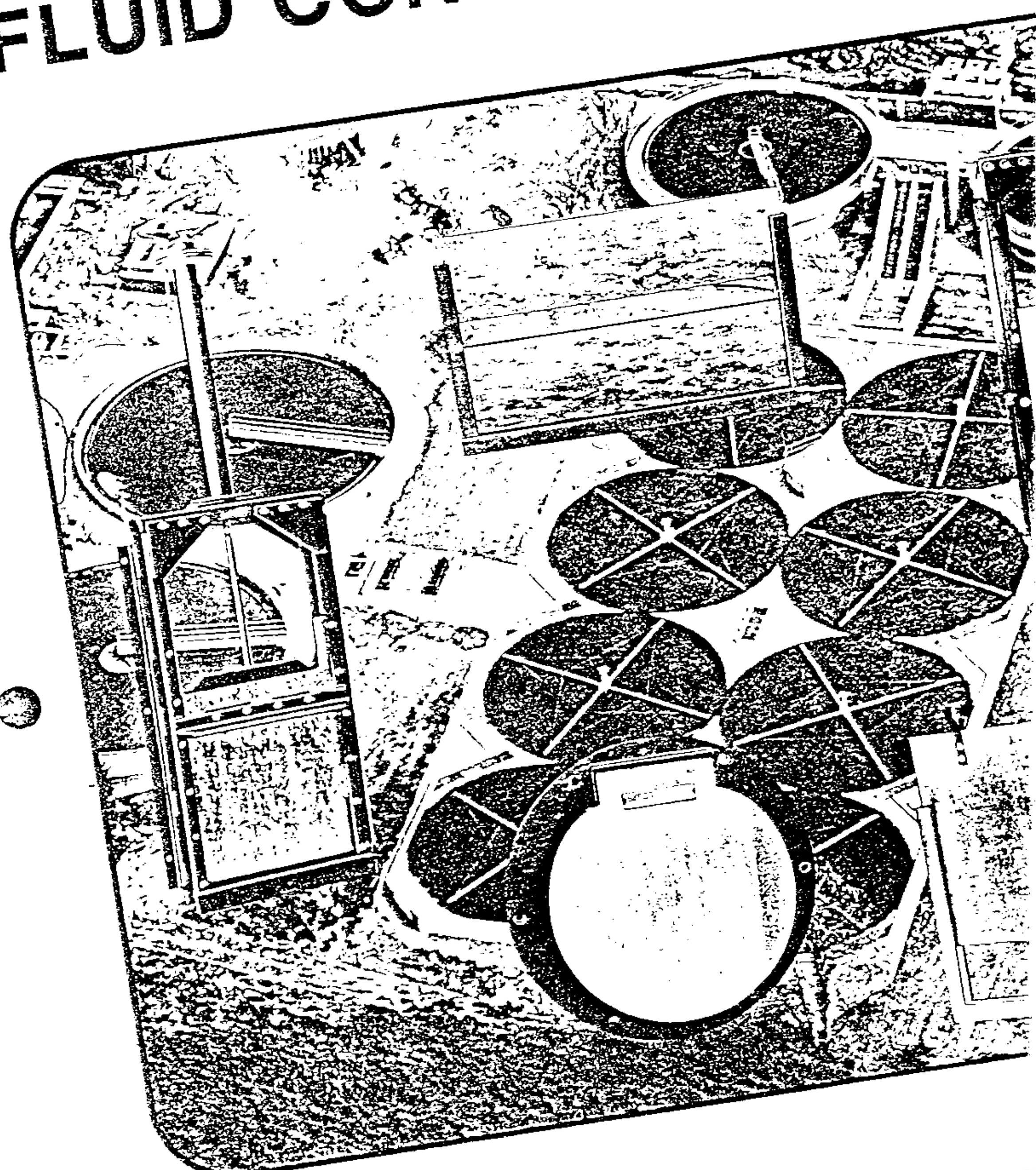


COPLAS
FLUID CONTROL EQUIPM



ASHBROOK-SIMO
WATER POLLUTION CONT
Design Specifications for Industry, Electric

COPLASTIX FLUID CONTROL EQUIPMENT

Sealing

Water tightness of Coplastix products is determined by the duty for which the product has been designed. For example: Stop Logs will not be as water tight as sluice gates, similarly Chased Inverts will be more water-tight than Flush Inverts. Generally, the degree of sealing achieved is defined as 28.04 & 28.05 from AWWA Spec . C501-67.

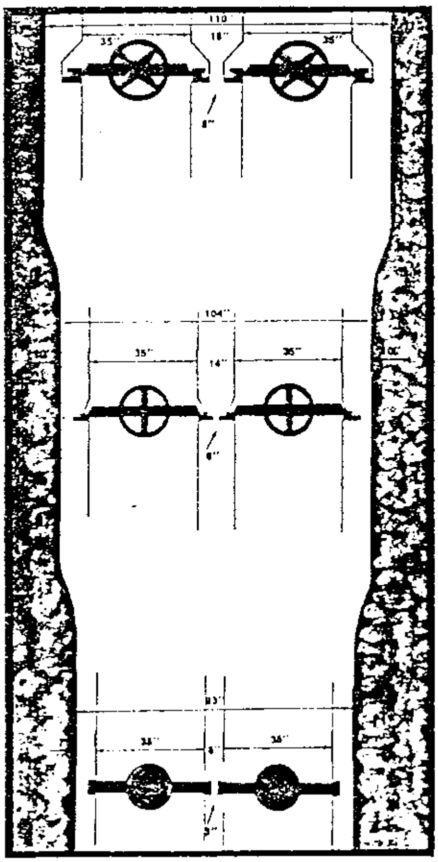
Sizes

Coplastix products are manufactured in a range of preferred imperial sizes. These preferred items are very competitive in price and are available either ex-stock or on a comparatively short term delivery. Items outside the preferred ranges, in metric or imperial sizes, and incorporating individual or special features, can be quoted on request. This provides an almost infinite choice of specification.

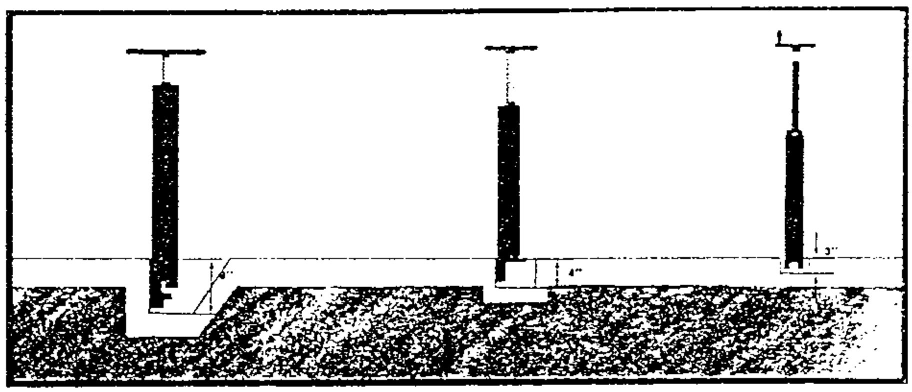
Future Developments

As pioneers in the introduction of synthetics in the manufacture of fluid control equipment, Coplastix has achieved a marked success in this field. Now, as part of Ashbrook-Simon-Hartley,it combines many years of experience in the manufacture of high performance equipment for similar types of applications. This experience together with knowledge of the water and sewage engineers' problems enables Ashbrook-Simon-Hartley to offer its services with complete confidence.

Rapid progress in the development of new synthetic materials and techniques means that full advantage will be made of new discoveries in order to continue the development of Coplastix products and Ashbrook-Simon-Hartley as the largest suppliers of plastic fluid-control equipment to the water, sewage and effluent treatment industries.



Comparisons shown here between a similar size of cast-iron sluice gate. Rubasil sluice gate and Coplastix sluice gate building-in requirements highlight the considerable savings that can be made when installing Coplastix channel sluice gates. No special reinforcing or deep recessing is needed; channel wall-thickness can be reduced by over 60% and the area necessary for the construction of a double 36 in. channel is reduced by 16%.



DISTINCTIVE FEATURES Common to all COPLASTIX Products

Advantages	Benefits
Greatly reduced weight	Reduction in handling and installation costs. Reduction in torque requirements for sluice gate door operation. Larger units before geared operation is necessary. Smaller and less costly actuators, when fitted.
Low friction in moving parts	Less physical effort for manual operation. Smaller handwheels. Smaller and less costly actuators for motorized units.
Corrosion-free materials	Longer functional life. Resistance to chemical attack is better than most metals. Hardly any maintenance needed.
Smooth plastic surfaces	No painting required; original color maintained. Cleans with a wipe. Repels algae and marine growth.
Non-toxic	Not affecting or effected by most chemical processes.
Low thermal expansion	Will not buckle or warp.
Flush-invert sluice gates	No grit pockets in channel floor.
Smaller overall frame size	Simpler mounting.
Wide temperature range	Expansion in the finished Coplastix products is well catered for between the ranges of minus 30°C and plus 68°C.
Good fire properties	Most Coplastix materials are completely self-extinguishing and are in Class I Spread of Flame rating table.

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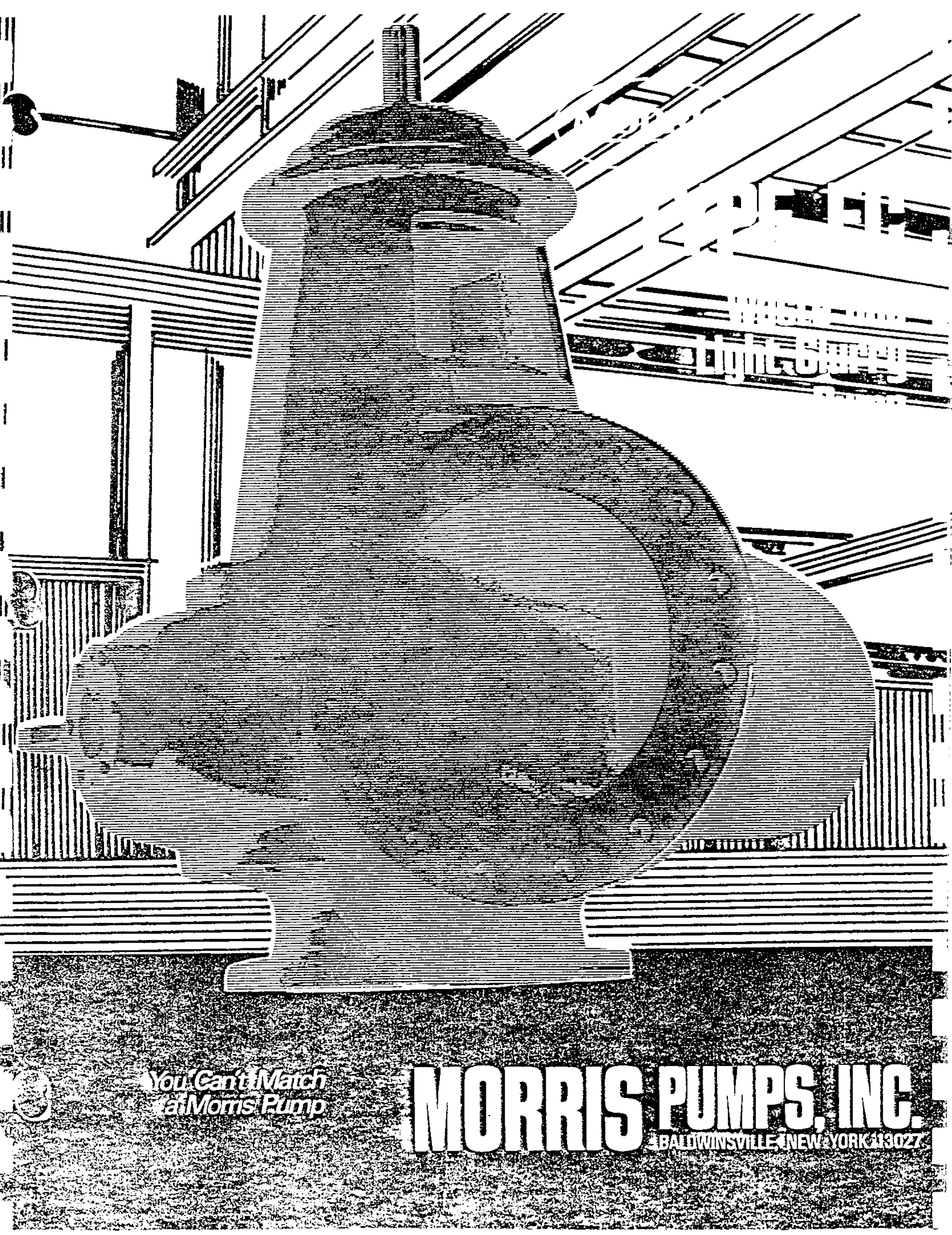
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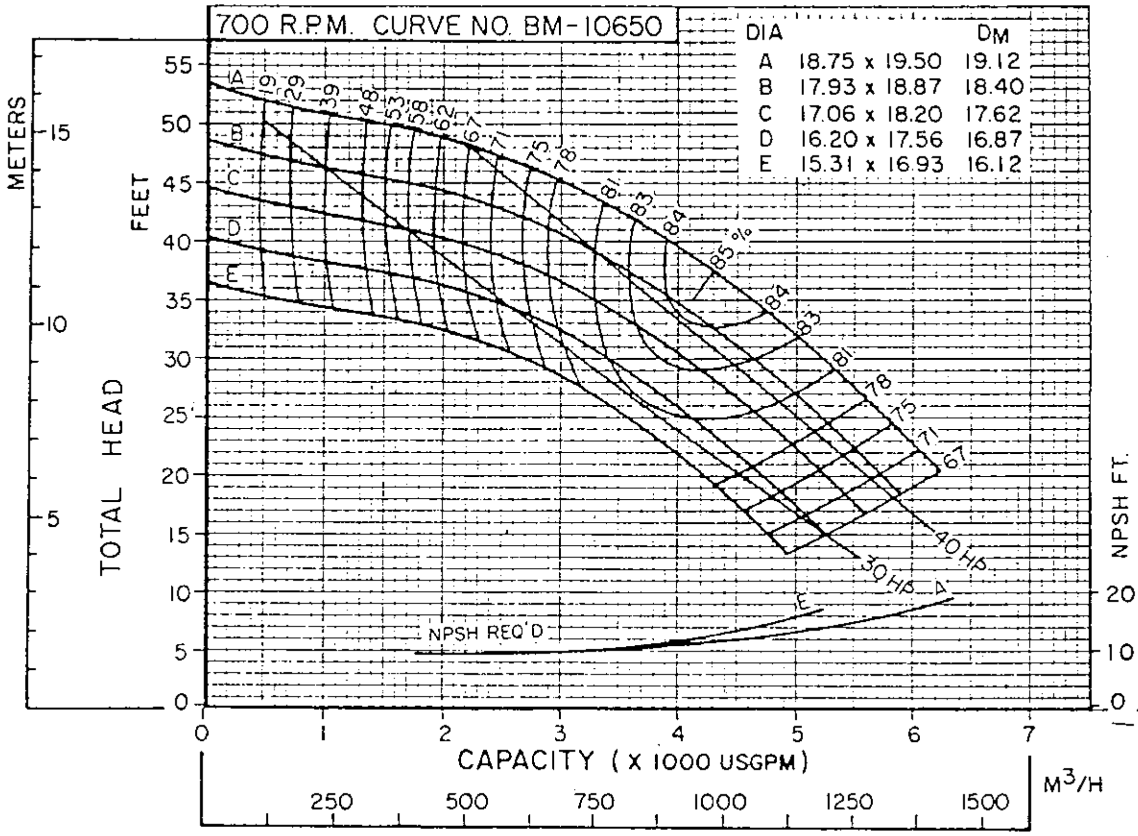
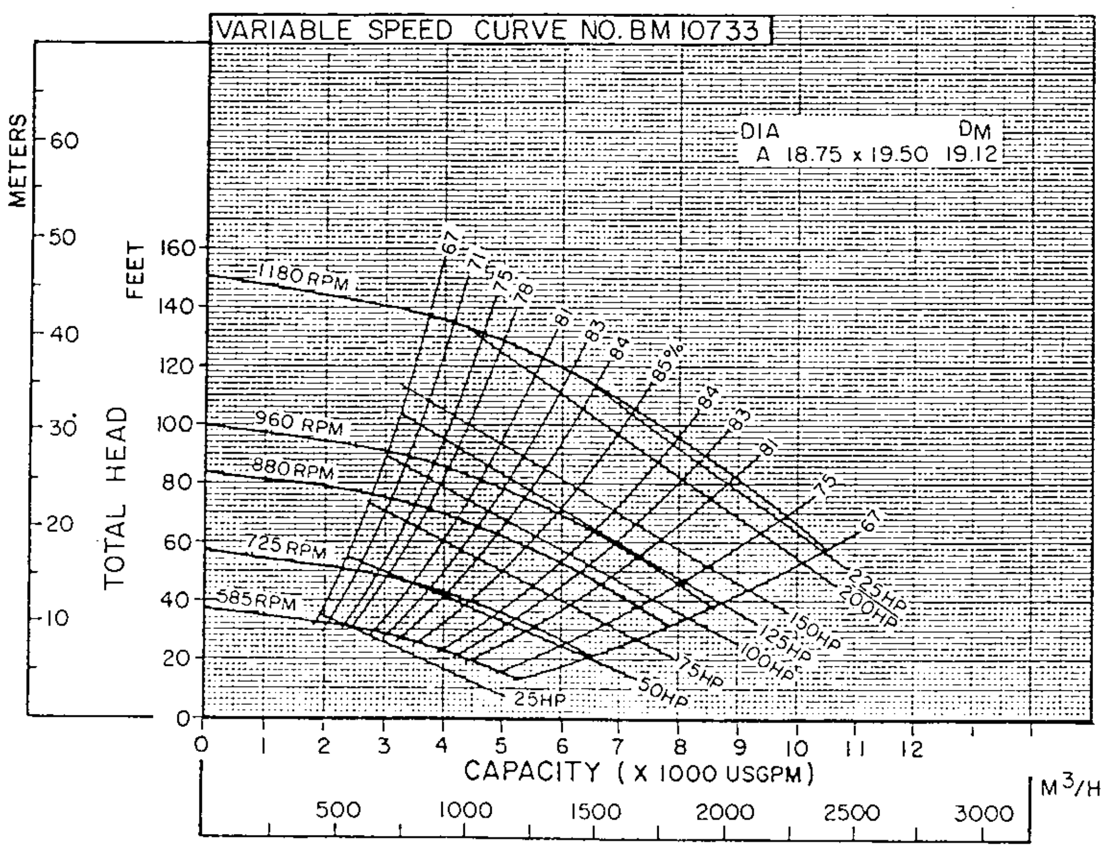
AUGUST 1974

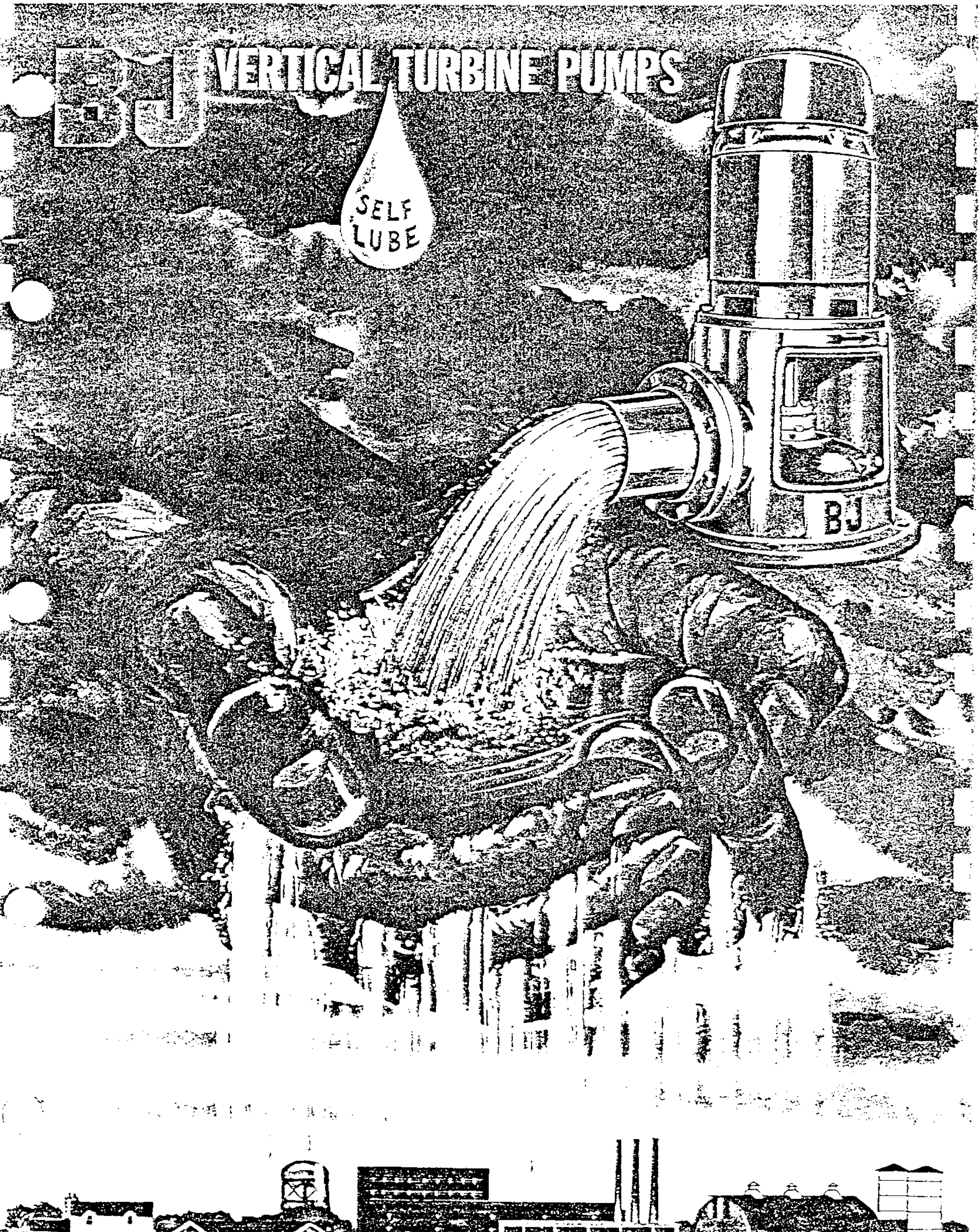
TYPE 12 EC — 3 VANES

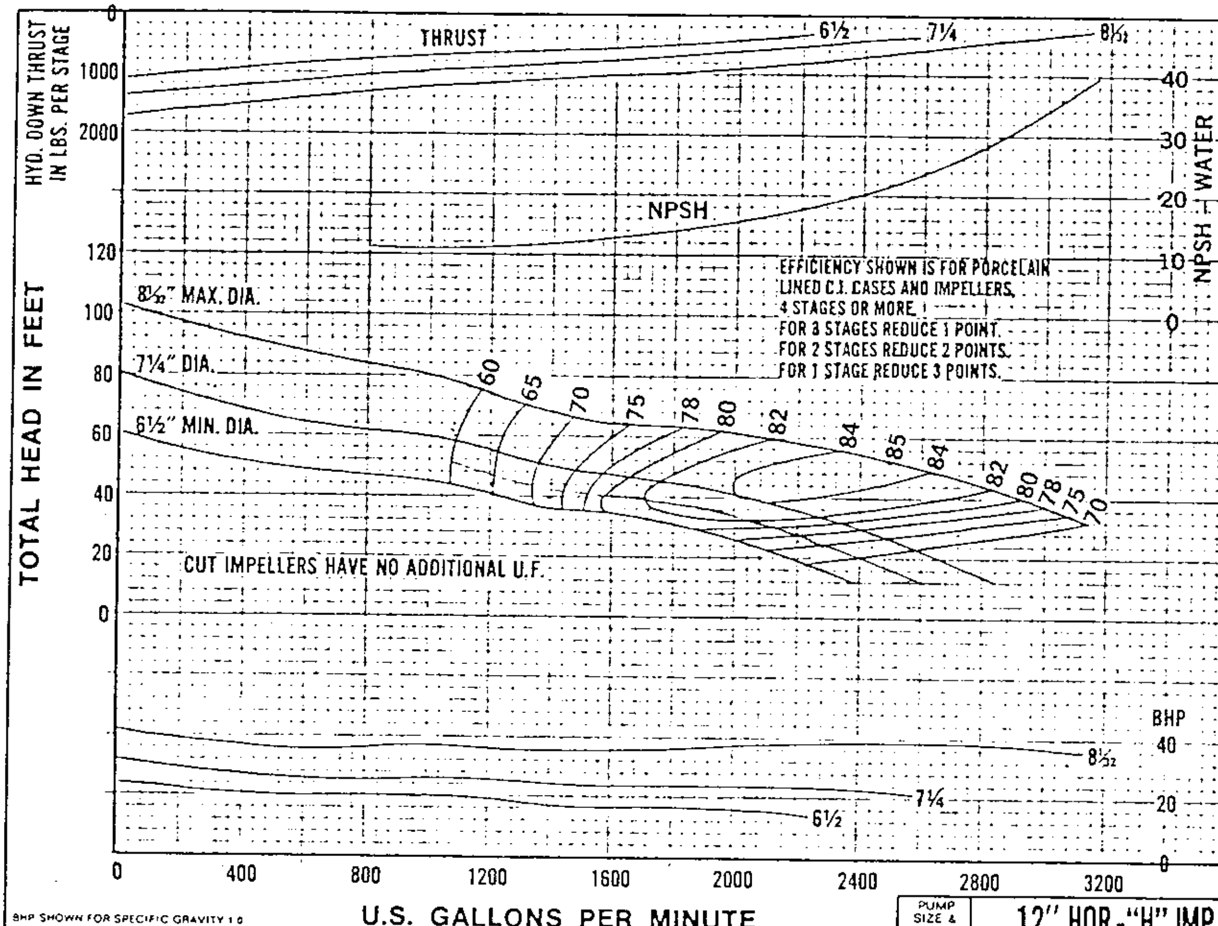
MAX. SPHERE 3.9" IMP. HYD. NO. 5604800 CAS. HYD. NO. 5604803





These curves show the characteristics when pumping clear water at 68°F./20°C


BJ
VERTICAL TURBINE PUMPS
SELF LUBE
BJ



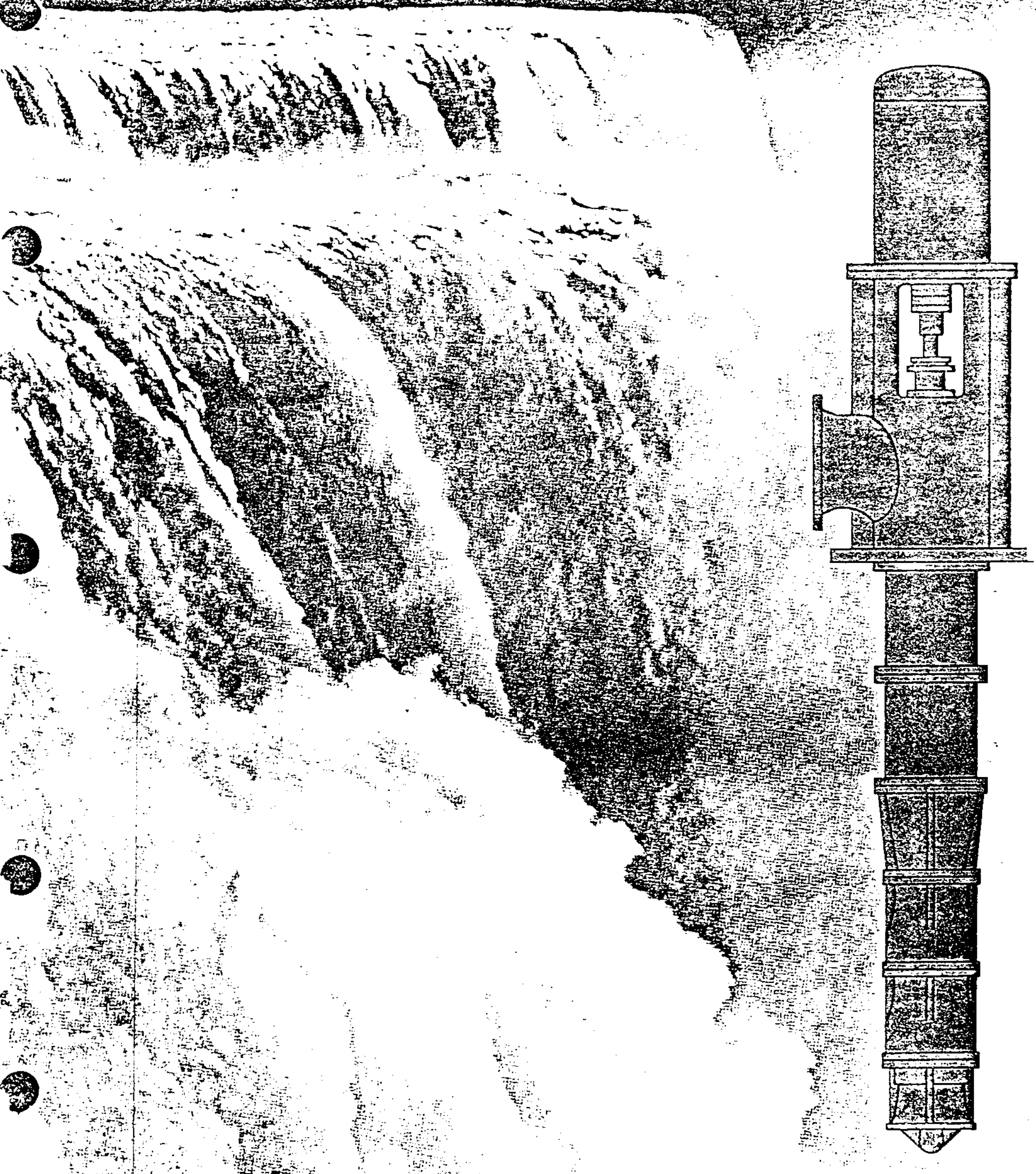
BHP SHOWN FOR SPECIFIC GRAVITY 1.0

DRAWN BY	CLAIBOURN
DATE	11-11-72
SUPERSEDES	
DATE	
BOWL DIA.	11¾
MINIMUM SUBMERGENCE	24"
IMPELLER NO	268178
N_s – PUMP SPECIFIC SPEED	4538
TYPE OF IMP.	☐ OPEN ☒ CLOSED
EYE AREA SQ. IN.	36.2
U. FT./SEC.	54.5
RPM	1770
BJ RATING NO.	3541

PUMP SIZE & TYPE: 12" HQR-"H" IMP. VERTICAL TURBINE

BYRON JACKSON HQ CIRCULATOR PUMPS

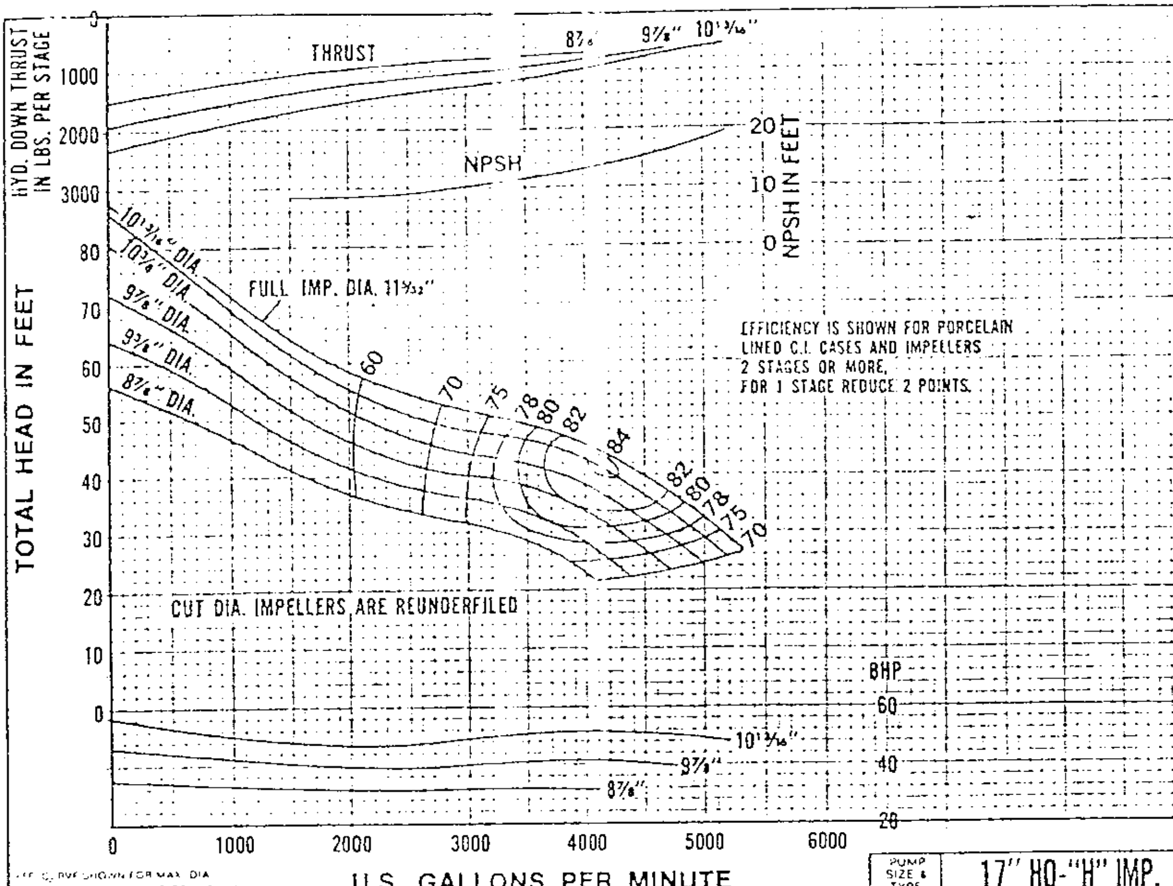
BULLETIN [illegible]



Byron Jackson Pump Division

DIVISION OF BORG-WARNER CORPORATION

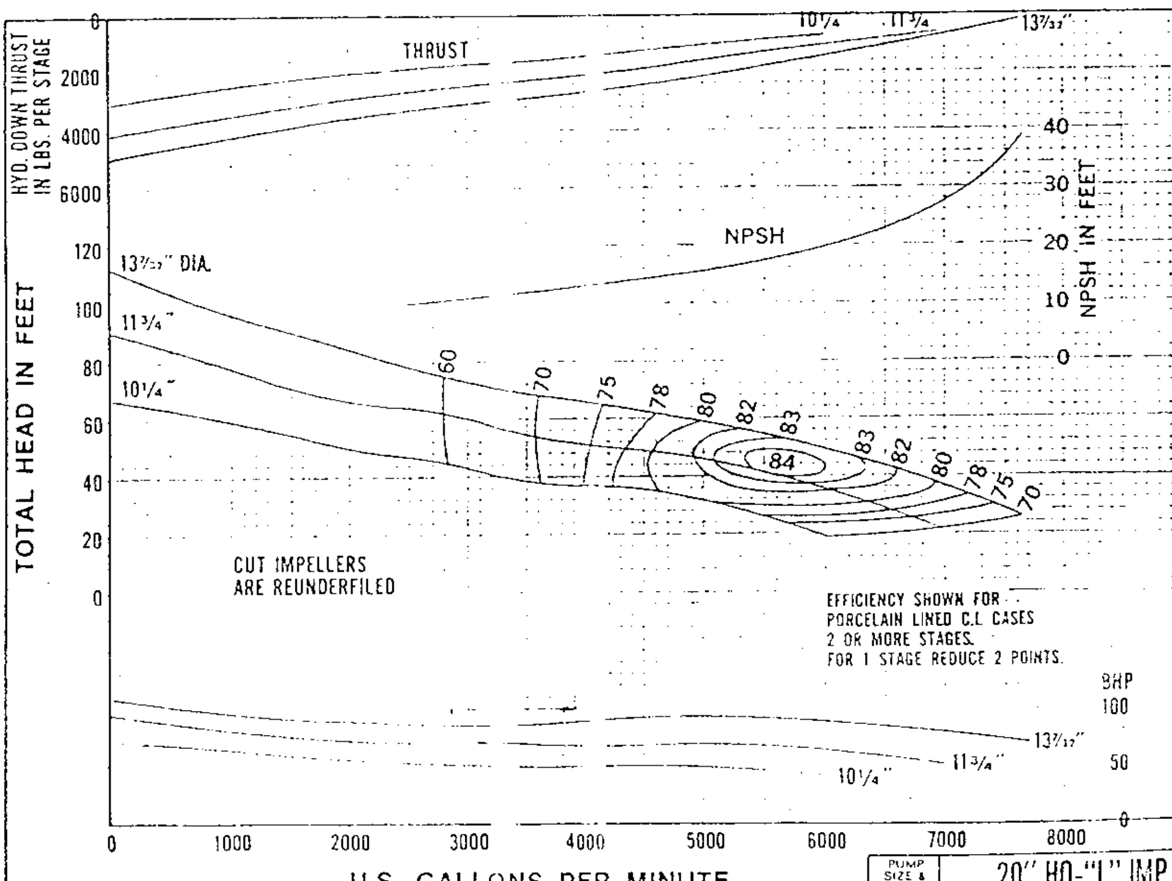
BORG WARNER



DRAWN BY	C.M.D.
DATE	10-15-73
SUPERSEDES	3448
DATE	12-12-70
BOWL DIA	16 7/8"
MINIMUM SUBMERGENCE	36"
IMPELLER NO	267610-LY
N_s — PUMP SPECIFIC SPEED	4568
TYPE OF IMP	☐ OPEN ☒ CLOSED
EYE AREA SQ. IN	71.4
U. FT./SEC	47.2
RPM	1170
BJ RATING NO	3448

EFF. CURVE SHOWN FOR MAX. DIA
BHP SHOWN FOR SPECIFIC GRAVITY 1.0

PUMP SIZE & TYPE: 17" HQ-"H" IMP. VERTICAL TURBINE

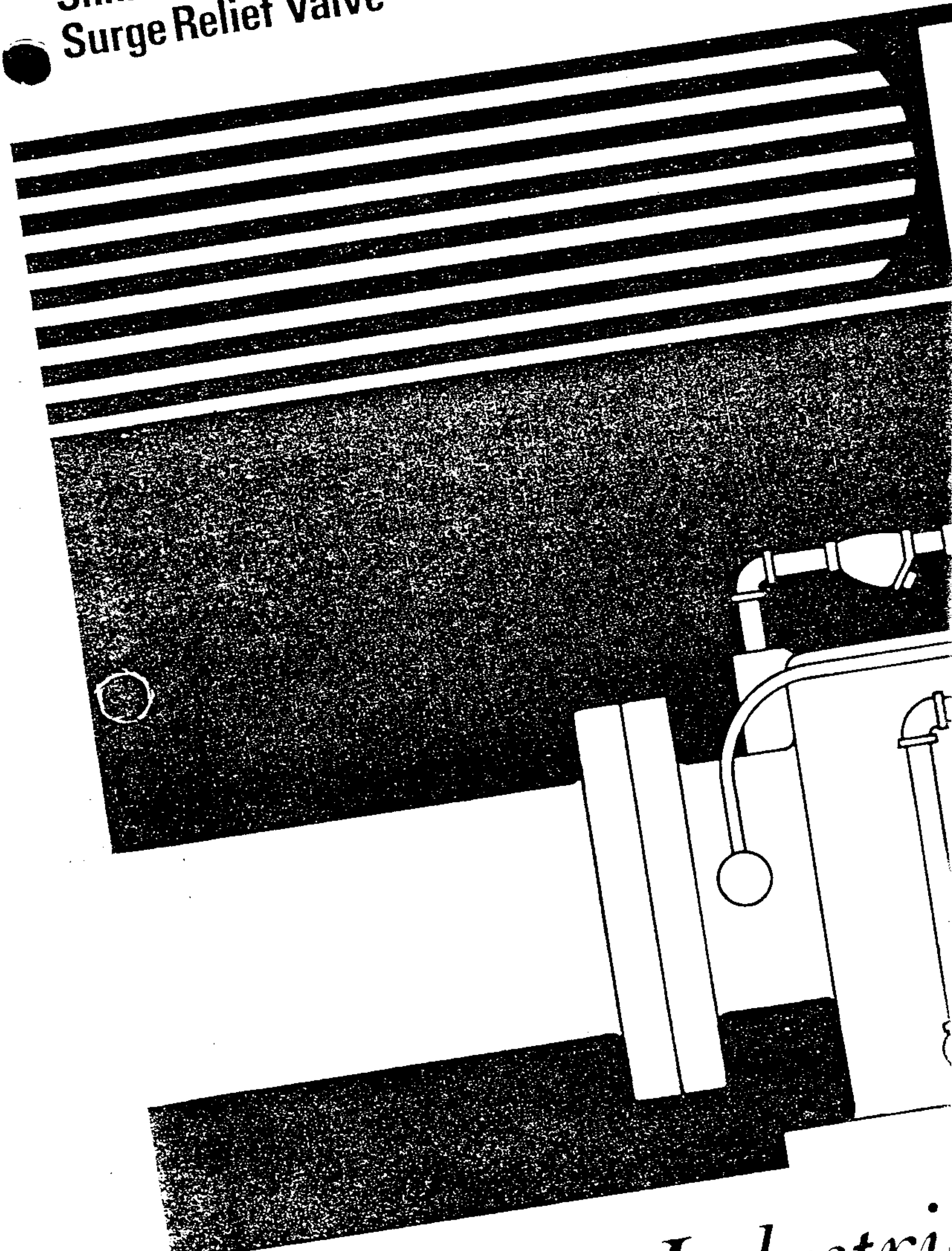


DRAWN BY	CLAIBOURN
DATE	12-11-71
SUPERSEDES	TEMP. 20HQ-1
DATE	2-12-71
BOWL DIA	20
MINIMUM SUBMERGENCE	60"
IMPELLER NO	267840-LY
N_s — PUMP SPECIFIC SPEED	4893
TYPE OF IMP	☐ OPEN ☒ CLOSED
EYE AREA SQ. IN	92.6
U. FT./SEC	57.1
RPM	1170
BJ RATING NO	3491

BHP SHOWN FOR SPECIFIC GRAVITY 1.0

PUMP SIZE & TYPE: 20" HQ-"L" IMP. VERTICAL TURBINE

Shhhockless Surge Relief Valve



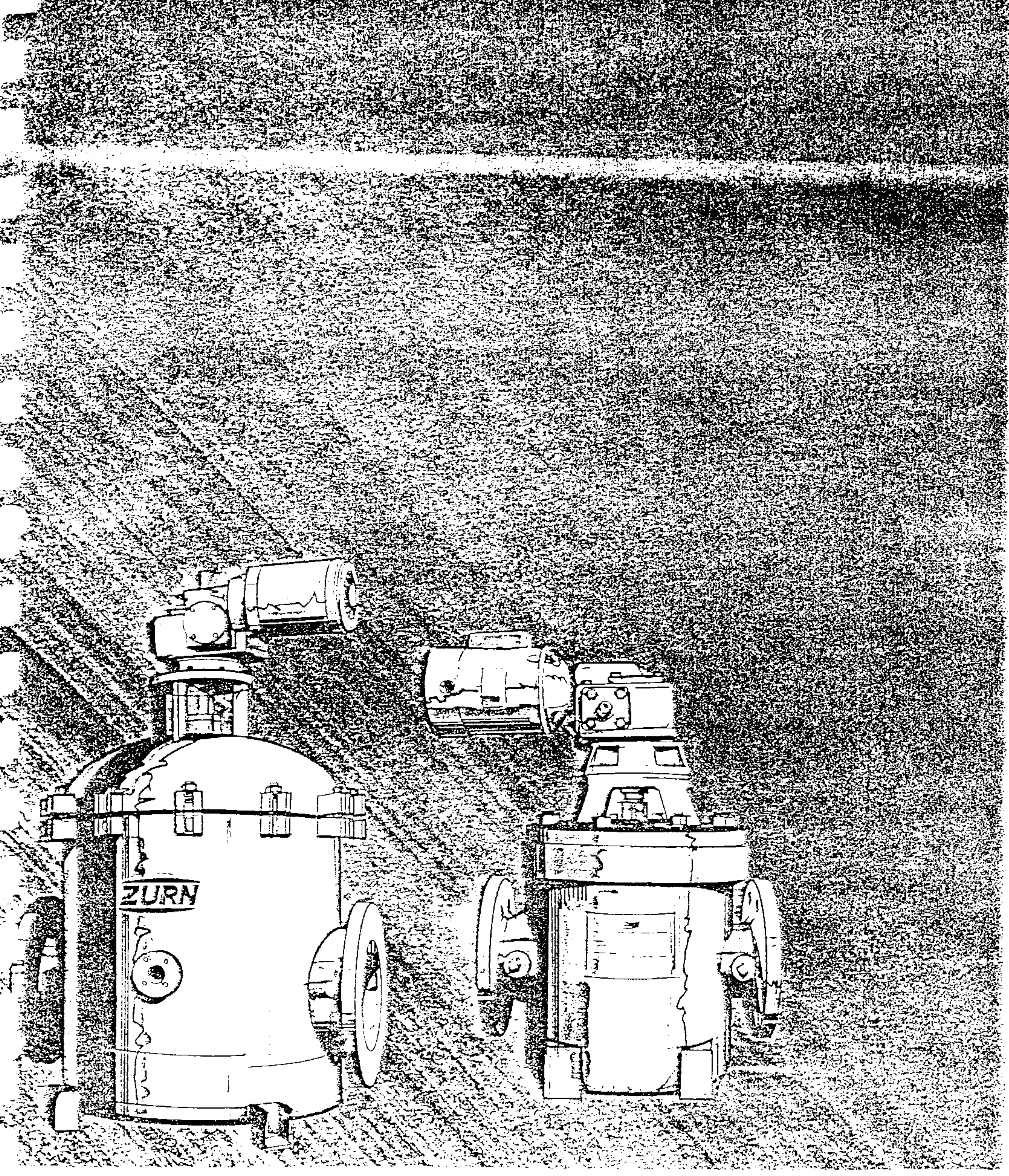
Industri

AUTOMATIC VALVE SPEC

RIDGE AVENUE PITTSBURGH PA. 15233•

Self-Cleaning Pipeline Strainers

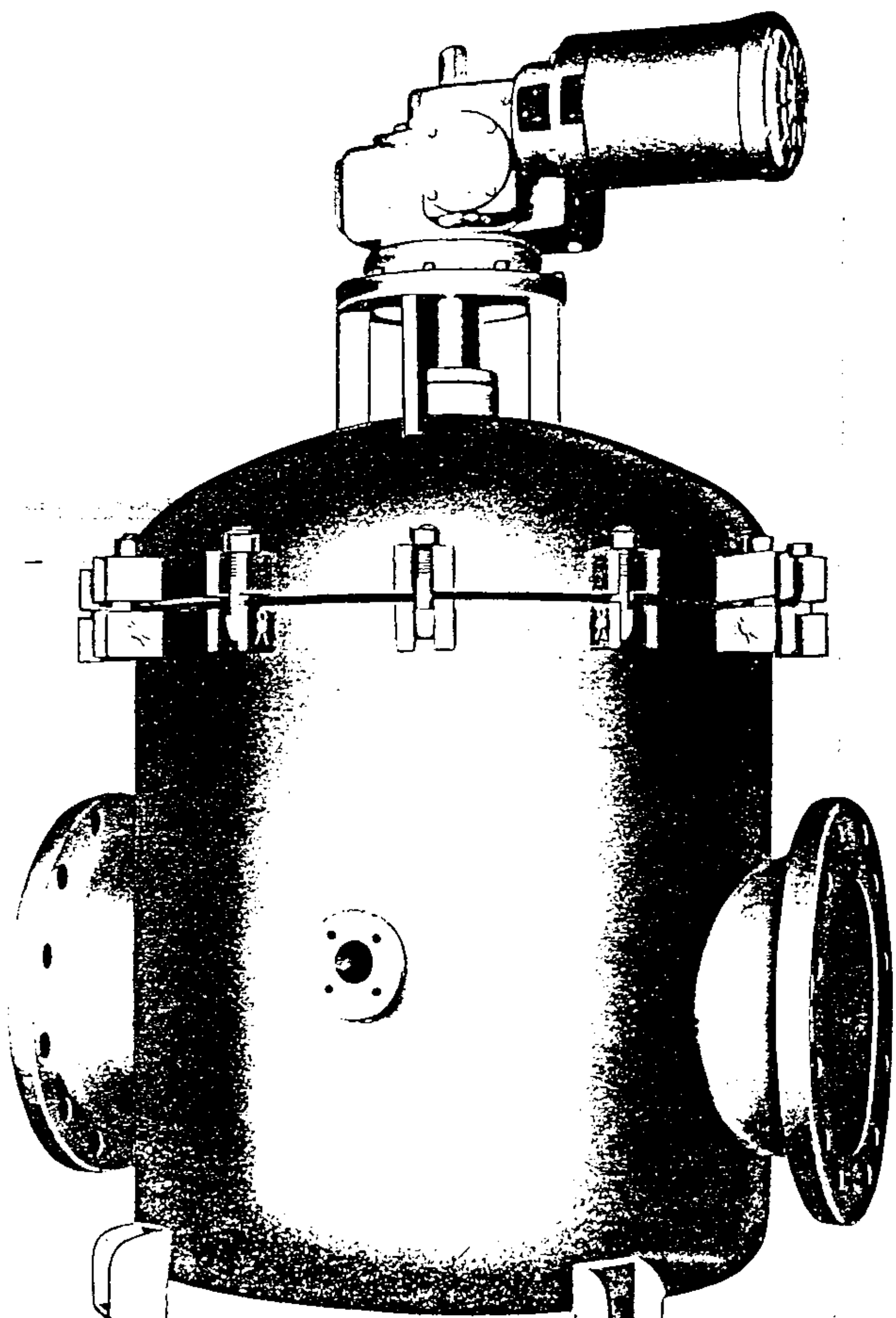
a step ahead of tomorrow



593 series

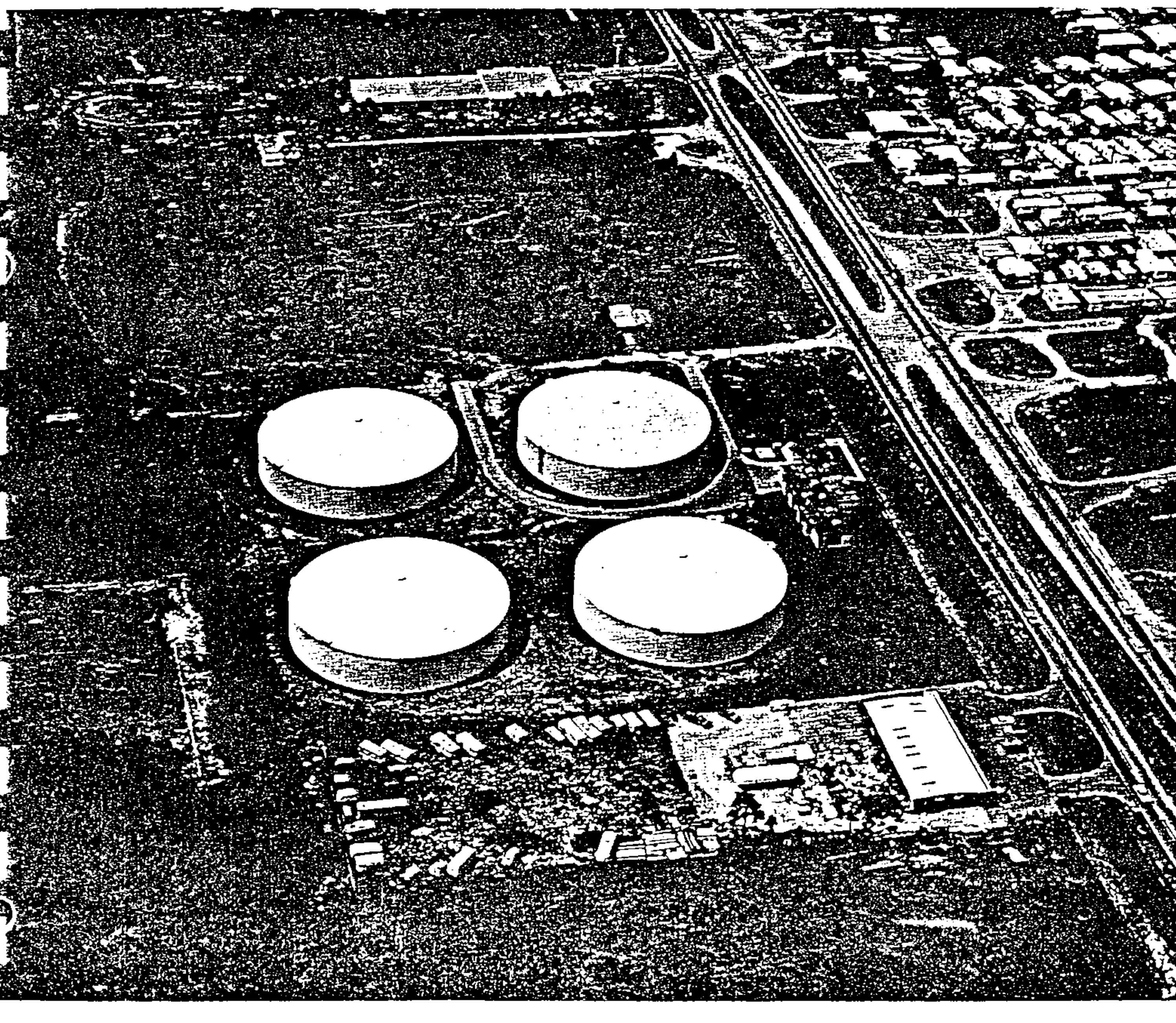
'train-O-Matic'
Self-Cleaning Strainers

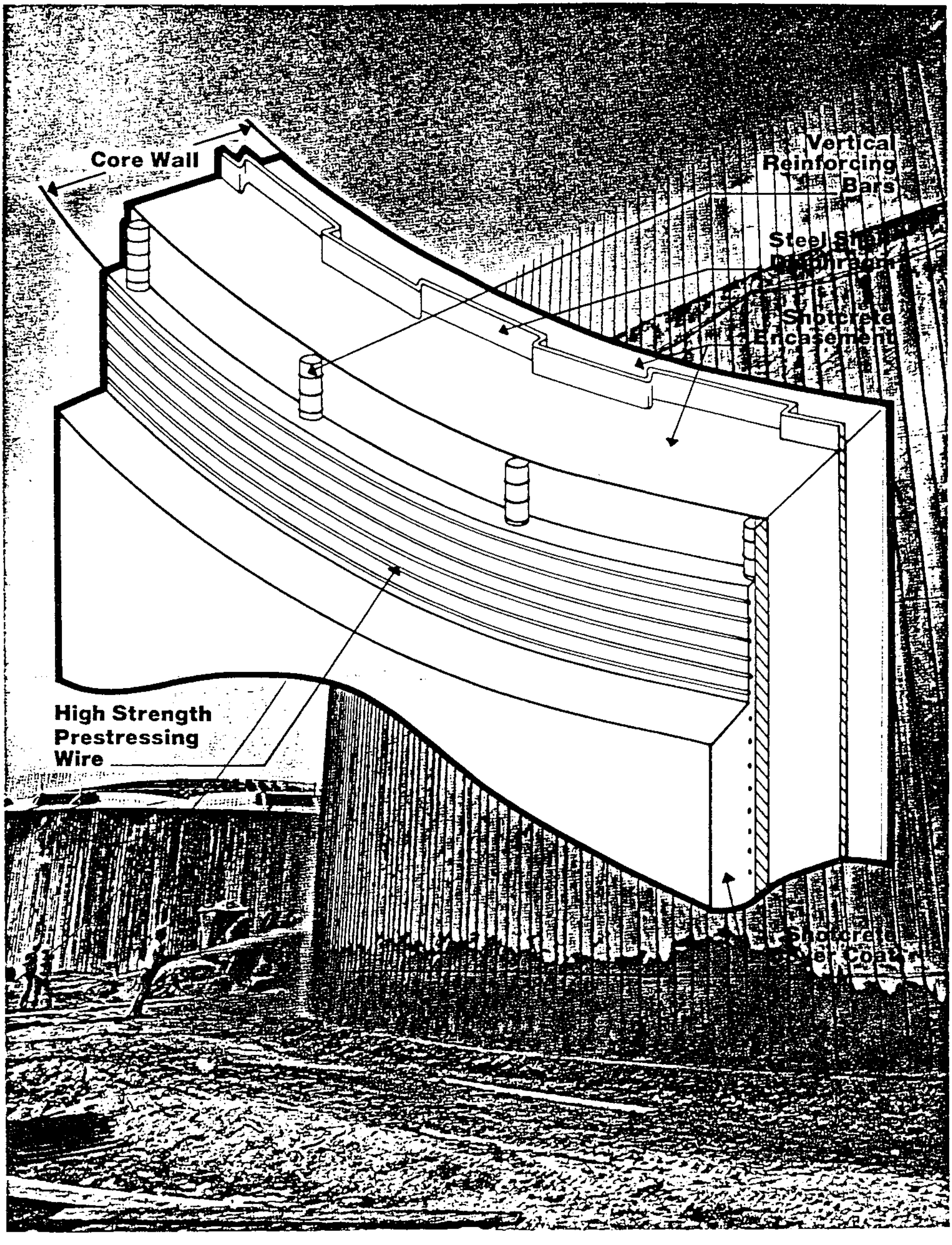
'Turn "Strain-O-Matic" removes ebris easily, economically, automatically



Prestressed Composite Tanks

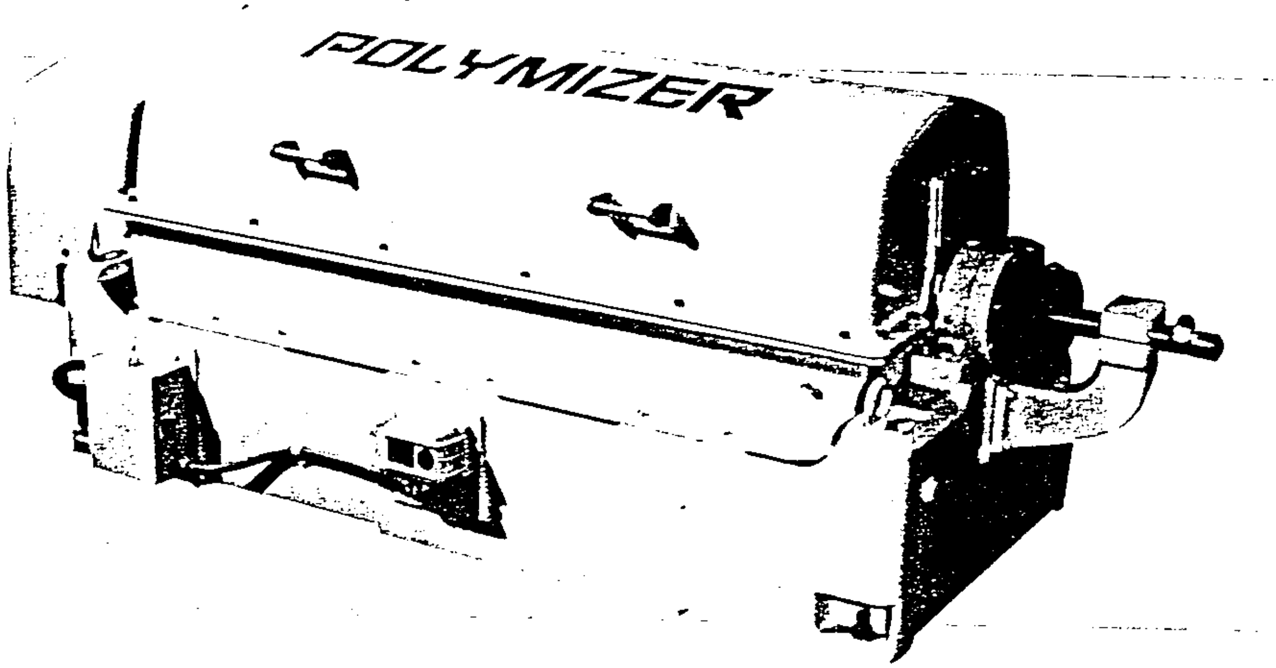
The Crom Corporation




Core Wall
Vertical Reinforcing Bars
Steel
Shotcrete Encasement
High Strength Prestressing Wire
Shotcrete

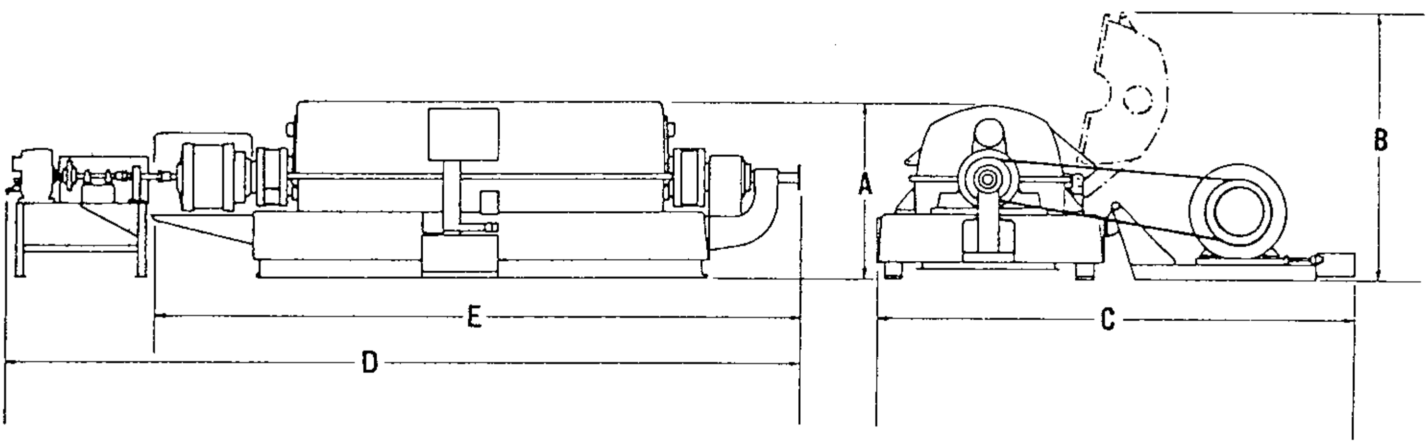
PENNWALT
CORPORATION

SHARPLES® POLYMIZER® CENTRIFUGES



Our latest innovation in cost-effective sludge dewatering.

Basic Specifications of POLYMIZER Centrifuges



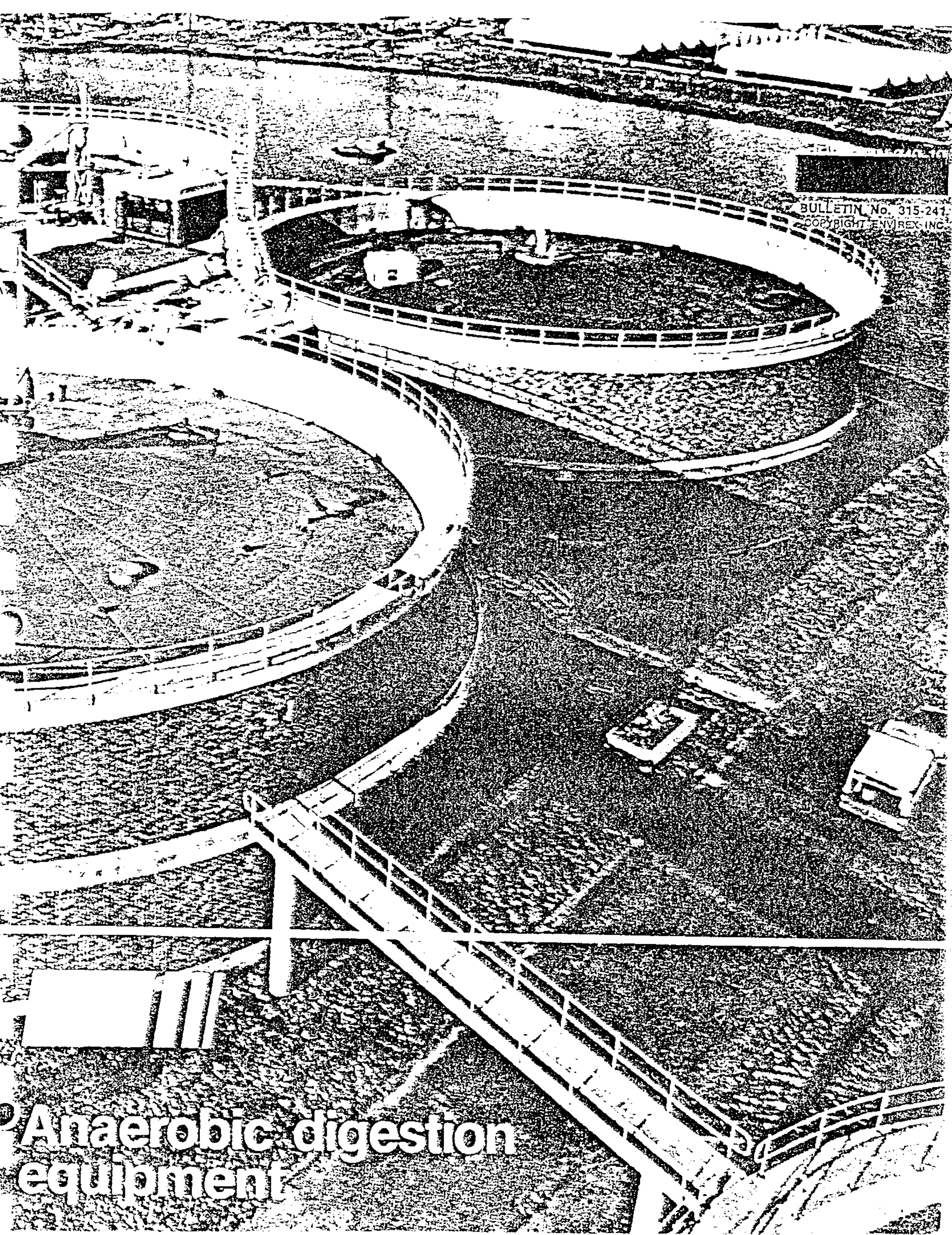
Side View End View

Model	Connected Motor Horsepower	Centrifugal Force (range in G's)	Height (in.)		Width** (in.) C	Length (in.)		Approx. Weight*** (lb.)
			Casing Closed A	Casing Open B		With Eddy Current Brake D	Without Eddy Current Brake E	
PM-20000	10-25	1000-2500	34	51	76	98	73	2000
PM-30000	20-40	1000-2500	38	54	76	116	92	2500
PM-35000	20-50	1000-2500	39	55	76	116	92	3100
PM-40000	50-75	1000-2500	42	64*	104	171	146	6000
PM-50000	60-75	1000-2500	42	64*	107	204	176	7400
~~PM-55000~~	~~60-100~~	~~1000-2500~~	[illegible]	[illegible]	[illegible]	~~204~~	[illegible]	~~9000~~
PM-60000	75-125	1000-2500	48	74*	140	199	165	11,700
PM-70000	75-125	1000-2500	48	74*	140	224	190	13,400
PM-75000	75-150	1000-2500	51	79	140	224	190	16,100
PM-80000	75-150	1000-2300	45	69	126	237	194	15,500
PM-95000	125-200	1000-2000	69	110	165	300	260	34,000

Specifications subject to change without notice.
*Casing cover not hinged. Dimension shown is vertical height of upper cover to clear adjacent machine.
**Without lube system.
***Without drive motor and back drive.

Consider POLYMIZER centrifuges early in your plans.

For a wide variety of sludge dewatering situations. Start with a telephone call to one of our experienced engineering and sales representatives. Ask him to arrange an on-site test. Or a visit to an operating Performance Center. See the back cover for the telephone number of the representative nearest to you.



BULLETIN No. 315-241
COPYRIGHT ENVIREX INC.

Anaerobic digestion equipment

Benefits of anaerobic sludg

Anaerobic sludge
number of advar
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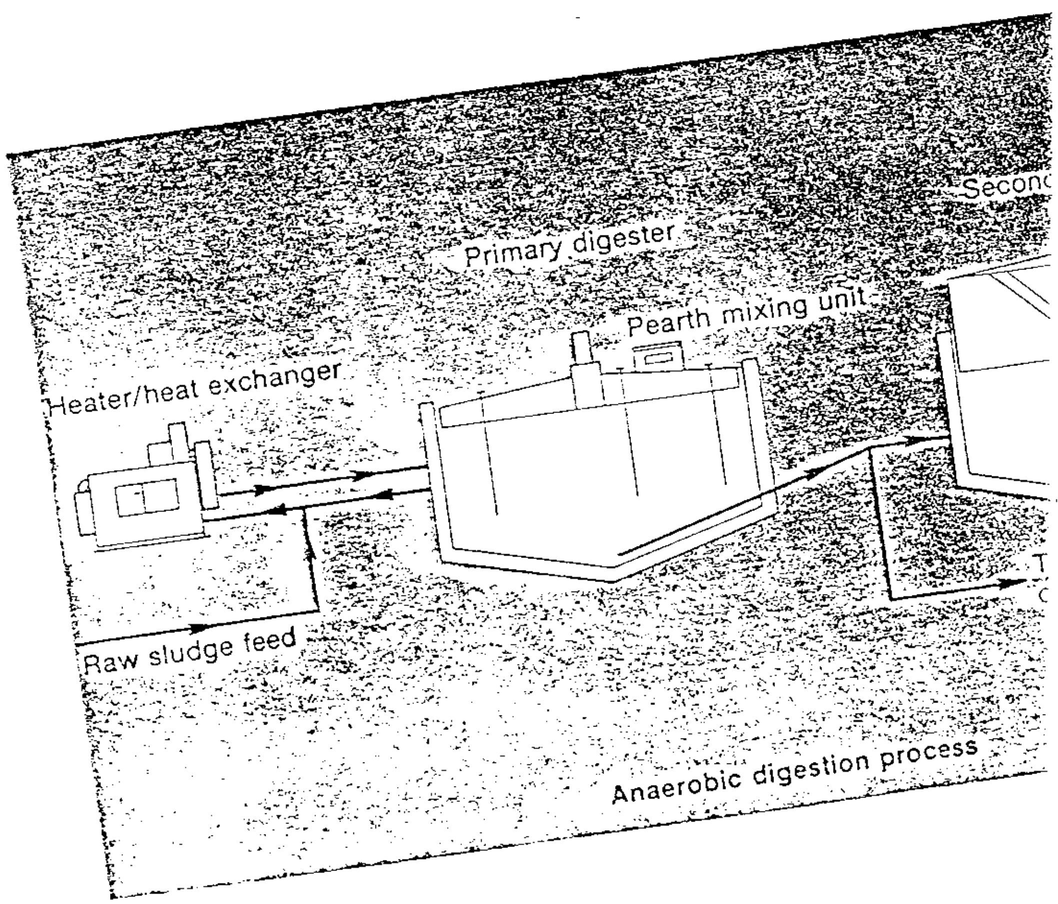
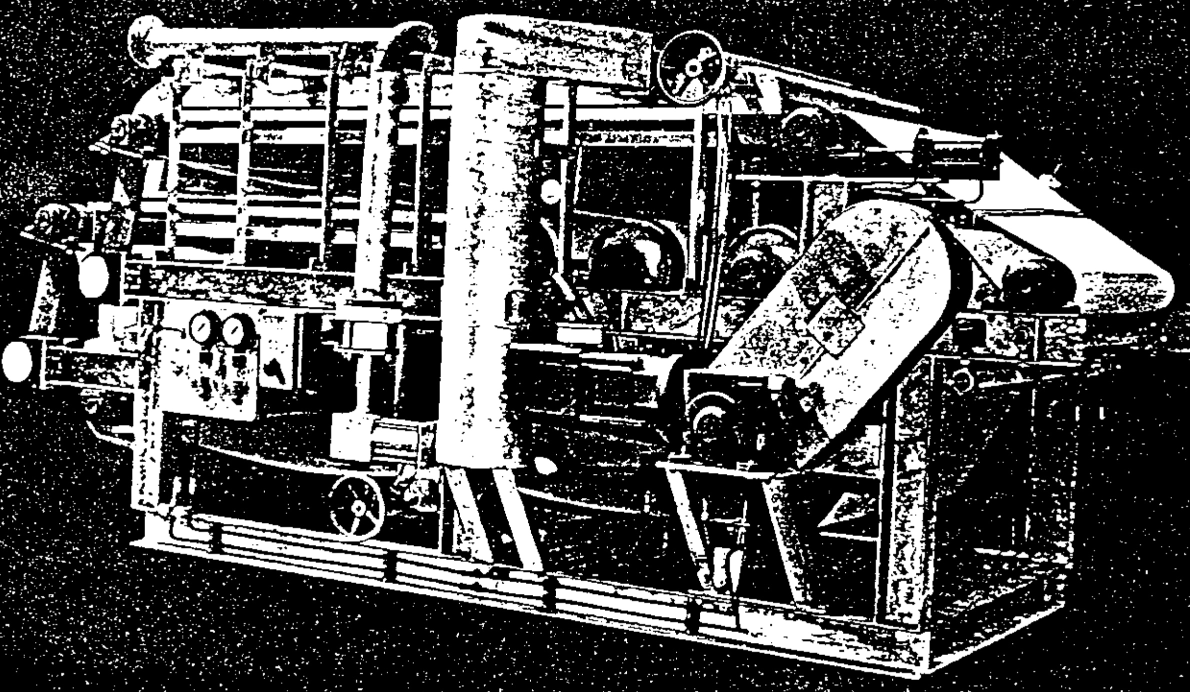


Anaerobic digestion process

EFFICIENT SLUDGE DEWATERING

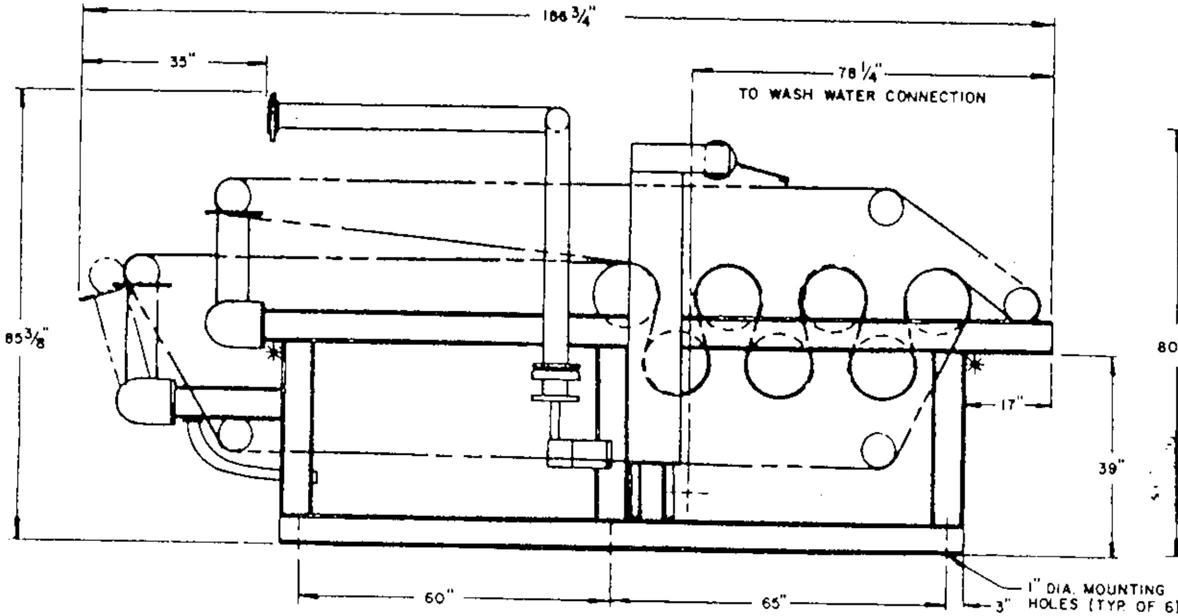
Continuous Belt Presses from Ashbrook-Simon-Hartley

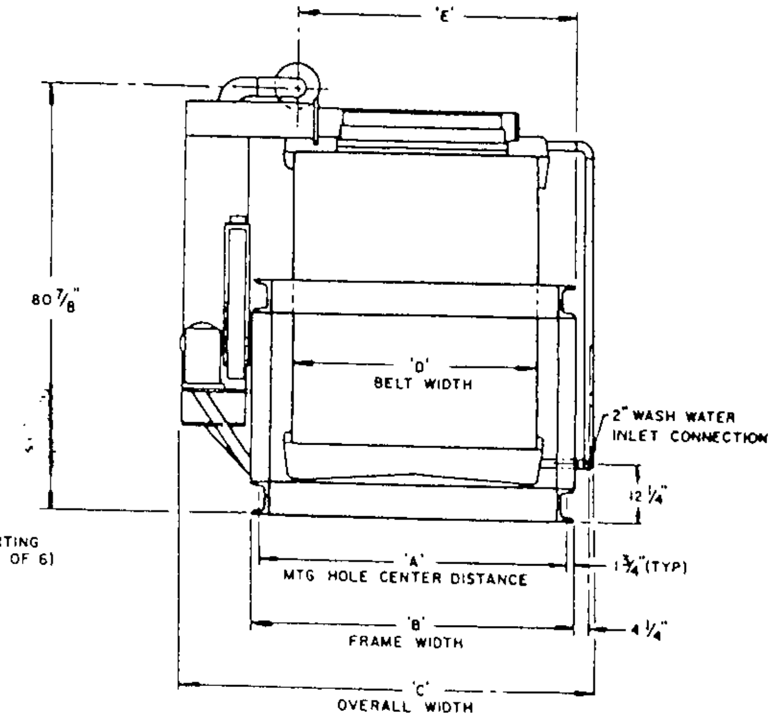


The Ashbrook-Simon-Hartley Model KP Belt Presses provide a better cost-efficiency than centrifuges, filter presses or rotary vacuum drum filters. Each press is delivered as a complete package, ready to produce a dry, manageable sludge cake.

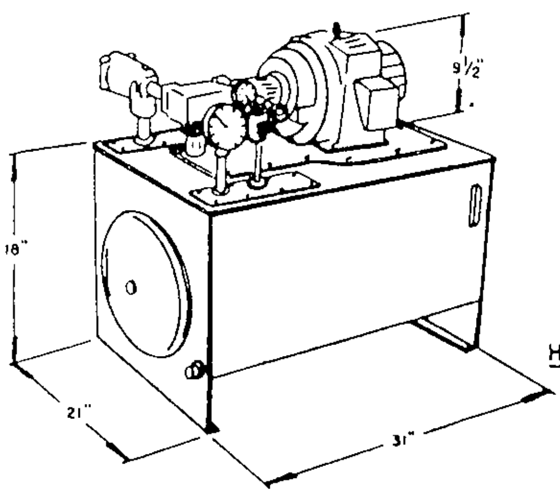
ASHBROOK-SIMON-HARTLEY

TABLE OF WEIGHTS AND SIZES – ASHBROOK·SIMON·HARTLEY MODEL KP BELT PRESS





✱ LIFT POINTS INDICATED (TYPICAL BOTH SIDES)



HYDRAULIC POWER UNIT

EMPTY WEIGHT – 440 LBS.
FULL WEIGHT – 580 LBS.

	POWER REQ'T. ✱✱								
	PRESS DRIVE	HYD. UNIT DRIVE	UNIT WT. (LBS.)	SLUDGE INLET	'A'	'B'	'C'	'D'	'E'
SIZE 1	3 HP	1 1/2 HP	15,000	4"	59 1/8"	62 5/8"	82 5/8"	47 1/4"	55 5/8"
SIZE 2	3–5 HP	1 1/2 HP	19,800	6"	79 3/8"	82 7/8"	102 7/8"	67 1/2"	75 7/8"
SIZE 3	3–5 HP	1 1/2 HP	26,400	6"	98 3/8"	101 7/8"	121 7/8"	86 1/2"	94 7/8"

✱✱ NAMEPLATE HORSEPOWER

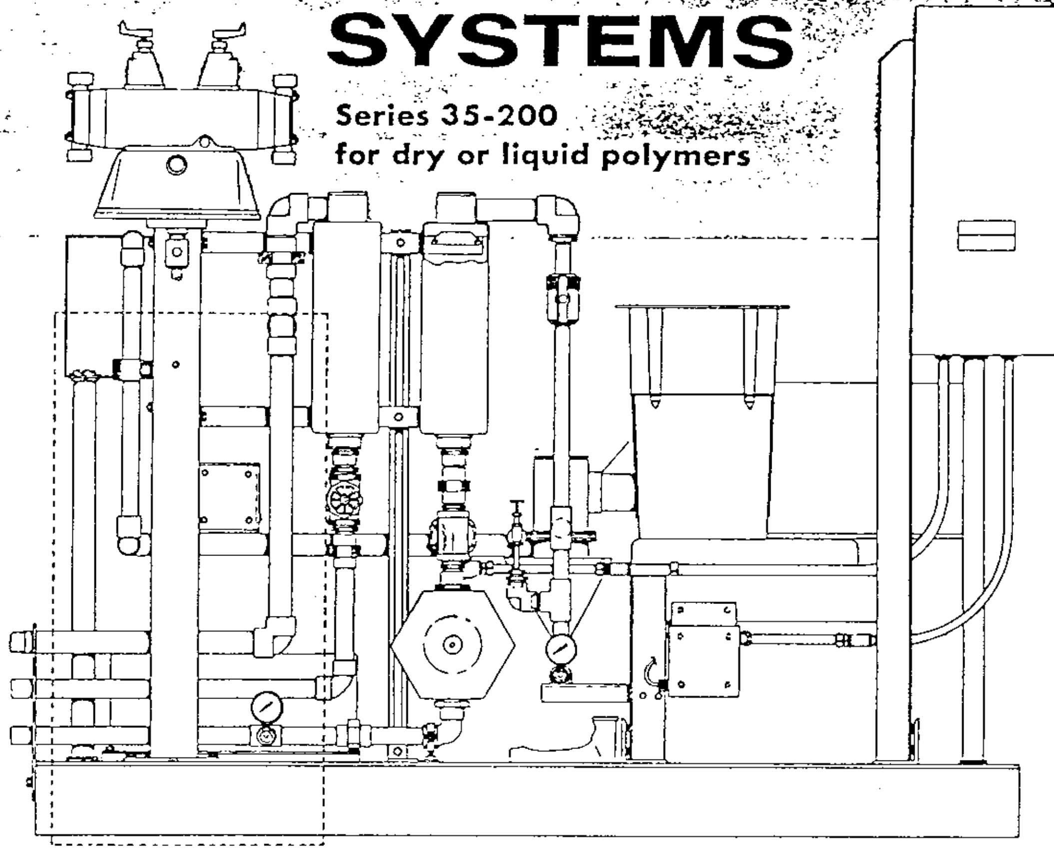
KP1/380

PENNWALT
CORPORATION

WALLACE & TIERNAN

POLYELECTROLYTE SYSTEMS

Series 35-200
for dry or liquid polymers



The standard W&T Polyelectrolyte System automatically prepares and ages solutions of polyelectrolyte from the dry chemical. An optional arrangement of the system prepares and ages solutions from either dry or liquid polymers. Both systems are also designed to dilute the prepared solution to the optimum degree and meter it to the point of application.

Whether for coagulation or flocculation, the polyelectrolytes are very efficient: They reduce to a small fraction the tons of conventional chemical required to do a good job of settling suspended solids. Thus W&T Polyelectrolyte Systems are ideal for potable water treatment, wastewater treatment, or industrial processes in such applications as:

- sedimentation of municipal water, sewage, and industrial wastes;
- settling of hydrous metal oxides in metal-finishing wastewater;
- improves solids capture and supernatant clarity; increases throughput in centrifugation of alum muds;
- gravity settling of steel mill scale, waste pickle liquor, rolling mill wastes, zinc, chromate, latex, and sugar mill wastes, tannery wastewater;
- brine clarification in recovering magnesium compounds from seawater;
- clarification of beet and sugar cane juice.

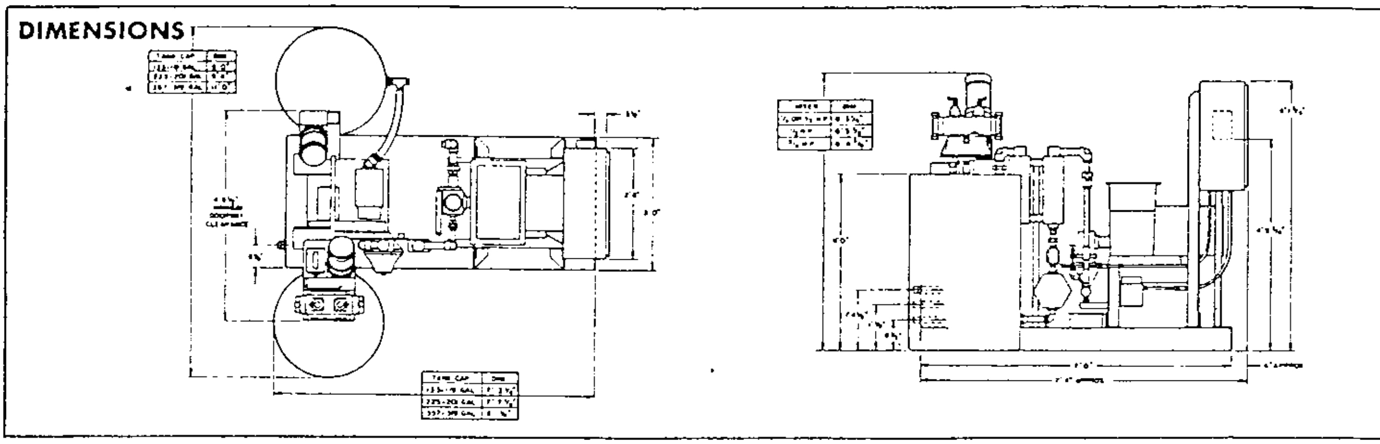
Also:

- thickening of coal refuse; dewatering aid for vacuum filters and drying beds; sludge conditioning for improved dewatering in secondary wastewater treatment;
- as a filter aid by imparting a "charge" to the filter media in filtration of alum muds, sewage sludges, and fermentation broths;
- increases retention of fillers, pigments, and other wet-end additions on cylinder paper machines.

WALLACE & TIERNAN
PENNWALT

TECHNICAL DATA

DIMENSIONS



capacity

System capacity is a function of tank size, solution ageing time, and metering rate. EXAMPLE: For ageing time of 30 minutes and a tank size of 130 gallons (95-gallon batch size), minimum recycling time would be 44.25 minutes (4.75 minutes to fill the tank, 30 minutes to age, and 9.5 minutes to transfer). Thus a new batch of aged solution could be ready for metering about every 44 minutes—if withdrawn that fast. This preparation rate would yield 32.5 batches per 24 hours or 3090 gpd of polymer solution. With a solution concentration of 0.25%, a system of this size is sufficient to treat a 15 mgd flow at a treatment level of 0.5 mg/L. Larger tanks, higher solution concentrations, and/or shorter ageing times will provide increased system capacity.

viscosity limits

133-gallon tank with ⅓ hp mixer: 6000 cps;
225-gallon tank with ½ hp mixer: 7500 cps;
357-gallon tank with ¾ hp mixer: 10,000 cps;
transfer pump: 10,000 cps;

Series 44-747/44-748 Pump with spring poppets; 10,000 cps at 65 spm;

Series 44-115/44-125 Pump with spring-loaded Viton balls: 10,000 cps at 114 spm, 5500 cps at 195 spm.

NOTE: All viscosities are as determined by a Brooksfield viscometer with #2 and 3 spindles at a speed of 3 rpm.

metering pump

Diaphragm type meters aged solution to application point. Capacities of 500, 1,000, 2,500, and 5,000 gpd; discharge pressures to 50 psi, and 10:1 operating range (for induction-motor pump). Corrosion- resistant plastic construction including a nylon-reinforced Hypalon diaphragm, PVC pumping head with other wetted parts made of KYNAR® vinylidene fluoride resin, PVC, Viton, stainless steel, and Monel. Control can be manual with a feed rate adjusting crank or, with optional variable-speed control, manual from the control panel or automatic by 1-5, 4-20, or 10-50 mA dc control input.

optional metering pump

Diaphragm type meters liquid polymer into the ageing tank. Capacities are 500 or 1000 gpd, 10:1 operating range. Manual control with a feed rate adjusting crank. Corrosion-resistant construction.

dry feeder

Screw-type with electric variable-speed control. For the polymer system, 22 combinations of screw sizes, belt pulley steps, and gearbox ratios provide maximum feedrate selections from 0.03 to 7.5 cu ft per hour. Continuous operating range is 15:1. Hopper capacity is 1.6 cu ft. Designed for easy cleaning and corrosion resistance. Upper hopper is fiber glass; hopper-wall agitators are Buna N; the feed screw and trough are stainless steel; the discharge spout is TFE. Manual feed rate control from the control panel with speed (feed rate) readout on a meter calibrated 0-100% of capacity.

water requirements

20 gpm at minimum pressure of 50 psi during wetting-cone operation. Dilution water flow varies with type of polymer and solution concentration required, but maximum is 18 or 53 gpm depending on maximum metering pump capacity.

tanks

130-, 225-, or 350-gallon seamless fiber glass with tight-fitting fiber glass covers and overflow and drain connections.

mixers

Standard and optional mixers have motors geared down to 400 rpm.

transfer pump

Compact, corrosion-resistant slow-speed progressive-cavity pump; capacity is 10 gpm.

custom systems

Custom-designed systems to meet a variety of requirements are also available.

motor characteristics

Dry feeder:	⅙ hp, 115 volt, 60 Hz.
Mixer:	⅓, ½, or ¾ hp, 115 volt, 60 Hz.
Metering Pump:	¼ hp, 115 volt, 60 Hz. (¼ or ½ hp, 0-90 volts dc for SCR drive).
Optional metering pump:	¼ hp, 115 volt, 60 Hz.
Transfer Pump:	¾ hp, 115 volt, 60 Hz.

power requirements

60 amperes (two 30-ampere circuits).

shipping weight

2800 lb.

REFERENCES

Literature on system components and related distribution equipment is available on request.

Progressive changes in design may be made without prior announcement.

WALLACE & TIERNAN DIVISION
PENNWALT CORPORATION
25 MAIN STREET

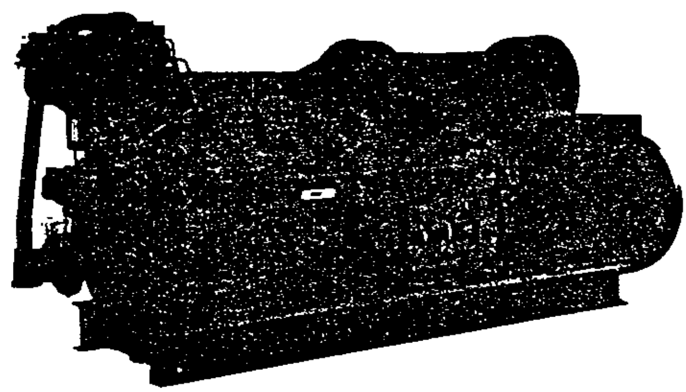
WALLACE & TIERNAN
PENNWALT

CATERPILLAR GENERATOR SETS

3512
800 kW PRIME — 1000 kW STANDBY

FEATURES

- **CAT DIESEL GENERATOR SETS**
Factory Designed . . . assembled . . . tested and delivered to you in a package that is ready to be connected to your fuel and power lines . . . supported 100% by your Caterpillar Dealer.
- **RELIABLE, FUEL EFFICIENT DIESEL**
The compact, four-stroke-cycle diesel engine combines durability with minimum weight while providing dependability and economy. The fuel system operates on a variety of fuels.
- **THE CAT GENERATOR**
Single-bearing wye connected brushless generator designed to match performance and output characteristics of the Caterpillar Diesel Engine that drives it.
- **BROAD LOAD ACCEPTANCE**
Regulator features three phase sensing . . . Precisely monitors and regulates output to maintain excellent control.



Arrangement may be shown with optional equipment.

STANDARD ARRANGEMENT

Engine:
Air Cleaner with
  Service Indicators
Breather, Crankcase
Cooler, Lubricating Oil
Exhaust Fitting and Flange
Filters
  Fuel, RH with
    Service Indicators
  Lubricating Oil
Flexible Fuel Lines
Flywheel
Flywheel Housing, SAE No. 00
Governor, Electronic 2301
  Speed Control/EG10P
Instrument Panel
  Engine Oil Pressure
  Fuel Pressure
  Oil Filter Differential
  Jacket Water Temperature

NOTE: Engine wired for
  Automatic Start-Stop.

Lifting Eyes
Manifold, Exhaust, Dry
Pumps, Fuel and Lubricating
  Fuel Priming
  Fuel Transfer
  Lubricating Oil
Pumps, Water
  Jacket Water
Rails, Mounting
Service Meter
Shutoff, Electrical
  Oil Pressure, Water
  Temperature and Overspeed
Shutoff, Manual
Tach Drive, Dual

Generator:
SR4 Brushless
  with Voltage Regulator

ACCESSORY EQUIPMENT

Engine:
Air Cleaners, Heavy-Duty
Cooling Systems
Exhaust Fittings
Flywheel and Housing,
  SAE No. 0
Governor, Woodward UG8D
Instrument Panel, Premium
Power Takeoffs
Protection Devices
Starting, Air and Electric
Tachometer Drive, Dual

Generator:
Current Boost System
Manual Voltage Control
Space Heater
Temperature Rise Detectors

Switchgear:
Automatic Start-Stop
Battery Charger
Circuit Breaker
  Manual
  Electric Operated
Enclosure-Floor Standing
  NEMA 1
Main Load Bus
Paralleling
  Manual
  Permissive
Protective Relays

GENERAL SPECIFICATIONS

CAT 3512 ENGINE
1800 RPM
Type—Watercooled Diesel
Aspiration—Turbocharged-
  Aftercooled
Cycle—Four-Stroke
No. of Cylinders—V12

Bore—6.7 in (170 mm)
Stroke—7.5 in (190 mm)
Piston Displacement—
  3158 cu in (51.8 liter)

CAT SR4 GENERATOR
60 Hertz
Type—Brushless, Revolving field, Solid-State Exciter
Construction—Single Bearing—Close Coupled
Phase—3
Wire, Connection— 4 Wire (689 Frame), Wye
  10 Wire (685 Frame), Wye
Meets or exceeds NEMA MG 1-22 std. requirements
Insulation—Class F with tropicalization & anti abrasion
Three Phase Sensing
Enclosure—Drip Proof
Alignment—Pilot Shaft
Overspeed Capability—125%
Wave Form—Less than 5% deviation
Parallel Capability—Standard
Voltage Regulator—Generator Mounted, Volts per Hertz
Voltage Regulation—Less than ± 1%
Voltage Droop—Adjustable for parallel operation
Voltage Gain—Adjustable to compensate for engine
  speed droop and line loss

VOLTAGES AVAILABLE

139/240 (685 Frame Only)
277/480, 346/600
(Adjustable a minimum of +5% −10%)

3512 GENERATOR SET 800-1000 kW

CATERPILLAR

PRIME
800 kW — 1000 kV•A w/fan
840 kW @ 0.8 PF without fan
1188 Engine HP without fan*

STANDBY
1000 kW — 1250 kV•A w/fan
1040 kW @ 0.8 PF without fan
1471 Engine HP without fan*

FUEL RATE DATA

	Prime				Standby			
PERCENT LOAD	25	50	75	100	25	50	75	100
kW with Fan	200	400	600	800	250	500	750	1000
gal/hr	21.5	34.0	47.5	60.5	25.1	40.5	56.7	74.2
liter/hr	81.4	128.7	179.7	229.0	94.8	153.3	214.6	280.9

DERATION DATA

		Prime				Standby			
AMBIENT TEMPERATURE	°F	68	86	104	122	68	86	104	122
	°C	20	30	40	50	20	30	40	50
ALTITUDE	feet	9252	8465	7546	6726	3118	2297	1312	492
	meter	2820	2580	2300	2050	950	700	400	150

TECHNICAL DATA

		SI METRIC			ENGLISH		
	Rating Type		PRIME	STBY		PRIME	STBY
Rating Information	Power Rating @ 0.8 PF w/Fan	kW	800	1000	kW	800	1000
	Power Rating @ 0.8 PF w/o Fan	kW	840	1040	kW	840	1040
Cooling System	Engine Coolant Capacity w/o Radiator	L	147.6	147.6	gal	39.0	39.0
	Engine Coolant Capacity with Std. Rad.	L	333.5	333.5	gal	88.1	88.1
	Standard Radiator Arrangement Data:						
	Air Flow (Max. @ Rated Speed)	m^3/min	1996	1994	cfm	70,400	70,400
	Air Flow Restriction (Max. Allowable)	kPa	0.12	0.12	in H_2O	0.5	0.5
	Ambient Air Temperature (Max. Allowable)	°C	55	55	°F	130	130
	Coolant Pump External Resistance (Max. Allowable)	m H_2O	11.1	11.1	ft H_2O	36.4	36.4
	Coolant Pump Flow @ Max. Allowable Resistance	L/min	1200	1200	gpm	317.0	317.0
Exhaust System	System Backpressure (Max. Allowable)	kPa	6.7	6.7	in H_2O	27	27
Mounting System (Eng., Gen. & Rad.)	Length Overall	mm	5102	5102	in	200.86	200.86
	Height Overall	mm	2555	2555	in	100.57	100.57
	Width Overall	mm	2223	2223	in	87.50	87.50
	Unit Dry Weight	kg	9545	9545	lb	21,043	21,043
Performance Data @ Rated Conditions	Combustion Air Inlet Flow Rate	m^3/min	87.5	100.3	cfm	3090	3545
	Exhaust Gas Flow Rate	m^3/min	215.5	285.0	cfm	7610	9120
	Exhaust Gas Stack Temperature	°C	470	500	°F	878	932
	Heat Rejection to Coolant (Total)	kW	563	710	Btu/min	32,018	40,377
	Heat Rejection to Exhaust (Total)	kW	911	1143	Btu/min	51,808	65,037
	Heat Rejection to Atmosphere From Engine	kW	130	156	Btu/min	7393	8876
	Heat Rejection to Atmosphere From Generator	kW	41.6	53.0	Btu/min	2367	3016

CONDITIONS & DEFINITIONS

Standby — For continuous electrical service during interruption of normal power.
Prime — For continuous electrical service.
Performance is based on SAE J1349 standard conditions of 100 kPa (29.61 in Hg) and 25°C (77°F). Peformance also applies at ISO 3046/1, DIN 6271 and BS 5514 standard conditions of 100 kPa (29.61 in Hg), 27°C (81°F) and 60% relative humidity.

Fuel consumption is based on fuel oil having an HHV of 45 570 kJ/kg (19,590 Btu/lb) and weighing 848 g/liter (7.076 lb/U.S. gal).

No engine deration is required for ambient temperature up to 50°C (122°F) except as shown on the deration data chart.

These capability charts apply to the engine only and include considerations for humidity. If air cleaner inlet conditions exceed the appropriate standard conditions, consult your Caterpillar Dealer for necessary deration.

*Fuel stop power ISO 3046/1 or DIN 6271 or BS 5514.

Materials and specifications are subject to change without notice.
The International System of Units (SI) is used in this publication.

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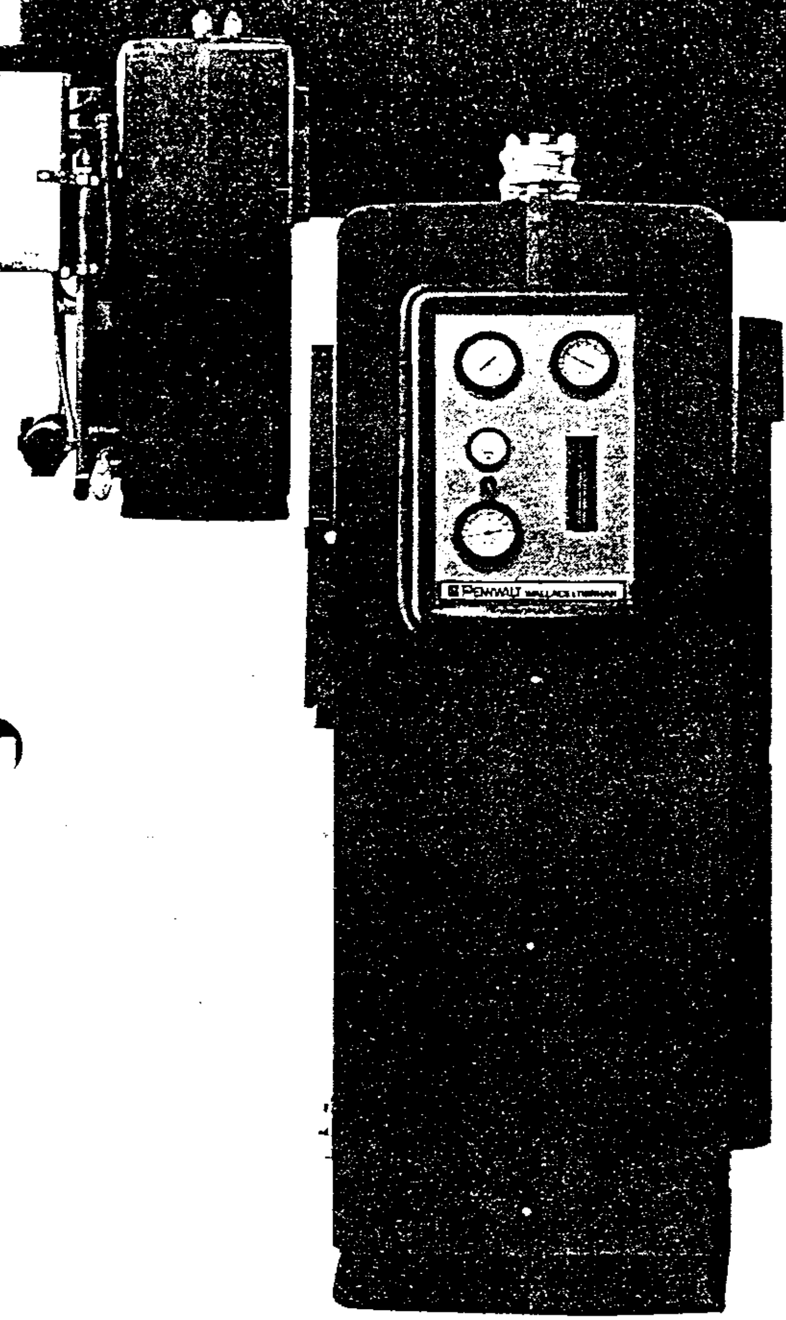
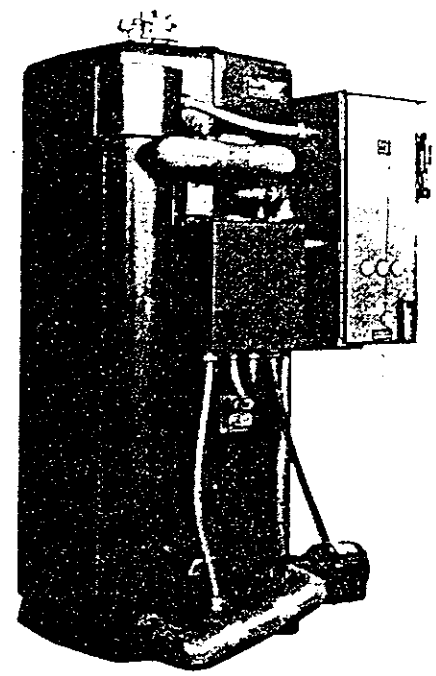
Printed in U.S.A.

PENNWALT
CORPORATION



The Wallace & Tiernan Series 50-202 Evaporator is an immersed-tank heat exchanger designed specifically to evaporate liquid chlorine, sulfur dioxide, or ammonia. Heat from a hot water bath converts liquid to gas. Operation is fully automatic; the system is simple and direct. It is designed to provide effective service with minimum maintenance.

The evaporator (sometimes called a "vaporizer") comes in two types: An electric type in which water for the bath is heated by an external heater and is circulated. A steam or hot water type gets hot water for the bath from an external steam or hot water heat exchanger or from some other hot water source.



Optional pre-wired electric evaporator with junction box in center and combination contactor at upper right. Electric water-bath heater is at left.

WALLACE & TIERNAN
PENNWALT
EQUIPMENT ■ CHEMICALS
HEALTH PRODUCTS

DIMENSIONS

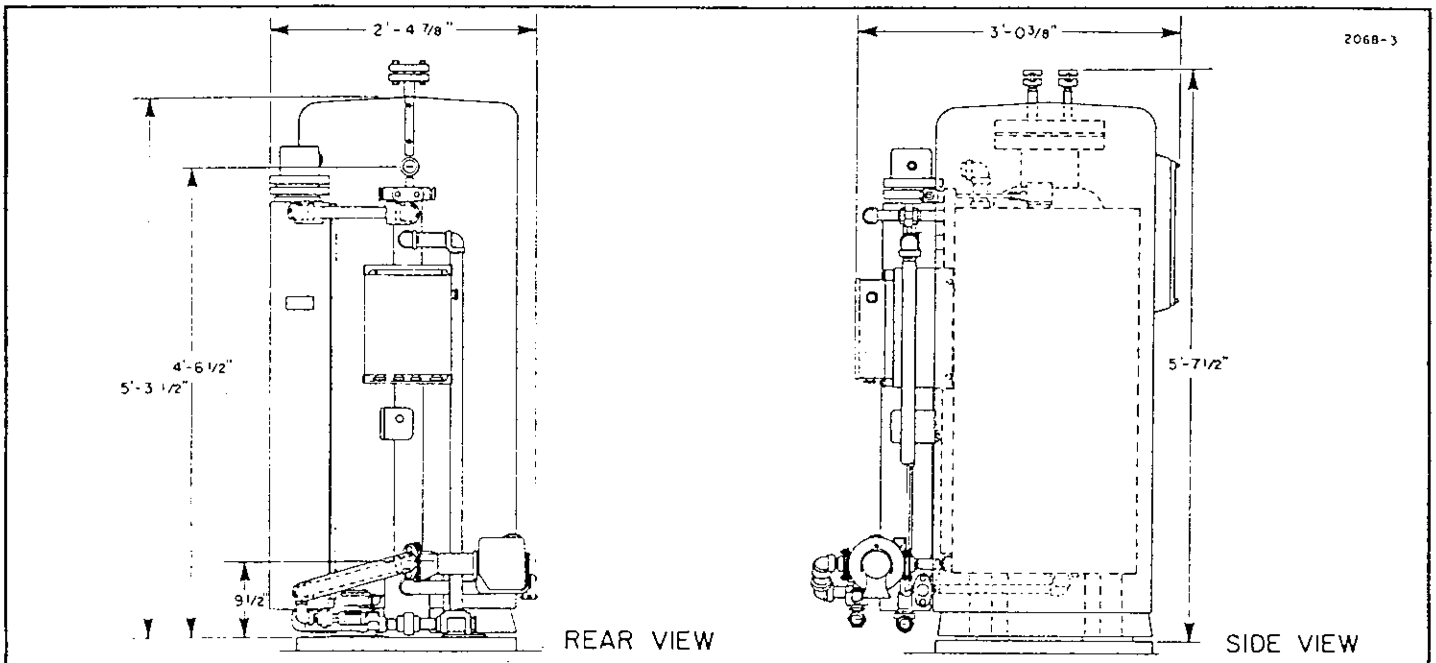
line pressure relief system; combination contactor (electric heater type only) in a NEMA 12/3 enclosure; liquid automatic switchover system for ton containers or tank cars. Both evaporator types can be factory pre-wired; the electric heater type can be ordered in a weather-resistant outdoor arrangement which can be pre-wired.

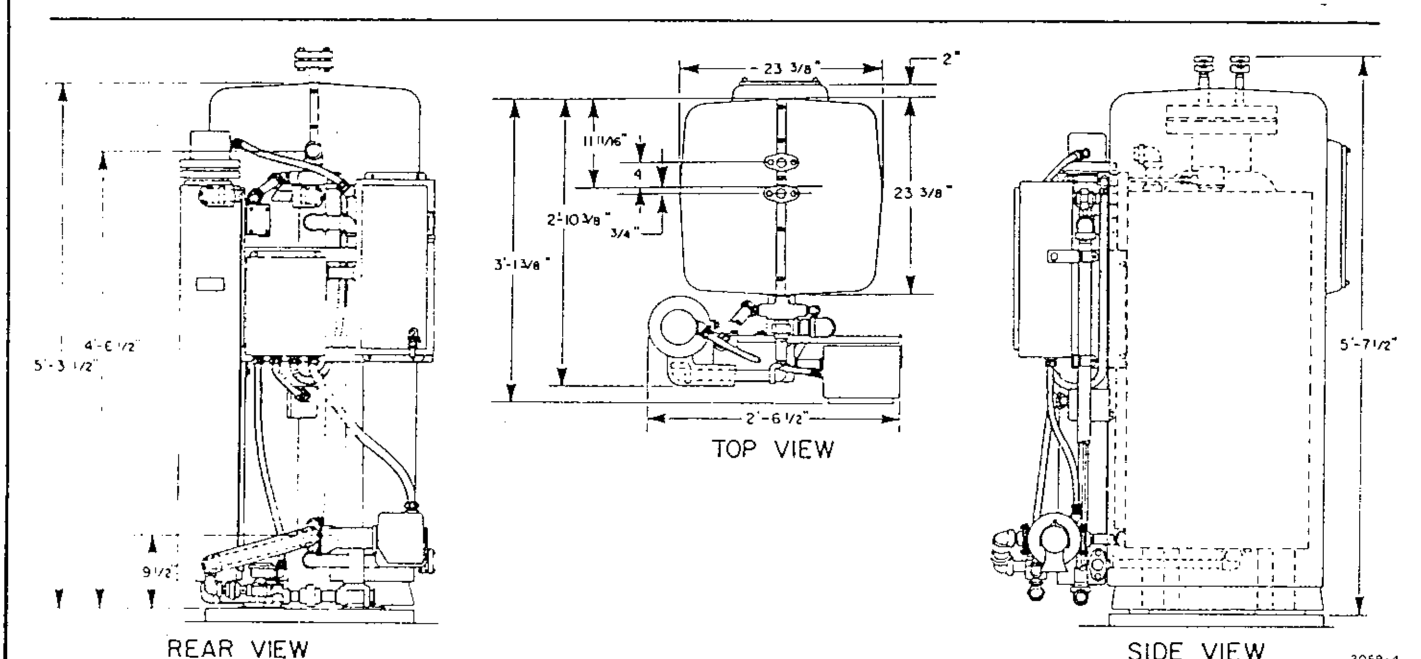
Optional but necessary for operation of the hot water type: a thermostat-controlled valve for steam or hot water; steam or hot water heat exchanger; hot water circulating pump; steam trap; steam relief valve; steam pressure regulating valve.

shipping weight Approximately 800 lb.

SERVICE AND REFERENCES

Prompt service on W&T equipment is available from branch offices in principal cities. More technical data available in other publications. Also see paragraph 2.2.5.3, "Liquid Discharge," in the Chlorine Institute's *Chlorine Manual* for operating recommendations on liquid chlorine supplies.





Progressive changes in design may be made without prior announcement.

WALLACE & TIERNAN DIVISION
PENNWALT CORPORATION
25 MAIN STREET
BELLEVILLE, NEW JERSEY 07109

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HEALTH PRODUCTS

Printed in USA

PENNWALT
CORPORATION

Left: 8000-lb Module
Middle: 2000-lb Module
Right: 500-lb Module

The Wallace & Tiernan Remote Vacuum V-800 Chlorinator is an important advance in the technology of chlorine application. It features the precision feeding of the V-notch Variable Orifice, virtual non-venting operation, and remote vacuum control. Automatic switchover arrangements and a diverse selection of automatic and remote control options are available. Feed rate can be selected from a wide range of capacities. Control components are housed in functional, modular cabinets engineered for easy installation.

With remote vacuum control, the chlorinator's system operates entirely under vacuum. W&T's unique vacuum regulator—pressure check unit reduces the gas pressure to a vacuum right at the supply source. It also results in gas-flow regulation that is essentially free from venting. Optional automatic switchover arrangements can automatically shift the chlorinator to a fresh supply without interrupting service.

The Series V-800 Chlorinator design complies with the recommendations of the Chlorine Institute. It also conforms to applicable current standards of the National Electric Code and the National Electrical Manufacturers' Association.

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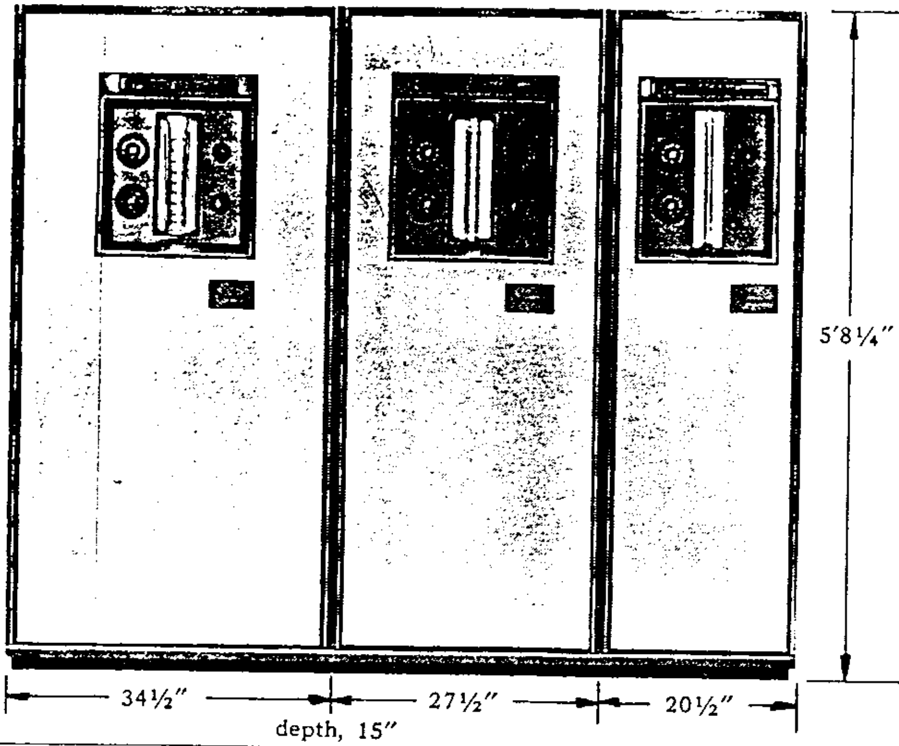
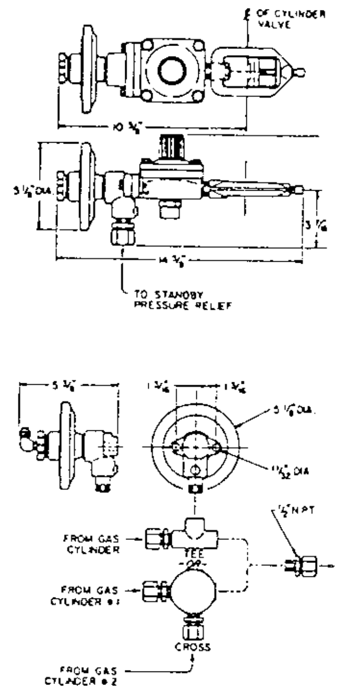
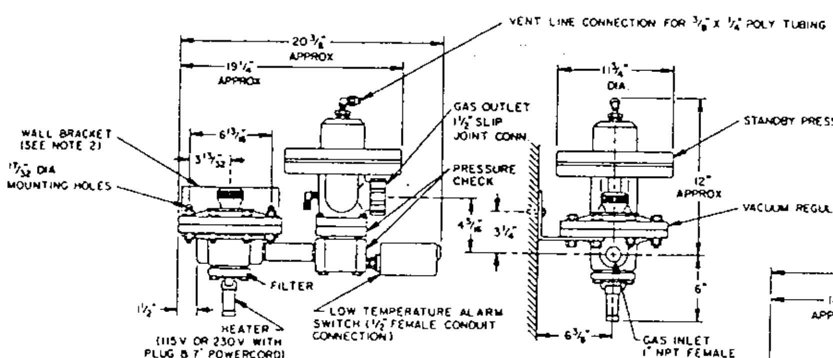
DIMENSIONS





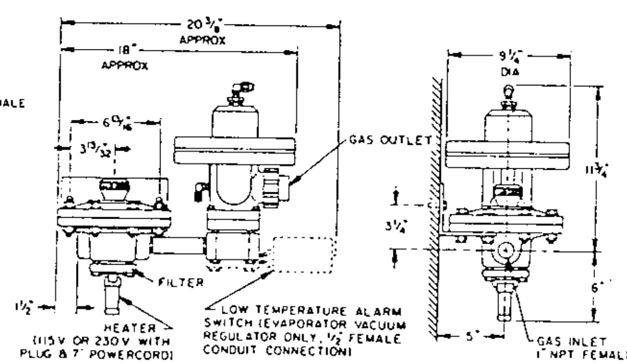
500-lb Module 120.073



EVAPORATOR VACUUM REGULATOR-CHECK UNIT

SIDE VIEW

8000-lb Module



VACUUM REGULATOR-CHECK UNIT

2000-lb Module

120.341

120.133

Progressive changes in design may be made without prior announcement

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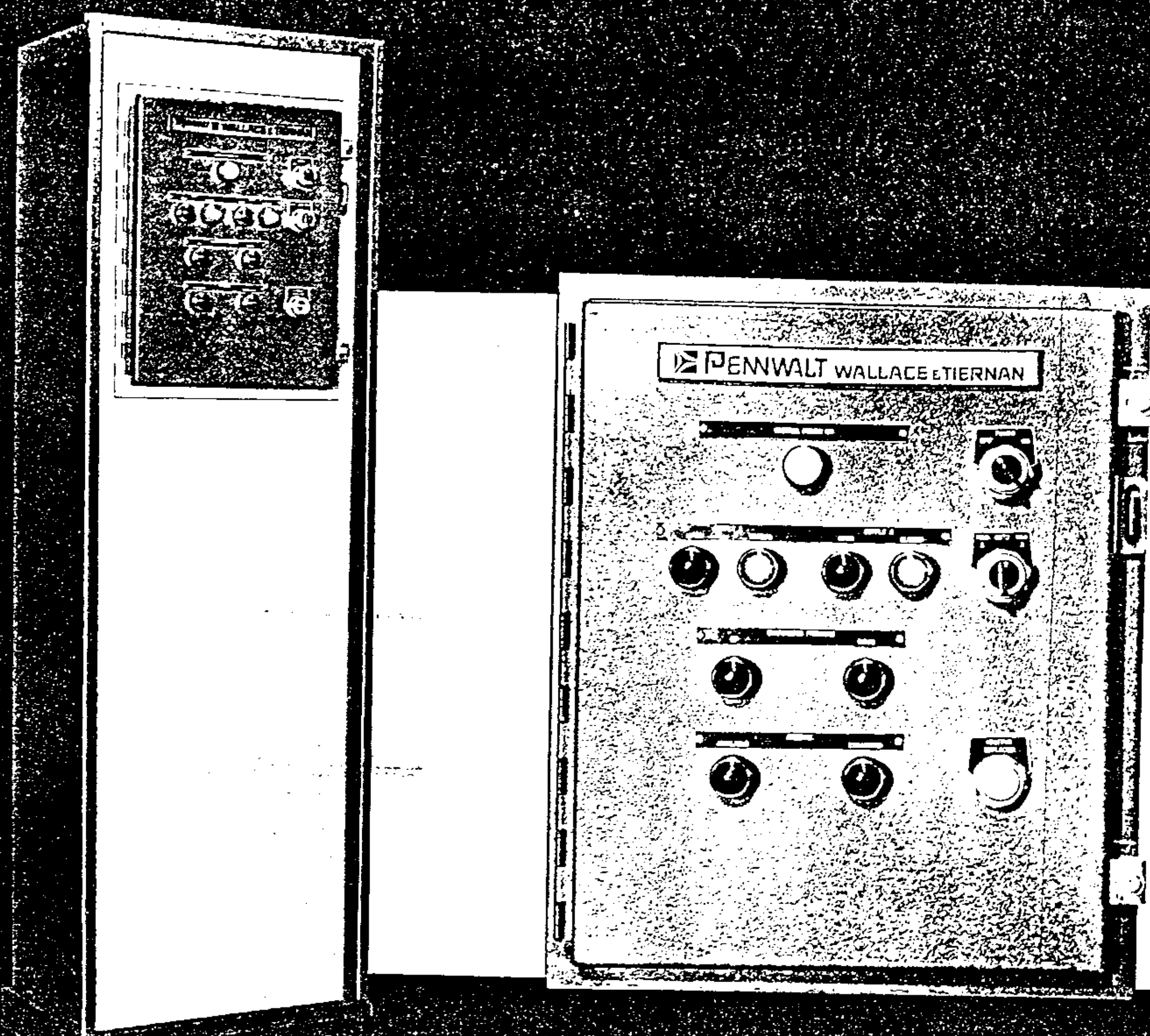
WALLACE & TIERNAN
PENNWALT

PENNWALT
CORPORATION



This liquid-handling system is designed to give automatic changeover to a new supply when the on-line supply runs out. For use ahead of evaporator-chlorinator combinations, the system consists of two motorized ball valves and a diaphragm-protected pressure switch. These units are monitored and controlled from a control panel. Changeover is fully automatic; but manual changeover can be accomplished by a selector switch on the control panel. For use on ton containers and other large supplies, the system is capable of supplying chlorine to a maximum of 10 manifolded evaporators. It can also be used on liquid ammonia or sulfur dioxide supplies.

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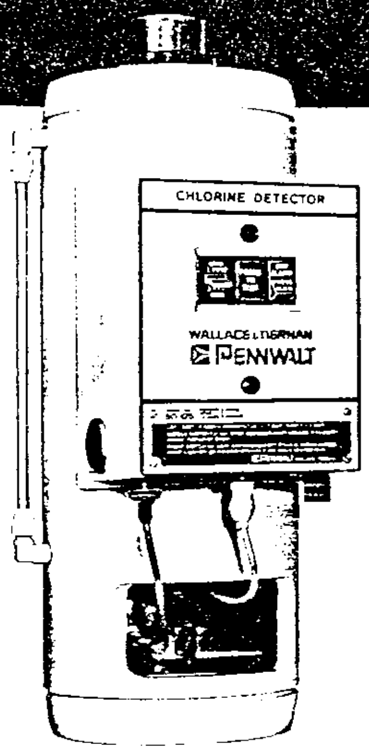
CORPORATION



Wallace & Tiernan Series 50-125 Chlorine Detector is designed to react quickly to the presence of chlorine in ambient air. The sensor will respond within seconds to 1 ppm of chlorine. This response is in accord with present OSHA regulations which restrict chlorine exposure to a ceiling limit of 1 ppm. If an alarm prior to reaching the ceiling limit is required, the sensitivity level of the detector can be converted easily in the field to respond within seconds to 0.5 ppm of chlorine.

Simplicity, in both design and operation, puts this unit in a class apart from the complex instrument-type detectors. It's the first truly uncomplicated chlorine detector ... dependable, economical, sensitive, easy to operate and to maintain.

FEATURES

SELF CLEANING SENSOR

The measuring electrode is continuously washed by gravity flow of the electrolyte. This minimizes the possibility of dirt and contaminants interfering with the sensor reacting to chlorine.

ADJUSTABLE RESPONSE LEVEL

Two sensitivity levels are provided with each detector, 1 ppm or 3 ppm. The level of operation is selected by a switch on the control panel. The sensitivity levels can be easily converted in the field to 0.5 ppm and 3 ppm. Alarm circuitry can be checked for all response levels with test button on front panel of detector.

POSITIVE/SAMPLING,/RAPID RESPONSE

A fan built into the wall-mounted detector arrangement, or a remote blower unit used with the panel-mounted and module detector arrangements, provides positive air sampling. The sensor is kept in continuous contact with the ambient air and is not dependent on room "air currents" to bring the air sample to the sensor. The presence of 0.5 ppm, 1 ppm or 3 ppm of chlorine is detected within seconds.

AUTOMATIC RESET

The Series 50-125 Detector can be wired to activate ventilating fans automatically in chlorine-alarm-level situations. Fans will operate until chlorine level is less than set sensitivity limit. The detector will reset (return to normal alarm readiness) automatically at this same condition.

POWER-FAILURE ALARMING

In the event of a power interruption (greater than 0.5 seconds) or a blown fuse, the detector automatically goes into an alarm state. (Alarm and power-on lights will not be lit; alarm contacts will be closed.) Ventilating fans and audible alarms will be activated. Upon restoration of power, the detector will automatically reset to alarm-ready state. Power-on light will go on. Detector will alarm immediately if chlorine has reached alarm level during outage.

LOW COST OPERATION, MINIMUM MAINTENANCE

Design simplicity helps give low initial cost, dependable operation. Minimum power required. The fuse is easily replaced; plug-in indication lights are replaced without soldering. The reservoir holds a normal 6 weeks' electrolyte supply; it is easily replenished without disassembly or interrupting service. A sight gauge shows electrolyte level at a glance. To determine if the detector is ready to alarm, it is usually necessary only to check the alarm circuitry with the test button and to see that electrolyte is dripping from the electrode.

DESIGN AND OPERATION

The W&T Chlorine Detector consists of an electrolyte tank with a level indicator and an air filter. The activated-carbon filter keeps chlorine gas away from the electrolyte. From the bottom of this tank a sensor projects down into a sensing chamber where it contacts sample air driven by the fan or blower.

The sensor is a plastic holder containing two platinum electrodes. Electrolyte drains slowly down the holder keeping it constantly wet and continuously washing off dirt and contaminants. Excess solution drops into a tray; some of it evaporates and the remainder drains through plastic tubing.

When chlorine-laden air enters the sensor chamber, chlorine reacts with the electrolyte at the electrodes to produce an electrical current. The current is amplified in the solid state electronic unit to light a built-in red alarm and denergize a double-pole, double-throw relay. The relay contacts are wired to a terminal strip to permit pick-up of a contact opening or closure for operating fans, chlorine shut-off valves, or external alarms.

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SPARLING

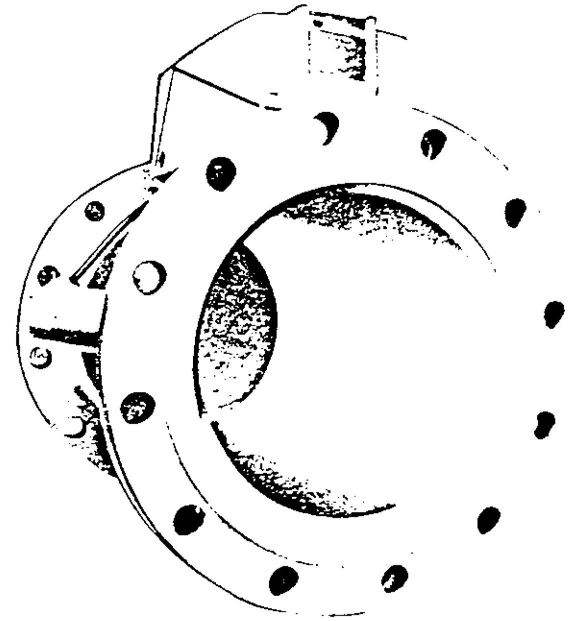
SERIES 500*
FLOWMETER
SYSTEM

SERIES 500 FLOWMETER SYSTEM

FS500/FS501 – Flow Sensor
FT500/FT501 – Flow Transmitter

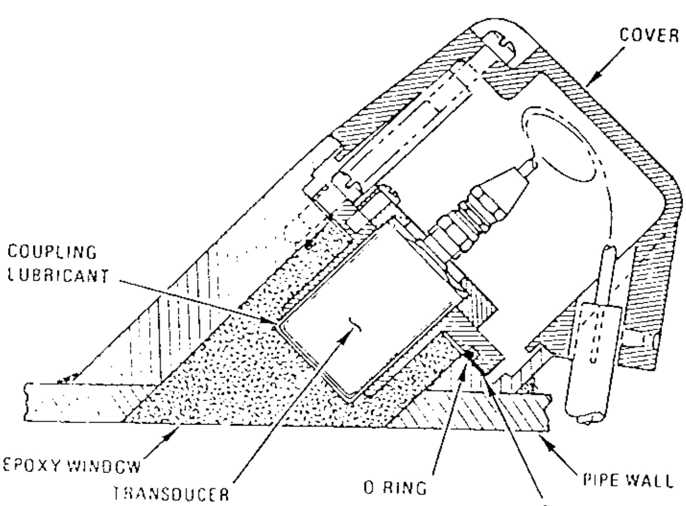
PDS-500
127750
Supersedes 037740

The Series 500 is a completely obstructionless, highly accurate, flow-measuring system that can be easily installed even in existing piping. Two inter-related components comprise the system: (1) the sensor, consisting of a precalibrated, fused epoxy-coated flow tube containing a pair of externally mounted electro-acoustic transducers, and (2) the transmitter or electronics package. Flow tubes are available in sizes ranging from 4" to 48." Output is a pulse rate calibratable from 0-10 PPH to 0-2500 PPS and simultaneously a 0-20 or 4-20mA. Performance of the system is unaffected by most solids in suspension; pipe wall coatings; fluid conductivity, density, and temperature; entrained air; velocity of sound in fluid; and process pressure, including full vacuum. The completely open-bore design of the sensor prevents the possibility of fouling by objects in the process stream and eliminates pressure loss. Applications for this advanced flow-measuring system include water, raw sewage and most sludge flows found in municipal water/waste treatment plants



NON-CONTACT FLUID MEASUREMENT

The transducers mount in an epoxy well to effectively isolate them from the process stream. Since the transducers are not in contact with the fluid, probe contamination and corrosion are never a problem. Transducers are easily replaced without process shutdown. The absence of liners precludes problems associated with steam cleaning and abrasion.



EXCLUSIVE PATENTED CIRCUITS

Utilizing high-speed sampling techniques and strobed signature pulses which discriminate when obstructions appear in the flow stream, an accuracy of +1% of actual flow from 1 to 40 FPS is attained. The upstream and downstream sampling rates are each 150 samples-per-second. Since only 2% of the sample pulses need to be received for the system to maintain its rated accuracy, momentary blockages and attenuation of pulses caused by air bubbles or solids in the flowing stream do not affect system accuracy.

Pipe wall coatings can attenuate the pulses. However, exclusive circuitry allows the reception of severely attenuated pulses. Thus, even thick pipe-wall coatings will not preclude system operation.

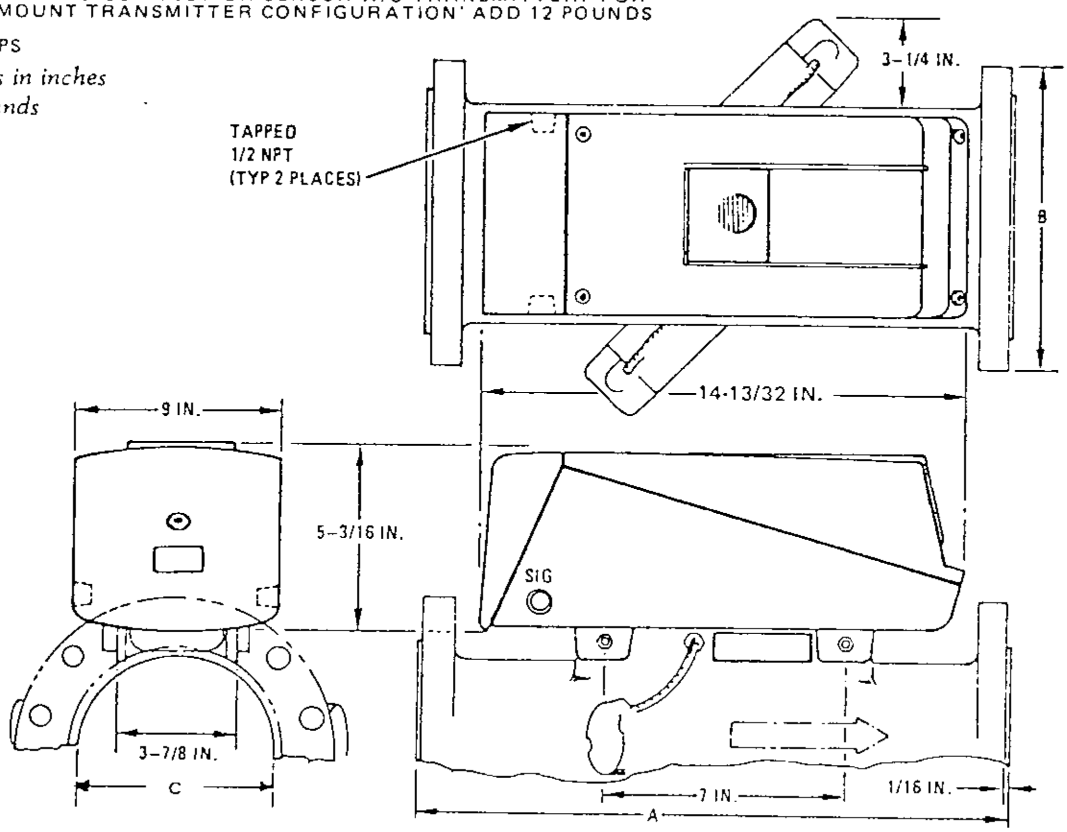
In many flow-measuring devices, large errors, introduced by plugged taps, partially coated electrodes, and other inherent deficiencies will be undetected. This cannot occur with the Series 500 Flowmeter System. In the event of signal loss for a period of time that would affect accuracy, a diagnostic light will illuminate. Thus, the light will let you know if process conditions are hindering performance. The same light will also illuminate in the event of most circuit malfunctions.

SIZE	"A" FLANGED TUBES VICTAULICS	"A" PLAIN ENDS	"B" FLANGED	"B" VICTAULIC PLAIN	"C" PIPE ID	NOMINAL SHIPPING WEIGHT FLANGED *	NOMINAL SHIPPING WEIGHT VICTAULIC PLAIN	VELOCITY CONVERSION (K)
4	20	26	9	4.5	4.026	65	40	0.02521
6	20	26	11	6.6	6.065	80	45	0.01111
8	20	28	13.5	8.6	8.070	120	65	0.006276
10	20	28	16	10.8	10.136	155	80	0.003978
12	25	28	19	12.8	12.090	245	125	0.002796
14	25	28	21	14	13.625	245	140	0.002202
16	25	30	23.5	16	15.625	290	170	0.001674
18	25	30	25	18	17.500	390	195	0.001335
20	30	30	27.5	20	19.500	500	215	0.001075
24	30	30	32	24	23.375	590	255	0.0007480
30	35	38	38.75	30	29.25	986	400	0.0004777
36	40	42	46	36	35.25	1435	530	0.0003289
42	45	48	53	42	41.00	2243	920	0.0002431
48	50	54	59.5	48	47.00	2782	1200	0.0001850

* 150 LB ANSI RF END CONNECTION SENSOR W/O TRANSMITTER. FOR INTEGRAL MOUNT TRANSMITTER CONFIGURATION ADD 12 POUNDS

† GPM (K) = FPS

Measurements in inches
Weight in pounds



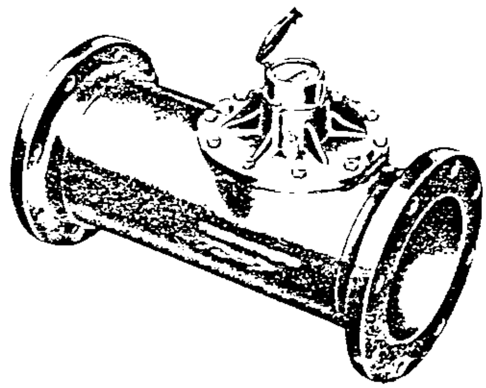
SERIES 100
METERS AND
ACCESSORIES

SERIES 102 DIRECT DRIVE METERS
150 and 250 psi Max. Working Pressure
2" thru 14"

PDS-102
8/10/79

The Series 102 Direct Drive Meters are designed to provide accurate and reliable flow measurement where mainline service is required in the municipal and industrial areas. These meters have been used for over 60 years and have proven their reputation for rugged, continuous duty with minimum maintenance.

The meter is available as a meterhead only or a meterhead complete with tube or saddle.



Typical Tube Type Meter

APPLICATIONS

The Series 102 Meter is ideally suited for all water flow applications where the temperature of the water does not exceed 100° F and the suspended solids do not exceed 0.5%. (Applications to other fluids, or outside the foregoing limits, are possible although factory engineering must be consulted before proceeding.

ACCURACY

Accuracy of all Series 102 Meters is within 2% of actual flow for the specified meter range. This accuracy is guaranteed by certified wet calibration.

FLOW

These meters measure accurately over a wide flow range of 10:1 or greater. The maximum flow ranges can be safely exceeded by 50% when used intermittently. Registration of flow is shown on a 6-digit direct-reading register which can be furnished in any standard units (i.e. gallons, cubic feet, etc.). Flow rates are provided in the table under specifications for each of the sizes.

TEMPERATURE LIMITS

Fluid working temperature should not exceed 100°F. Propellers may be stored in air temperatures up to 175° F.

OPTIONS

High Velocity Flows – Where continuous flow rates are above the standard flow range ratings, high velocity construction of the meters is available. Special propellers and bearings are supplied to ensure continuous trouble-free operation.

Transmitters – Electronic transmitters are available for installation on these meters.

Rate-of-Flow Indicators – When continuous rate of flow indication is required, an optional rate-of-flow indicator and totalizer is available.

Forward–Reverse Flows – Special tubes are available for forward–reverse flows. These tubes contain straightening vanes both upstream and downstream from the meter propeller. A forward–reverse totalizer is used in place of the standard register to totalize flows in both directions.

Hot Water Service – Meters are available with special propellers, packing, and registers that can withstand continuous operating temperatures to 300°F.

TUBE TYPE METERS

END CONNECTIONS

All meters 2" thru 14" are standard with flanged ends for either 150 psi or 250 psi service. The flanged ratings are shown in table 1. Meters 6"thru 14" are available with either plain ends or grooved ends for victaulic couplings.

Meters 2" thru 4" are available with National Pipe Thread end connections. These meters are rated for 150 psi service.

STRAIGHTENING VANES

All meters, except the 2 and 3 inch, are supplied with straightening vanes to preclude the possibility of errors due to flow swirls.

INSTALLATION

These meters can be installed as simply as a short length of pipe and in any convenient reading position, either horizontally, slanted or vertically on an intake or discharge line. The only restriction being is that sufficient head pressure be available, under all flow conditions, to assure a full pipe. Installation directly downstream from a throttling valve, gate valve, or similar fitting which might cause a jet or a strong spiral flow into the meter should be avoided.

SPECIFICATIONS

SPECIFICATIONS

Table 1

	2	3	4	5	6	8	10	12	14
Min Flow	30	35	60	80	100	120	160	200	250
Max Flow	80	200	400	750	900	1200	1600	2200	3000
Weight-150	40	60	80	105	155	220	340	400	425
250	55	75	95	115	170	235	355	455	600
Flange-150	125	125	150	125	150	150	150	150	150
250	250	250	300	250	300	300	300	300	300

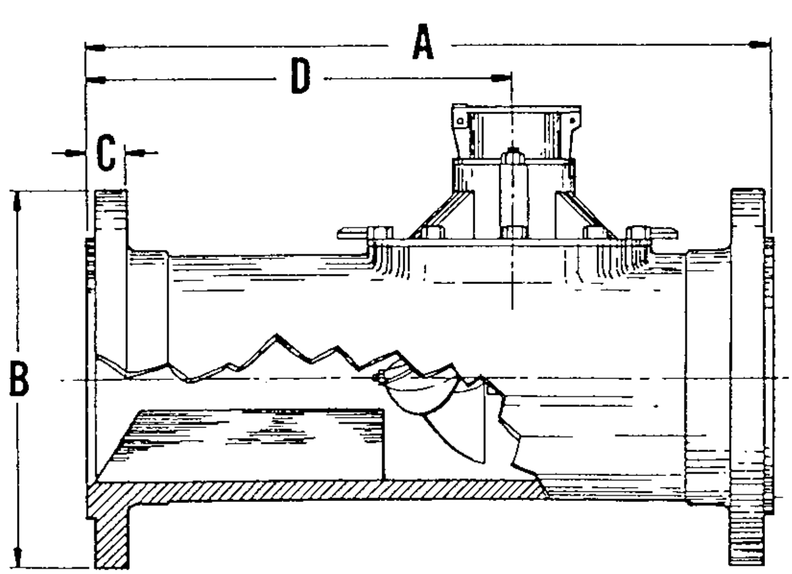
Note: Flow is in U.S. GPM
125 and 250 Flanges - Meet AWWA requirements
150 and 300 Flanges - Meet ANSI requirements

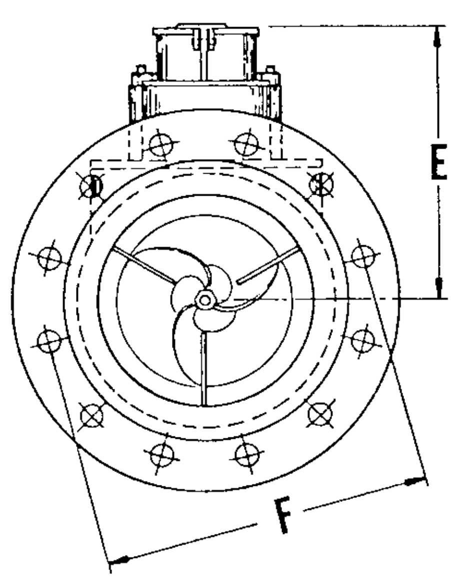
Materials:

Tube ------ 2"– 5" Cast Iron with stainless steel liner
6"–14" Neoprene coated fabricated steel— Standard
6"–12" Cast Iron with stainless steel liner – Optional

Meter Head ---- 2"–14" Cast Iron
Propeller
Standard -------- Molded Polyethelene
Gear Box ---- 2"–14" Brass
Mechanical Parts ------ Stainless Steel
Bearings ---------- Stainless Steel and Delron

DIMENSIONAL DATA

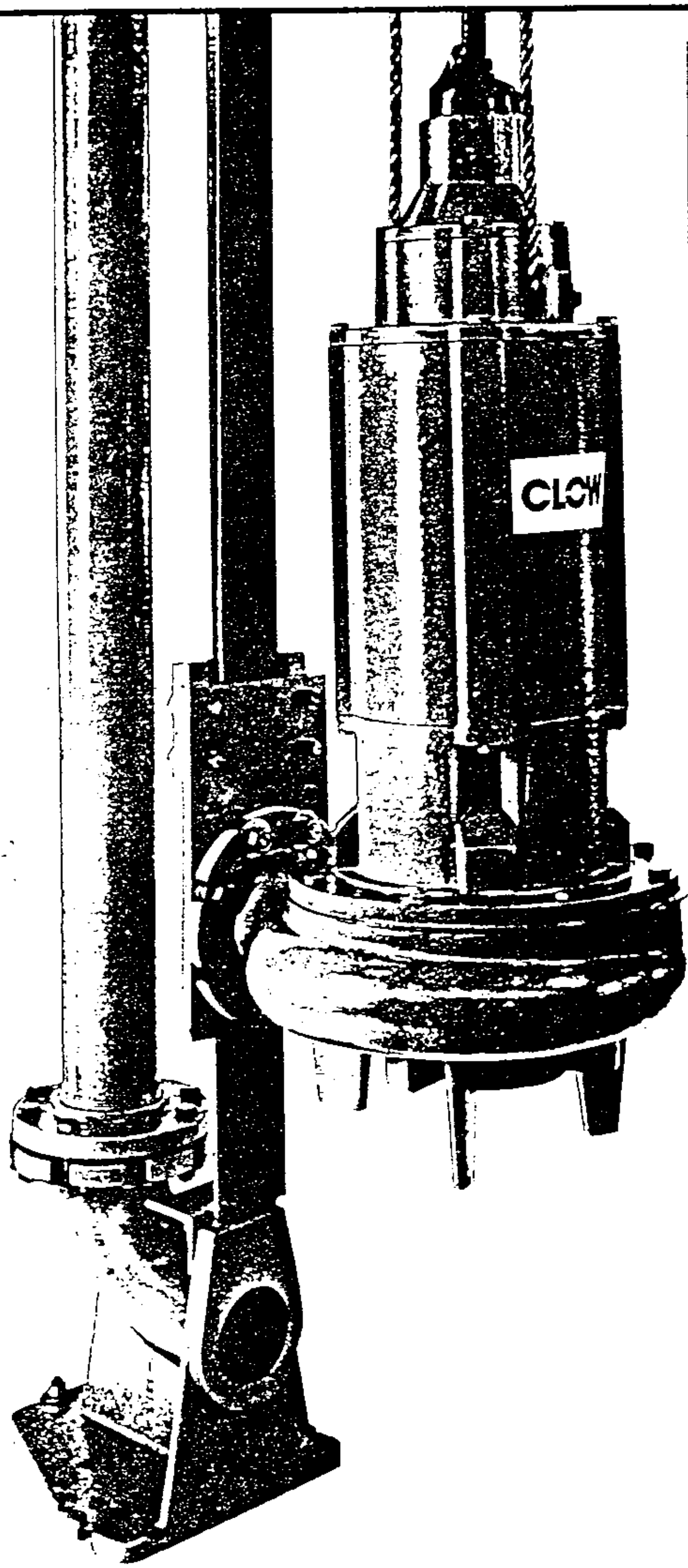




CLOW

YEOMANS PRODUCTS

- Series 9000-for Industrial, Municipal Uses
- Capacities to 7000 GPM; Heads to 240′



The quality-built submersible pump, designed for easy installation... long-life performance... maintenance free operation.

Total reliability and maintenance free operation, even under the toughest wastewater pumping applications, are the outstanding features of the Series 9000 pumps.

These pumps are built to handle sewage and wastewater from commercial and industrial buildings, processing plants, utilities, sewage plants, water treatment facilities, recreation parks and pools.

The Series 9000 provides submersible wastewater pumps in a complete range of capacities — from 50 to 7000 GPM with 3 to 12" discharge. Horsepower range of 3/4 through 150 H.P., and capacity for head selection of 5 to 240 feet.

DEPENDABILITY PLUS

Clow submersible pumps provide round-the-clock reliability, under the toughest conditions. They are guaranteed to operate efficiently and continuously when installed according to factory recommendations.

PLUS — Easy Installation

A simplified design is the key to fast, low-cost installation in a wet pit. All connections may be made quickly and easily.

PLUS — Maintenance Free Operation

Special maintenance free designs, incorporated into these pumps, eliminate routine maintenance and pump down time providing a cost savings.

A quick lift device (optional) permits rapid installation of pump and motor, making it unnecessary to enter the wet pit.

PLUS — Built-In Safety

Both thermal overload protection and a moisture detection system are built-in. Pump will continue to operate without damage even when no water or sewage is present.

Cl

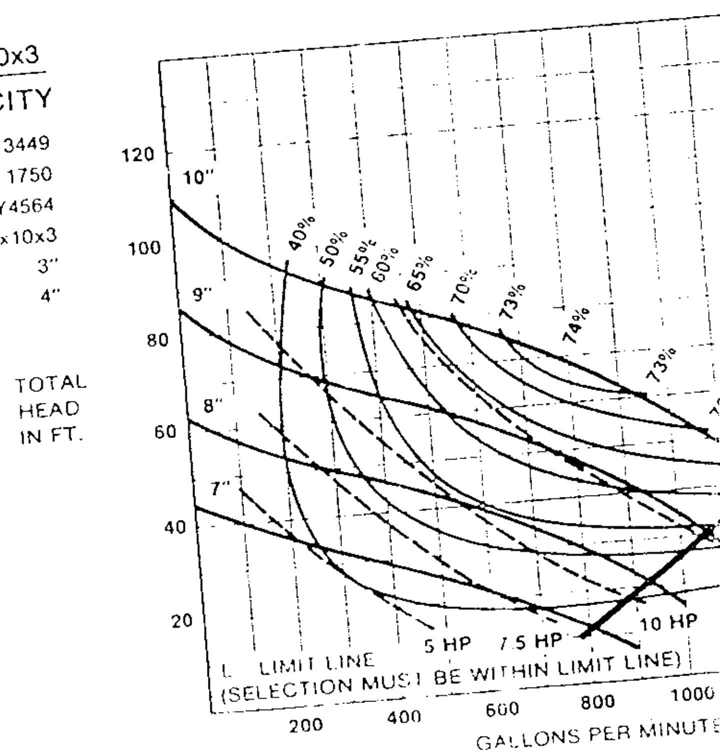
Pump Division
Clow Corporation
Melrose Park, Illinois

SUBMERSIBLE PUMP CURVES
1750 R.P.M.

MODEL: 4x4x10x3

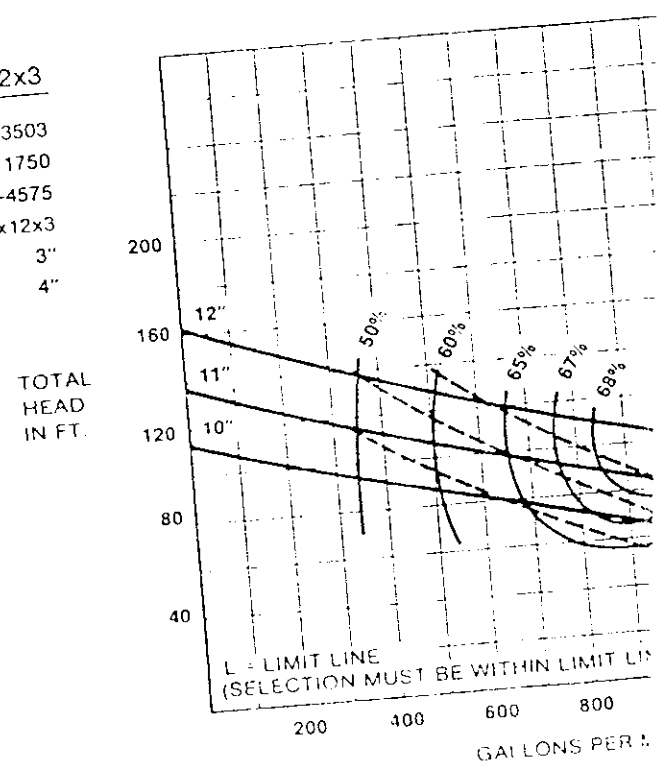
LARGE CAPACITY

CURVE NO.:	3449
SPEED:	1750
IMP. NO.:	Y4564
SIZE:	4x4x10x3
MAX. SPHERE:	3"
DISCH. SIZE:	4"

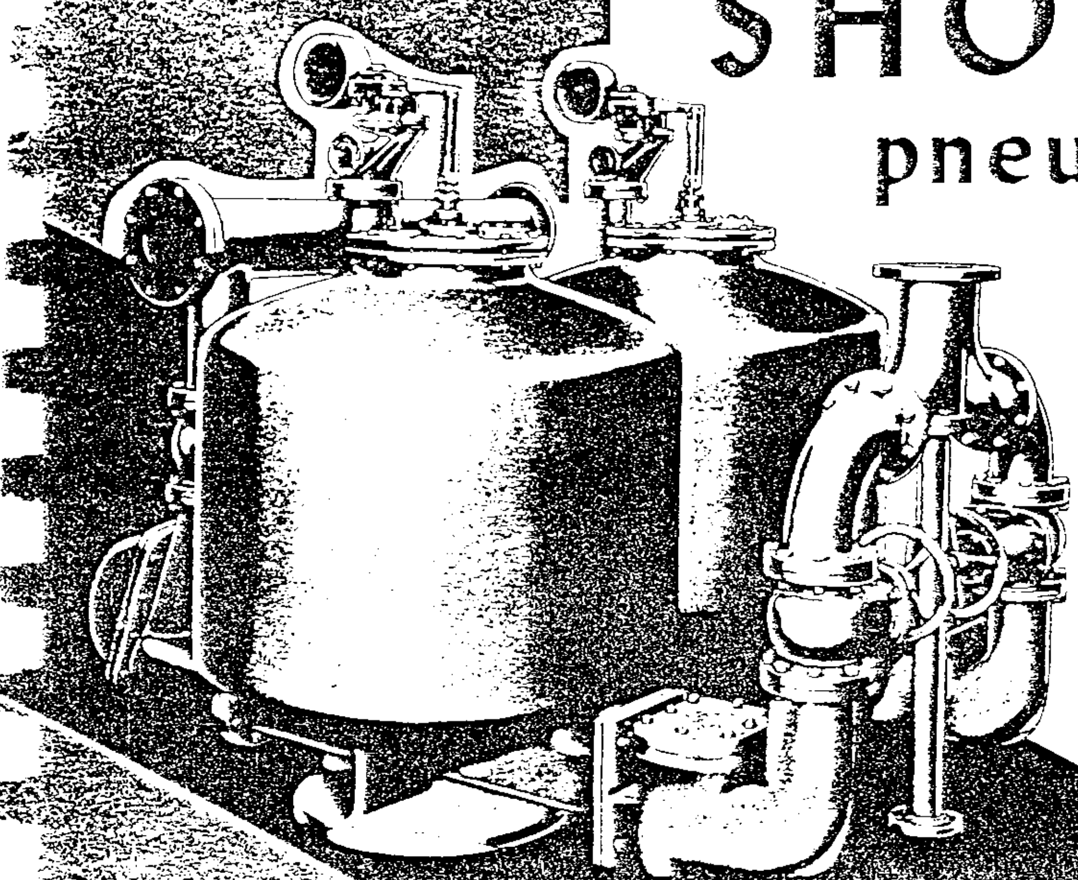


MODEL: 4x4x12x3

CURVE NO.:	3503
SPEED:	1750
IMP. NO.:	Y-4575
SIZE:	4x4x12x3
MAX. SPHERE:	3"
DISCH. SIZE:	4"



SHONE®
pneumatic ejectors



SHONE®

Pneumatic Ejector Mechanically Controlled

Warranted 25 Years

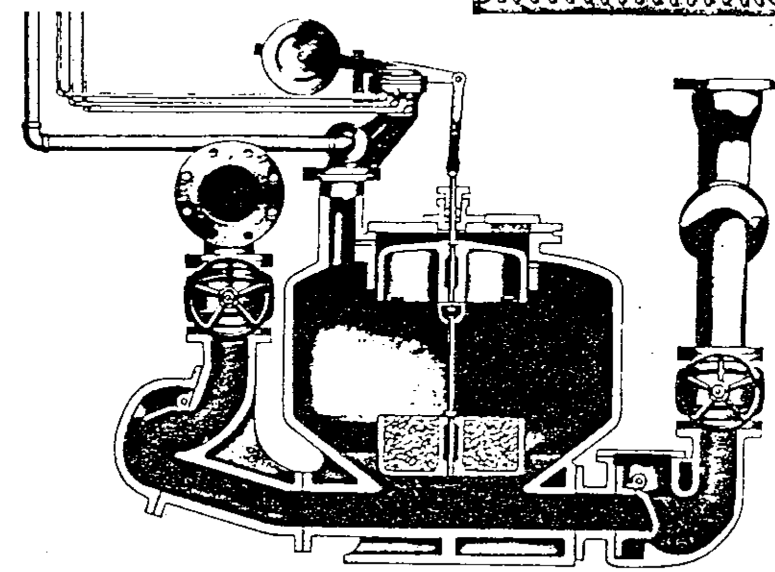
CLOW

Bulletin 4000-C

SHONE gives trouble-free pumping when gallonage is limited but solids are not

SHONE*
Pneumatic Ejector Mechanically Controlled
Warranted
25 Years

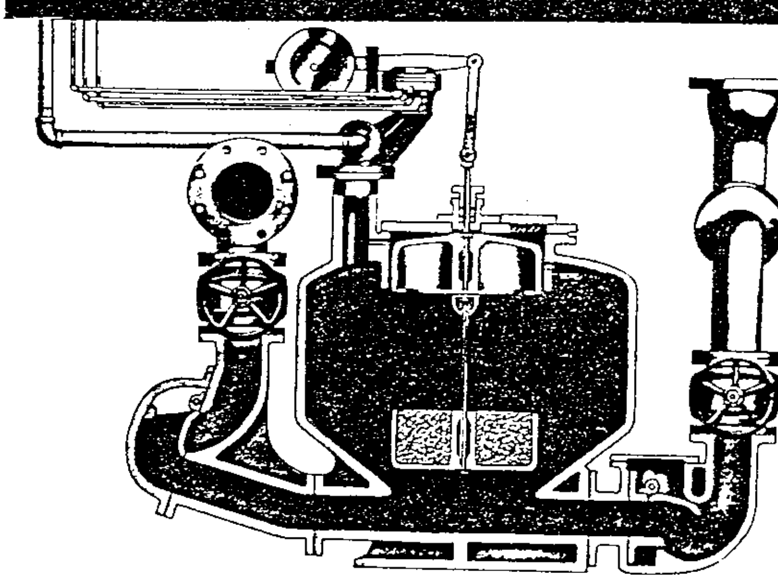
The Shone Ejector is designed for extreme dependability in pumping jobs where gallonage is limited but solids are not – domestic wastes, industrial wastes, heavy slurries. It is clogproof. It has no rotating pump parts, no airtight floats, no high-speed shafts or bearings ... there's nothing to restrict the flow through the receiver.



The Shone is completely sanitary and safe. Being hermetically sealed, it can't expose liquids to the atmosphere. There is no release of noxious or toxic gases, no wastes retention, no sludge accumulation.

Why not investigate the possibilities of the Shone for those special pumping applications in the 30-1000 gpm range.

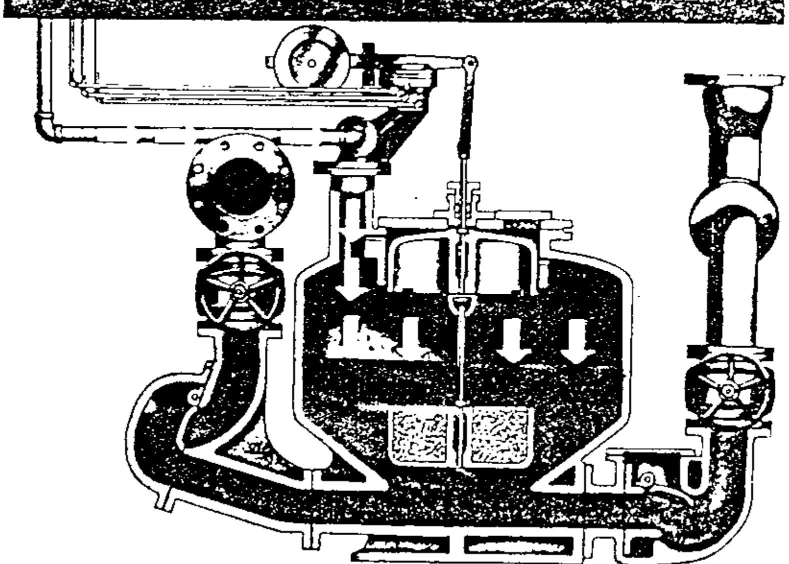
simple, mechanical operation



The operation is accomplished by the rise and fall of two open cast iron bells. Through a pilot valve the motion of the bells actuates a piston valve that admits compressed air to the receiver, pushing liquid out through the discharge line. In the filling position, the receiver is vented to atmosphere and the bells are in the lower position. Weight of the incoming liquid holds the inlet check valve open, while the weight of liquid in the lift line holds the discharge check valve closed.

As the level of the liquid rises above the lower bell, it is buoyed by the weight of the water displaced. Continuing to rise, the liquid partially submerges the upper bell, trapping air beneath it. The upper bell is given sufficient buoyancy to remove still more weight from the rod.

With this final decrease in weight, the counterbalance on the pilot valve lowers and shifts the pilot slide valve, sending air to the other side of the piston valve. This closes the exhaust connection in the piston valve and admits compressed air to the receiver.



The pressure of the air on the surface of the liquid is greater than either that of the inlet line or the discharge lift line. The inlet check valve closes and the discharge check valve opens. Liquid is discharged under pressure into the lift line.

As the receiver empties, first the upper bell loses buoyancy as the liquid level falls below it. When the lower bell becomes exposed, its added weight on the rod overcomes the counterweight. The pilot and piston valves change, shutting off the supply of compressed air and venting the receiver to atmosphere. The check valves automatically assume positions to permit the receiver to refill.

APPENDIX B

PRELIMINARY DESIGN
OF THE
PLANTATION DISPOSAL WELL
CITY OF PLANTATION, FLORIDA

AUGUST 1983

PREPARED FOR:

CAMP, DRESSER, & McKEE, INC.
FORT LAUDERDALE, FLORIDA

PREPARED BY:

GERAGHTY & MILLER, INC.
GROUND-WATER CONSULTANTS
WEST PALM BEACH, FLORIDA

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INTRODUCTION

The City of Plantation is planning to use a deep injection well tapping the boulder zone for the disposal of treated sewage effluent. The City's consulting engineer, Camp, Dresser, & McKee, Inc. (CDM), retained Geraghty & Miller to research the geology of the area and develop preliminary design criteria for an injection well based on the results of the research and information (provided by CDM) on the quantity of effluent to be disposed of. This report contains sections documenting geologic conditions in the area and preliminary design criteria and includes a discussion of the factors related to the City's noise ordinance and well construction activities.

GEOLOGY

Regional

Peninsular Florida is underlain by a thick sequence of carbonate rocks, predominantly limestone with lesser amounts of dolomite. Certain formations contain dense, impermeable beds of evaporites such as gypsum and anhydrite, and at some locations the limestone has been replaced by beds and nodules of chert. This sequence of rocks was deposited in a geologic environment such as exists in the Bahamas today. The total thickness of the sedimentary rocks in the southern portion of the state is well in excess of 10,000 feet.

As far as deep well disposal of the treated waste is concerned, rocks of Tertiary and younger age are of significance. The thickness of this portion of the sedimentary column in Broward County is on the order of 5,000 feet. Throughout southern Florida, rocks of this sequence range from those of Recent and Pleistocene age which are exposed at the surface to those of the Paleocene epoch. A brief description of the various geologic formations is given in the following paragraphs, primarily to acquaint the reader with the nature of each unit. For more detailed information, refer to the publications listed in the bibliography.

The youngest and uppermost formations of Recent and Pleistocene age in the series are comprised of alternating layers of sand, shells, clay, coquina, limestone, and silt. This sequence overlies the Tamiami Formation of Miocene age which has a similar lithologic makeup. These two units are of extreme importance as they comprise the Biscayne aquifer, the area's only source of potable ground water. The thickness of these beds is approximately 200 feet.

The Hawthorn Formation, of Miocene age, underlies the Biscayne aquifer. It is composed primarily of a "tight" green clay ranging from a rather pure, plastic substance to a silty, sandy material. Phosphate minerals are found throughout the formation. Stringers, or layers, of limestone are present at the base of the formation, near the contact with the underlying rocks. Generally, clays of the Hawthorn grade into limestone and the contact with the underlying formation is not sharp. The Hawthorn functions as the confining bed for the underlying Floridan aquifer. The clays of the Hawthorn are, for all practical purposes, impermeable and constitute a barrier to the movement of water contained in the Floridan, which is under artesian pressure (Stringfield, 1966). The Hawthorn Formation in the Plantation area is present to a depth of 900 to 1000 feet.

Limestone of the Ocala group unconformably underlies the Hawthorn Formation. Rocks of the Ocala have been affected by solution activity so that they possess a high degree of secondary porosity and are highly permeable. The Ocala group forms the upper portion of the Floridan aquifer, which is capable of yielding large quantities of brackish water to individual wells. It should be noted that brackish ground water (with less than 10,000 mgl of total dissolved solids and classified as a Class G-II ground water) found in the Floridan is to be protected. The thickness of the Ocala is variable owing to the fact that at some time in the past it was eroded when it was exposed at the surface and it was deposited unconformably on the underlying Avon Park limestone. Available data indicate that it is about 100 feet thick in the general area.

The Avon Park and Lake City Limestones are grouped together because of similar lithologic characteristics indicative of a similar depositional environment. The distinction between the two formations is made on the basis of characteristic micro-fossils present in each. The rocks forming these units consist of limestone and dolomite ranging in nature from soft-porous to hard, dense, crystalline material. The color of the rock is variable, but is usually tan to brown; yellowish- to grey-colored rock also may be present. The total thickness of these units is about 1400 feet in Broward County. Both permeable and tight zones are present in the Avon Park and Lake City, but on the whole they would tend to act as confining beds, particularly in the basal section of the sequence.

The Oldsmar Limestone is the oldest formation of Eocene age. The Oldsmar is composed essentially of dolomite and limestone and may contain minor amounts of evaporites; chert also can be present. Studies show that the Oldsmar is well developed and widely distributed throughout peninsular Florida. In the Plantation area, its top is estimated to occur at a depth of about 2400 feet. The Oldsmar is comprised of a chalky, fine-grained, tight limestone in its upper portion and the basal section is composed of a dense, brown, crystalline dolomite. These two units are of extreme importance; the upper one constitutes a good confining bed, while the lower dolomite contains zones of high transmissivity. These will be discussed in greater detail in a subsequent section of the report.

The oldest and deepest formation of Tertiary age is the Cedar Keys Limestone of Paleocene age. This unit is comprised of alternating layers of dolomite and evaporites (gypsum and anhydrite) with a thickness of about 1000 feet. On the whole, the Cedar Keys is relatively impermeable. The beds of anhydrite and gypsum are quite dense and generally constitute a significant percentage of the formation. Thus, this formation functions quite effectively as a confining bed, preventing the movement of fluids.

In the past few years, the level of knowledge of subsurface conditions in southern Florida has increased dramatically as a result of regional investigations and the drilling and testing of a number of oil and injection wells. These investigations have relied on the analysis of subsurface data such as cores, cuttings, geophysical logs, geologic logs, and driller's logs. A rather large number of these wells have been drilled throughout south Florida and they are reasonably well distributed so that good coverage of that part of the state is available. As a result, a considerable amount of knowledge is available on the nature and distribution of potential injection zones and confining beds. Analysis of this data demonstrates the widespread occurrence of these units.

The most definitive works to date are two reports published by the State of Florida. Chen, 1965, investigated stratigraphic and rock characteristics of the Paleocene and Eocene formations of Florida. This study demonstrated the widespread occurrence of the various formations and mapped their thicknesses and depths of occurrence. Also, much additional knowledge on the environments of deposition was gained.

Puri and Winston, 1974, published a report which is devoted expressly to the geologic nature, occurrence, and distribution of highly transmissive zones and confining beds throughout south Florida. Their study concentrated on the various formations of Eocene age, where the major occurrences of highly transmissive zones have been reported. Puri and Winston identified and mapped major rock types within formations of Eocene age, related them to their ability to transmit water and/or act as confining beds, and developed a three-fold division of the Eocene. They found that the bulk of the various limestone beds can be divided into two principal rock types: grainstone and packstone. The grainstone is fine to medium grained, poorly cemented, fossiliferous, and tan to cream in color. This rock can be permeable, owing to poor cementation and the effects of solution activity. Packstone is predominantly cream in color, but can be tan or white. It contains considerable chalky cementing material which gives it a low permeability. In addition, beds of chalky limestone also are present. This rock has a low effective porosity, which is microscopic in size, and consequently a low permeability. Dolomite (dolostone) is also

present. It is usually brown to tan in color, although it can
It ranges from cryptocrystalline to coarsely crystalline or s
occurs either as thick, massive, discrete beds or as thin zo
limestone beds. Permeability depends on the degree to which
affected by solution.

Generally, the grainstone beds are the locus of zon
transmissivity, while the packstone and chalky limestone
confining units. The dolomite can either act as a confini
highly transmissive.

Based on lithologic characteristics, the Eocene has been s
Puri and Winston) into three units designated as EO1, EO
order of increasing geologic age. Each has fairl
lithologic characteristics, which in turn control the
highly permeable zones and confining units.

The EO1 unit comprises the upper third of the Eocene a
rock of the Ocala group and the upper portion of
limestone. In the Broward County area, EO1 is formed b
and grainstone; dolomite is believed to be absent.
zones are present in grainstone of the Ocala grou
porosity is high because of the effects of solution act

The EO2 unit, formed by the lower part of the Avon
the Lake City Limestone, contains grainstone,
limestone, and dolomite beds. The grainstone can
the chalky limestone and packstone are tight a
prevent the movement of water. In Broward County, d
cavernous and quite permeable, based on data from a
southwestern Palm Beach County near the Browar
various data from the Margate and Fort Lauderdale di

Unit EO3, which is comprised primarily of th
contains dolomite beds, grainstone, packstone,
Dolomite constitutes the majority of the lowe
This unit is of special importance as it contain
zone, which is found in the dolomite of the lowe
The boulder zone possesses an extremely high t
the presence of a vast, interconnected netwo
solution features. Beds immediately overlying
to be dense and relatively impermeable due
limestone and packstone also overlying the boul

The Puri-Winston study revealed that more
transmissivity exists in south Florida. In
to occur in beds of Cretaceous, Paleocene,
deep well disposal is concerned in south
present in the basal part of Unit EO3 (Ol
important. Hereafter, that zone will be ref
to distinguish it from others that may
Florida.

The Oldsmar zone has been mapped as occurring everywhere south of a line crossing the state from northeastern St. Lucie County on the east coast to the Charlotte-Lee County line on the west coast (Puri-Winston, 1974, page 46). In southeastern Florida it has been penetrated at West Palm Beach, at Belle Glade, in Margate, at several sites in Dade County, and at Port Everglades in Fort Lauderdale. The depth to the top of the zone ranges from approximately 2800 to 3300 feet, depending on location. It is estimated to occur at approximately 3000 feet at Plantation.

At the West Palm Beach regional wastewater treatment plant, the Oldsmar zone was found to be about 600 feet thick. A similar thickness was penetrated by one of the recently completed injection wells at the George Lohmeyer Wastewater Treatment Plant located at Port Everglades. At Margate it was found at a depth of 3070 feet and is at least 230 feet thick.

Beds overlying the Oldsmar zone function as confining units. These consist of dolomites, packstones, chalky limestone, and some grainstone. At West Palm Beach, for example, approximately 600 feet of such rocks are found to overlie the zone. At Margate, both the Avon Park and Lake City Limestones contain considerable thickness of materials which have been identified as confining beds, and similar conditions were found at the Fort Lauderdale site.

Thus, there is no doubt that the Oldsmar zone and confining beds are found throughout southeastern Florida and, as will be noted in the next section, favorable conditions for deep well injection exist in the study area.

Local Conditions

Local conditions are defined as those existing within the study area. Fortunately, excellent information exists from the drilling and testing of the Margate and Fort Lauderdale wells. Because the various units have been shown to be widespread throughout southeastern Florida, this information can be considered as sufficiently representative of the study area and can be used to serve as the basis for determining injection well design.

Throughout the study area, favorable geologic conditions exist both for constructing and operating a deep well disposal system and for meeting regulatory agency criteria for such a system.

Based on interpretation of the Margate disposal well data, the Oldsmar zone is present at a depth of ± 3000 feet. Test results and operational data from that disposal well system clearly demonstrate that the zone is highly transmissive and capable of accepting large quantities of treated effluent at high injection rates. For example, in August of 1976, after several years of operation, a maximum injection rate of 6.9 mgd (million gallons daily) or 4789 gpm (gallons per minute) was recorded with a reported maximum injection pressure (pounds per square inch) at the well head. Comparison of initial operating data indicates that no change has occurred start of operations.

The presence of a suitable confining bed is shown by evalua core and geophysical log data. Chalky to dense, ye dolomitic limestone occurs from 2500 feet or so to th Oldsmar zone.

From 2100 to 2270 feet, a dark brown and gray, cavernou present. This zone is highly transmissive. Furthermore salty water. The geophysical logs of the well show that zone between brackish and salty ground water occurs between 1900 and 2000 feet. This agrees quite closely found at the West Palm Beach disposal well site. Thus, zone is the first saline water-bearing zone and can monitoring purposes.

Most of the dense to chalky, dolomitic limestone overly cavernous zone also functions as a confining bed. Th from 1400 to 2100 feet.

White to cream, soft limestone of the Ocala group is and is water-bearing between 950 feet and 1150 constitutes the upper portion of the Floridan aquifer.

Between roughly 300 feet and 950 feet, silt and clay materials comprise the Hawthorn Formation, which se bed for the Floridan aquifer and prevents the overl from being contaminated by the brackish water occ pressure in the Floridan.

Thus, to briefly summarize, favorable conditions certainly are present in the study area. S monitoring zones exist; both contain salty wa 10,000 mgl total dissolved solids). At least th are present.

In view of these facts, it is Geraghty & Miller well systems can be constructed throughout operated providing that properly designed installed. Furthermore, experience has shown proper construction and monitoring techniq proven, such wells can be installed without c Biscayne aquifer.

INJECTION WELL DESIGN

Injection Casing Design

The range in sizes of the casings used in constructing an injection well is dictated by the size of the inner or injection casing, which in turn is controlled by the quantity of effluent to be injected. In general, a velocity of 10 fps (feet per second) is used as an approximate guide for determining the inner casing diameter. This is based on a guideline for sewers which handle gritty, abrasive material, and presumably eliminates or minimizes the potential for erosion. This is probably a very conservative figure—injection wells handle secondary treated effluent that does not contain gritty material—and could be exceeded for short periods of time without adverse effects.

The Plantation treatment plant will generate effluent at an average daily rate of 8.3 mgd (5,764 gpm) for the design period 1986-1991. A maximum four-hour rate of 16.6 mgd (11,527 gpm) is anticipated, with a peak (one-hour) maximum of 20.8 mgd (14,444 gpm), according to data furnished by CDM. These rates can be handled by a 24-inch-diameter well (actually 23 inches in diameter because a 0.500-inch wall thickness pipe is used for the inner casing).

Calculated values of velocity and friction loss (Hazen & Williams C-100) are listed in the following table.

	Rate (mgd)	Velocity (fps)	Friction/ (ft/1000')	Friction/ (ft/3000')
Average daily	8.3	4.44	4.5	13.5
Maximum 4-hour	16.6	8.91	16	48
Maximum	20.8	11.2	23	69

In addition to the friction loss figures shown above, the total pumping head or pressure observed at the well head should include an additional 76 feet of head to allow for the density differential between native salt water and treated effluent and the bottom hole driving pressure. The calculated values for total head are listed below for each of the rates.

	Rate (mgd)	Head (ft)	Head (psi)
Average daily	8.3	89.5	38.6
Maximum 4-hour	16.6	124.0	53.4
Maximum	20.8	145.0	62.5

Construction Details

To take full advantage of the boulder zone's capabil injected fluid, disposal wells should be drilled to a feet. Protection from contamination will be achieved b the disposal well with four separate strings of casi approximate depths stated. It should be noted that the a which a casing is set will be determined by field evaluation of the data from each individual well. drilling program should be set up to provide for pilot-h allow for the selection of casing points; the specifica add or deduct provisions for the casings to take differences in field conditions that dictate changes in addition, the specifications should stipulate that electric resistance welded (ERW) pipe (ASI 5L Grade B inner casing, and that spiral wound pipe could be u casings.

A conductor casing (54-inch diameter, 0.375-inch wall set to a depth of about 100 feet. This is necessar purposes and will prevent the upper portion of collapsing, which would endanger the drilling rig Hawthorn Formation is being drilled. There is no r the entire Biscayne aquifer as more than adequa contamination will be achieved by the setting and ce three strings of casing.

Surface casing (42-inch diameter, 0.375-inch wall t to a depth of approximately 1000 feet. This purposes. It will seal off the Hawthorn Formatio of soft, plastic clay that can squeeze into a seal off the entire Biscayne aquifer.

An intermediate string of casing 34 inches thickness of 0.375 inch will be set to a dept feet, or below the base of the transmissiv interval between 2100 and 2270 feet. This cas off the Floridan aquifer and provide protec contamination of the Floridan will occur in second injection well is needed. Setting transmissive zone will make it possible to ass will be completely cemented. Normally the in set above this zone at a depth of approximatel would require that the cement for the ir emplaced opposite the transmissive interva necessary protection. Judging from this zon on the Margate well), this may not be p unconventional, sophisticated, and very c intermediate casing below this zone would s costs both for cementing and eliminate the r Details of the deep monitoring capability ar

Finally, a 24-inch-diameter inner casing with a wall thickness of 0.500 inch should be set from land surface to a depth of approximately 3000 feet. The inner casing will penetrate and will be sealed well into the confining bed overlying the boulder zone.

Casing data, bit size, and other pertinent information are summarized below.

Casing	Diameter (inches)	Wall Thickness (inches)	Setting (feet below grade)	Bit Size (inches)
Conductor	54	0.375	100	62
Surface	1,000	0.375	1,000	52
Intermediate	2,300	0.375	2,300	40.5
Inner	3,000	0.500	3,000	32
Open Hole	—	—	—	22

Each string of casing will be completely cemented in place using Class H cement and lost circulation additives to insure that a good bond is obtained between casing, cement sheath, and the walls of the bore hole. Bond logs may be performed to insure that proper cement seals are obtained; provisions will be included in the specifications requiring the contractor to take remedial measures to repair any defective cement job at no extra cost to the owner.

The injection well will be equipped with a monitor tube set in the annular space outside of the intermediate casing. It will tap the transmissive, saline water-bearing zone occurring between 2100 and 2270 feet. The monitor zone will be gravel packed. To provide additional protection from corrosion, the inner casing in this interval will be coated with a field-applied epoxy coating. This type of construction will serve two purposes. First, monitoring of a saline water-bearing zone will be accomplished; second, it will eliminate the need for cementing of this zone, which would be difficult—if not impossible—and certainly very expenssive to successfully cement because it is so cavernous and permeable. This type of procedure has been successfully used in deep test and disposal wells. Furthermore, it will not jeoparidze the well's integrity and it will reduce the costs for monitoring.

Well Construction Method

Disposal wells must be drilled by methods which will provide a maximum amount of environmental protection at a reasonable cost. Contamination from spills of fluids during construction can be avoided by drilling disposal wells from a concrete pad designed to contain any fluid spills. Shallow observation wells will be located at critical points on the perimeter of each pad, and monitored on a regular basis during the drilling operations. Each pad will be constructed with water-tight sumps to temporarily store fluids. Steel-lined tanks should be used to contain drilling fluids. Cuttings, drilling mud, and other fluids and potential contaminants will be properly disposed of only in approved areas.

Disposal wells can be drilled by the conventional mud rotary method through the Hawthorn Formation. For the deeper formations, only some approved form of reverse circulation drilling method should be employed. No surface discharges of drilling and formation fluids should be permitted. These methods make it possible to collect extremely reliable formation samples and eliminate the possibility of costly and time consuming lost circulation problems. Using the reverse air circulation method for drilling through the Oldsmar zone results in the completion of the most efficient injection well possible, because the bore hole is cleaned by the drilling process. Furthermore, it is the most effective method for drilling through this zone. Many parts of the Oldsmar contain fine- to coarse-grained rubble which tends to cause the hole to collapse. This material should be removed to provide an efficient installation; the reverse method is the only practical way to accomplish this.

One of the potential principal problems in constructing deep, large-diameter disposal wells is the inability to set the casing deep enough, particularly the inner string. A chief cause of this is an insufficiently straight hole—one that deviates from the vertical or contains dog-legs (abrupt changes in hole direction). This can be avoided with proper specifications, by proceeding cautiously with the drilling operation, and by running directional surveys at specified depth intervals during the drilling process to insure that the hole is being drilled to specifications.

Data Collection and Testing

Successful construction of a disposal well is dependant on the reliability of the information collected during the drilling. While confining beds and injection and monitoring zones are regionally extensive, existing data are not adequate to arrive at precise figures for depths and casing settings at the Plantation site. Therefore, the specifications for well construction must be flexible to allow for the slight variations that may be expected to occur.

To take these into account, it is necessary to collect rather detailed information by a variety of methods so that a reliable picture of subsurface conditions can be developed, making it possible to complete the well to provide a reliable installation. In addition, the regulatory agencies (Florida Department of Environmental Regulation assisted by the Technical Advisory Committee) will pass judgment on the adequacy of the design and the data collection program when evaluating the permit application for the construction of a test injection well and will require that subsurface data be collected and submitted as part of the report supporting the application for an operating permit following the installation of the test injection well.

Aside from selecting the proper positions at which to set casings, the principal reason for collecting the various data is to determine the nature and location of the confining bed(s) and the position of the injection zone, and to enable selection of a suitable monitoring zone. To accomplish this, the usual formation samples will be collected at precise depth intervals (10 feet) and at formation changes. Cores of confining beds will be taken in the confining bed interval between 2300 and 3000 feet, either by conventional or sidewall methods, and will be analyzed to determine porosity and permeability. A suite of geologic logs such as induction, lateral, neutron porosity, density, acoustic, caliper, and gamma ray will be taken. This information will be utilized to determine the characteristics of the confining bed and monitor zones, to delineate the injection horizon, to select the proper setting for casings and monitor pipes, and to determine final details of the cementing program.

The last step in the construction of the test injection well is the performance of a test. Where conditions permit, the usual practice is to run a high-rate pump-out test. In some cases, an optional lower rate injection test is performed also. Generally, a lower rate injection test is performed because of the lack of an available water source at a resonable cost. Also, effluent has been used for injection testing.

A pump-out test cannot be performed at the Plantation site because there is no way of disposing of saline water. Thus, an injection test will have to be conducted. Either the nearby canal adjacent to the treatment plant could be used as a source for a 10,000-gpm test or effluent could be used. The selection of alternative would depend on (1) the quantity available from the canal, (2) the willingness of the DER to allow the use of effluent, and (3) the cost. Each of these options should be evaluated as part of the detailed planning for the program.

OTHER CONSIDERATIONS

The proposed well site is located on the northwest corner of the treatment plant property about 600 feet or so from a hospital and about 750 feet from residential neighborhoods to the east and west. An industrial area is located to the north. The well location may pose a problem because of noise created by the drilling, construction, and testing operations, which are conducted 24 hours per day, 7 days per week. The most objectionable noises are those made during the tripping out of the drill string—caused by the laying down of the drill pipe—and the sound of the rig brakes when the casing strings are being lowered during setting operations. Additional sources are the rig and pump motors, which are continuously running, and noise created during cementing operations.

According to the City of Plantation noise ordinance (Section 17-15), it is unlawful for "noisy" businesses to operate "except between the hours of 8:00 a.m. and 6:00 p.m. on weekdays only from December first to April first of each year, and except between the hours of 7:00 a.m. and 6:00 p.m. during the remaining part of the year on weekdays only" when the noise source is located "so close to inhabited dwellings, apartments, or hotels so that the noise emitted from the operation of such business or enterprise shall disturb or is detrimental to the health, peace and quiet of the occupants." There is a provision in the ordinance allowing for a variance to be granted upon application to the City Council and review and approval pending the results of the council's investigation.

Drilling operations are carried out on a continuous basis because (1) it reduces costs, and (2) some procedures cannot be stopped because to do so would jeopardize the integrity of the operation. By limiting the time for drilling, constructing, and testing to the hours stated in the ordinance, the cost of a well would be increased 40 to 50 percent above normal. In addition to taking longer to construct the well because the effort is spread over a longer period of time, inefficiencies are created requiring considerable duplication of effort in starting and stopping, adding further to the cost.

Certain operations, such as drilling through the Hawthorn Formation, dredging, coring materials, testing, setting casing, and cementing, must be done on a continuous basis; there is considerable risk of a failure or at least serious construction problems otherwise.

For these reasons, it is strongly recommended that the Plantation well be constructed using the normal 24-hours-per-day, 7-days-per-week procedure. To accomplish this, the following should be done. First, strict noise control procedures should be written into the specifications. This would include the muffling of all engines (required by ordinance), the construction and maintenance of sand barriers (stacked bales of hay were used successfully at Fort Lauderdale), and the covering with wood of all metal surfaces (catwalk, V door, and rig floor) that would come in contact with drill pipe, tools, casing, etc. Second, application for a variance should be made to the City Council, backed up by evidence from a sound engineer attesting to the adequacy of the sound-reducing procedures, with support from an injection well expert justifying the need for round-the-clock operations.

Finally, the City should develop and implement a public relations program to educate the neighbors and potential objectors to the need for continuous operations. An added benefit to this would be that the public would become educated about deep well disposal. While the practice is finding more acceptance as the list of successful applications grows, there is still a large segment of the public who could object to the practice primarily because they are unacquainted with it.

Mention was made previously of the use of an annular monitoring tube to tap the transmissive zone at approximately 2100 feet (this zone corresponds to the so-called 2000-foot zone encountered at the Fort Lauderdale site) to reduce costs and preserve well integrity. Standard practice (by the DER) calls for the monitoring of two or more zones. At Plantation this could be accomplished by installing a shallow (as much as ± 1600 feet deep) monitor well to tap zones containing water with less than 10,000 mgl. This could be done as a separate contract (to encourage more bidders) either at the time the test injection well is installed or at some time in the future when monitoring results indicate that another monitor well is needed. If it is drilled during the test injection well program, it should be started after the pilot hole to the boulder zone is completed and logged to provide information on its actual depth. If the latter course of action is chosen, the DER will have to be pursuaded to accept this alternative.

As a final note, the injection well system will be handling an average of 8.3 mgd. Current DER (West Palm Beach office) policy requires that a second or standby well be installed for systems designed to treat 7 mgd or more. This is not official and nothing exists in writing to the best of Geraghty & Miller's knowledge. The need for a second well doubles the initial capital cost and, in many instances, creates a significant financial burden. Thus, a priority item in the program should be the determination of the need for a standby well. To eliminate this need will require convincing the DER and the Broward County Environmental Quality Control Board (which requires a variance) that (1) an acceptable standby emergency surface discharge exists, and (2) the well's design and the nature of the boulder zone assure that a single well will have a long, trouble-free life. Current designs and a rapidly growing body of operating data attest to the latter.

Respectfully submitted,
GERAGHTY & MILLER, INC.

Vincent P. Amy

Vincent P. Amy
Principal

August 10, 1983

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APPENDIX C

Position Paper

Back-up Requirements for Injection Well Systems

It has been the State's position that all injection well facilities which function as the primary disposal method of municipal and industrial waste have back-up facilities. This supplemental system is necessary in the event of a temporary well shutdown. The type of non-operative period we are concerned with here is on the order of days or weeks as opposed to hours. Most shut-downs that only require hours - geophysical logging or T.V. surveying - do not involve a retention of flows beyond treatment plant capabilities for the short term.

The back-up system is not required to be another injection well unless there is no other acceptable option. The question of whether or not surface waters may be used as that back-up system must be resolved by the Bureau of Water Quality and the District Permitting Staff on a case-by-case basis. Factors to be considered in the evaluation of a back-up system are:

The amount of flow to be discharged.

The period allowable for discharge.

The frequency with which discharge is expected.

The operating record of other wells in the vicinity.

Characteristics of the proposed alternative system.

Reliability of the treatment facility.

The nature or cause of the primary disposal method's shutdown.

Characteristics of the effluent being discharged.

Generally, the back-up system need not acquire a separate operating permit, however the primary disposal method's permit should adequately address the back-up disposal system as well as other conditions of the permit.

In the case of Margate, a back-up system would be a contingency of injection into a new well. Since there are injection wells in the area, including Margate's existing well, which have operated successfully for up to ten years, it could be presumed that a well failure more often than once every ten years would be unlikely. In this event a temporary discharge, 90 to 120 days of 5 to 15 mgd, once every ten years would have little adverse environmental impact. Assuming this to be acceptable there is a canal . . . in the vicinity of Margate's wastewater treatment facility that could provide the required back-up capacity.