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LAKE PONTCHARTRAIN STORMWATER DISCHARGE, LOUISIANA

JEFFERSON PARISH DEMONSTRATION PROJECT

SUMMARY OF EVALUATION PHASE

Prepared By:

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In Conjunction With:

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Metairie, Louisiana 70001

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SECTION I
EXECUTIVE SUMMARY

Section 307 of the Water Resource Development Act of 1992 authorized the Secretary of the Army to design and construct projects in Orleans and Jefferson Parishes, Louisiana to improve water quality in stormwater discharges into Lake Pontchartrain. Funding for the demonstration project would be cost shared at 75 percent Federal and 25 percent Non-Federal. Operation and maintenance costs would be 100 percent Non-Federal.

Presently, all stormwater runoff on the Eastbank of Jefferson Parish is discharged into Lake Pontchartrain. Jefferson Parish began working with the New Orleans District Corps of Engineers in early 1993 on plans which would reduce the effects of stormwater runoff on Lake Pontchartrain. In March, 1994 the Corps completed a technical report which evaluated various alternatives to improve stormwater quality. This document investigated solutions such as in-line detention, disinfection, flushing, diversion and sediment capture. The Corps evaluation, in conjunction with input from the Parish, recommended that the most acceptable alternative for improving the quality of stormwater discharges was to capture a portion of the stormwater runoff within the existing drainage canal network for treatment and discharge into the Mississippi River.

Due to limited funding under the 1992 Act a full-scale Parish wide program was not feasible at this time but a demonstration project was conceived which would attempt to prove that treatment of first flush runoff at the Eastbank Wastewater Treatment Plant (WWTP) would be beneficial to the lake water quality. The recommended demonstration plan included construction of a 33,000 gallon per minute (gpm) combined stormwater/wastewater lift station at the intersection of the Suburban Canal and Canal No. 4 along West Napoleon Avenue; installation of a 48 inch sanitary force main for conveyance to the WWTP; and the required modifications at the plant. The total estimated cost for this demonstration plan was estimated to be $22,444,000 including engineering, water quality monitoring, administration, relocations, right-of-way acquisition, construction and contingencies.

This report evaluates in more detail the selected plan identified in the Corps of Engineers March, 1994 technical report. In an effort to expand upon the selected plan, this report defines the operational requirements of the proposed plan and reviews the lift station requirements. In addition, the study compares the benefits of three types of pumps for use at the lift station, and evaluates three force main route alternatives, various force main sizes and available force main materials. The report also considers the capture of stormwater at a single source and multiple sources; defines requirements for revision to the WWTP, refines the cost estimates for the various elements involved; and anticipates the effectiveness of the plan.

As a result of the additional evaluation of alternatives and further investigation of operational requirements the following is hereby recommended:

A. Construction of a 20,000 gpm stormwater lift station and 60-inch diameter force main.

B. Construction of a drainage pump station in an isolated basin to demonstrate effectiveness of multiple site stormwater capture.

C. Installation of a pollution monitoring station in a small subbasin to quantify first flush contaminant levels.

D. Expand the existing treatment plant facilities by adding aeration basins, sludge holding tanks, filter presses and additional dechlorination systems to handle the additional volume and varied pollutant loading from stormwater.

The estimated cost for implementation of all recommendations is $22,776,000 which is 1.5% greater than the estimated cost in the March, 1994 report. These recommendations are discussed in further detail in Section 7 of the Report.

SECTION II
INTRODUCTION

A. BACKGROUND DESCRIPTION OF PROJECT

Water quality along the southshore of Lake Pontchartrain has fecal coliform levels which are considered unsafe for human contact. The Jefferson Parish Sewerage Department has within the last ten years significantly reduced the levels of fecals discharged into the lake. This reduction has been accomplished through an intensive program which has upgraded all facets of Eastbank Sewage transmission and treatment systems and has produced the following results:

1) Rerouted the treated sewage discharge from Lake Pontchartrain to the Mississippi River.

2) Built new lift stations and force mains to replace existing dilapidated and undersized facilities, thereby reducing the incidence of sewer overflows.

3) Repaired gravity sewer system leaks under an on-going point repair program.

4) Eliminated direct bypasses into the stormwater system.

Isolated problems, however, still exist and these can be considered one source of the lake pollutants. Another source of contamination has been identified to be the "first flush" stormwater runoff which carries urban pollution such as fertilizer, gasoline, hydrocarbons and heavy metals into the stormwater drainage canals. In an attempt to reduce the levels of these pollutants in Lake Pontchartrain, this demonstration program was conceived to attempt to capture as much of the first flush runoff as possible and transport it to the Eastbank Wastewater Treatment Plant (WWTP) for treatment and discharge into the Mississippi River. The WWTP has a demonstrated wet weather capacity well in excess of the dry weather capacity. It is anticipated that with some treatment plant upgrades, significant quantities of stormwater could be treated in dry weather and, to a lesser extent, during wet weather. Additionally, this program would improve the pumping capacities of various existing sewage lift stations by redirecting current flow patterns which could virtually eliminate all remaining wet weather sewer overflows.

B. REVIEW OF CONCEPTUAL PLAN

The plan identified in the conceptual phase proposed to construct a 33,000 gallon per minute (gpm) lift station at the intersection of the Suburban Canal and Canal No. 4 along West Napoleon Avenue. This site was selected because of its' location near the center of the Parish drainage network; its' proximity to the existing large regional sewer force main; and the relatively close proximity to the required outfall at the WWTP. It was anticipated that the lift station would be equipped with both a suction line from the adjacent drainage canals and an automatically operated valve arrangement for direct discharge of the existing regional force main, which parallels the West Napoleon Canal, into the station. The lift station would pump stormwater during dry weather periods and would switch to wastewater during wet weather events. A new 48 inch force main would be constructed to transport the fluids from the new station to the WWTP. Modifications would be made at the WWTP to handle the additional hydraulic and pollutant loading associated with the treatment of stormwater. The conceptual plan has been modified during the course of this evaluation phase as will be explained herein. The estimated cost of the conceptual plan was $22,444,000.00.

C. FORMAT OF REPORT

The report is broken into two major components:

1. The capture and transmission of stormwater via a new sewer lift station and force main.
2. The modifications required at the WWTP to treat the influent stormwater.

The components associated with the capture and transmission of the stormwater will be discussed in detail in Sections III and IV. The WWTP modifications will be elaborated upon in Section V of this report.

SECTION III
ALTERNATIVES CONSIDERED

A. GENERAL

A major portion of this evaluation phase provided for considering various alternatives to the conceptual plan described in Section II. The alternatives considered included removal of stormwater at multiple sites as opposed to the single source discussed in the conceptual plan; operating conditions of the proposed lift station including various methods of extracting stormwater; design details of the station and force main including type of station to be constructed and force main material to be utilized; various force main routing schemes; and various methods of treating the stormwater at the WWTP and the modifications required to achieve the required treatment levels. These are discussed in detail herein.

B. SINGLE SOURCE AND MULTIPLE SOURCE STORMWATER REMOVAL

B.1 SINGLE SOURCE SITE

The East Jefferson stormwater drainage system consists of a network of subsurface culverts draining into an open canal system which transports flows to large pumping stations located along the south shore of Lake Pontchartrain. Ideally, to capture as much of the first flush stormwater as possible, it would be beneficial to remove the stormwater at the point of maximum flow, which would be near the drainage pump stations. This location however, provides the greatest distance to the point of treatment (the WWTP), and to implement a collection system at this location would result in a construction cost far in excess of the demonstration project budget. Therefore, a single source site was selected which could effectively extract a large volume of stormwater while also helping to alleviate the majority of the remaining sewage overflow problems.

The selected site is located at the intersection of the Suburban Canal and Canal No. 4. It was selected not only because of its' central location to the interconnected drainage system, but also because of its' close proximity to the WWTP and the regional force main which is the major sewage transmission line that services the northern portion of Metairie. This transmission line is a 5.5 mile force main which originates as a 42 inch diameter line at the former Helios WWTP and ultimately transitions to a 72 inch diameter line discharging into the WWTP. Seven lift stations are manifolded into this line, with a combined capacity of 71,700 gallons per minute. In wet weather events, this line becomes surcharged which prohibits some of the lift stations from achieving their design capacities. As a result, some sewage overflows occur in extreme wet weather events. Figure IV.1 illustrates the layout of the regional force main and corresponding lift stations.

In an effort to relieve this problem, a new large diameter force main with discharge directly to the WWTP was evaluated. This force main will tie into the existing regional force main but be valved to provide a direct route to the treatment plant for three of the seven stations which are presently manifolded together. This new route will significantly increase the design capacities of all seven stations, and will virtually eliminate sewer overflows. Three alternative sizes for this force main were considered as discussed in Subsection C.2 below. In the event a sixty inch diameter force main is selected it will also allow for some expansion of the Stormwater Quality program in the future should it be so warranted. This concept is explained in detail in Section IV B.

The drawback to the single source station is the limited amount of stormwater which can be extracted from the canal system at this location. A plan that combines storage of the first flush stormwater along with backflushing lake water at the drainage pump stations is proposed to maximize the removal of first flush stormwater. It is estimated that approximately 20 million gallons of runoff can be stored in the Suburban Canal basin during "minor" rainfall events before drainage pumping would be required. This stormwater could then be extracted at the lift station over a 24 hour period, temporarily lowering the canal levels in the general area approximately two feet. In an effort to cleanse the canals and to maintain normal canal levels, lake water would be backflushed into the system to allow the canals to rise to the typical dry weather levels and the cycle would be repeated. This concept is explained in detail in Section IV A.

B.2 MULTIPLE SITES

Expansion of the demonstration project on a Parish-wide basis would include extraction of stormwater at multiple sites with conveyance to the WWTP through a series of new or existing sewage force mains. To demonstrate the effectiveness of such a plan, an existing lift station site has been selected which is located within an isolated drainage system. The station is located on the south bank of the West Esplanade Canal at West William David Parkway, as indicated on Figure IV.1.

This portion of the West Esplanade Canal collects stormwater from the Bucktown/Papworth areas and is not subjected to the interconnected canal network as is the main lift station site on the Suburban Canal. Therefore, the effects of this stormwater treatment project could be more accurately quantified at this location. The existing sewer lift station is a submersible type station with three pumps and a design wet weather capacity of 3,120 gallons per minute. Considering that approximately one-third of this capacity typically represents dry weather flow, approximately 2000 gpm could be available at this station during dry weather periods to collect and transmit stormwater from the canals. A description of the operation of a multiple source station is provided in Section IV D.

Another consideration for a multiple source scenario is the direct capture of runoff from the subsurface outfall pipes, with conveyance (either by pumping or gravity flow) directly to an adjacent lift station. This type of capture can also be demonstrated at the West Esplanade/West William David site as a 30 inch arch equivalent outfall pipe from an isolated subsurface collection system is located nearby. The effectiveness of this type of capture could be quantified in this demonstration phase by one of two methods: 1) as a water-quality monitoring only station, or 2) as a pump station with force main tied to the existing lift station and monitored at the outfall collection site. A monitoring-only station would be an extremely inexpensive method of accurately determining the type of stormwater discharges which occur at an individual site. A definitive first flush pollutant level could be determined and it could be assumed that this pollutant level would be removed if a station, or group of stations, were built to transmit this runoff to existing lift stations. If the budget allows, an actual station could be built to positively demonstrate the effectiveness.

C. COMBINED STORMWATER/WASTEWATER STATION VS. STORMWATER EXCLUSIVE STATION

C.1 LIFT STATION

As discussed previously, in the conceptual plan it was proposed to build a 33,000 gpm station which would be equipped with automatic control valves to switch from pumping stormwater to bypassing the regional force main into the station for re-pumping as a "booster-type" station. The discharge from this station would be through a separate 48 inch force main proceeding from the lift station site along the West Napoleon Canal and North Woodlawn Avenue towards the WWTP. The 48 inch size was initially selected in an effort to minimize construction costs. It was considered to be the minimum size that could handle the 33,000 gpm flow as this size resulted in velocities of nearly 6 feet per second (the maximum desirable for sewer installations). While a larger size could have provided additional hydraulic benefits (lower head and velocity), it would have been at an increased cost which was perceived to be beyond the financial constraints of this demonstration project.

The bypassing of the regional force main into the lift station would reduce the head in the downstream portion of the force main allowing the four downstream stations, which are manifolded into the force main, to be more efficient and eliminate some existing sewer overflows. In the conceptual design stage, it was assumed that this regional force main, which commences at lift station G6-9 located at the former Helios Treatment Plant site, discharged into the station at the former West Napoleon Treatment Plant site, Station F6-2 (See Figure IV.4). This station, in turn, discharged through a larger diameter force main directly to the headworks of the WWTP. During the elevation phase, it was determined that the force main does not discharge into station F6-2 but rather is manifolded with a force main from Station F6-2. Therefore, to divert the Helios flow to the new lift station would greatly affect the design of the Helios Lift Station, as well as lift stations G6-4 and F6-13 because the head conditions of the pumps would vary drastically. This is due to the fact that the Helios pumps, when pumping to the new station, would be pumping through a force main less than half the length of the regional force main. The only solution to such diverse head requirements would be to install two-speed or variable-speed pumps and motors at these three stations to handle the widely different head conditions. This would be a costly solution, both initially and long term, as maintenance on this type of equipment is generally much greater. Therefore, this became a much less attractive alternative.

In an effort to avoid this situation, another solution was evaluated and is recommended. In lieu of building a separate force main along West Napoleon between Suburban Canal and North Woodlawn Avenue, the existing force main would be connected to a new branch provided down North Woodlawn Avenue. Included in this scenario is the installation of two gate valves which allow the flows from Helios/L.S., G6-4/L.S., F6-13/L.S. and F6-11/L.S. to be diverted to the new force main along Woodlawn Avenue on a regular basis instead of only in wet weather as originally planned. This configuration allows the operation of the new lift station to become simplified as the station becomes a stormwater only station and eliminates the need for the automatic valves which would have required a manual switch to change from pumping stormwater to pumping sewage. An additional benefit of this scenario is that should a break occur in the regional force main downstream of North Woodlawn Avenue, the "normally-closed" gate valve could be opened to allow sewage from the downstream stations to be rerouted through the new force main to the WWTP on an emergency basis, thus eliminating, or at least greatly reducing, the large overflows that would otherwise occur if this alternate force main was not available.

This new scenario utilizes the existing force main for approximately 1000 feet along West Napoleon Avenue. It was perceived that construction of the force main in this area would be difficult due to the location of the existing force main and an existing waterline as well as the planned construction of the roadway and canal paving. The only drawback is that the existing 48 inch line limits the size of the proposed lift station. It is anticipated that approximately 13,000 gpm of sewage will be present in the force main during average dry weather conditions. This allows for 20,000 gpm of stormwater to be introduced and this provides a basis for sizing the lift station design flow.

Figure III.1 provides an illustration of the proposed force main layout with respect to the proposed lift station location. The subject is discussed in greater detail in Section IV-B.

C.2 FORCE MAIN

Alternative sizes of the proposed force main have also been evaluated. Although originally conceived as a 48 inch line (sized for the 33,000 gpm station), this size can neither allow for expansion of the program in the future, nor adequately convey the required amount of sewage in the event of a line failure as discussed above. Future expansion of the program should be considered at this time since this is a demonstration project with aspirations of future implementation on a parish-wide basis.

Hydraulic calculations and cost estimates were computed for 48 inch, 54 inch and 60 inch lines. When compared to the 48 inch line, the 54 inch size would allow for an additional 12,000 gpm while the 60 inch can accommodate an additional 26,000 gpm future stormwater flow. Therefore, the 60 inch could provide more than twice the stormwater pumping ability than this demonstration project. Cost estimates for all three sizes are provided in Appendix A. The typical section for constructing the force main is presented as Figure III.2. This section was used as the basis for computing quantities for the cost estimates.

It is prudent that future consideration be taken into account when installing a force main of such significant size. The incremental cost differential can be considered fairly minor and basically represents only the increased material cost of the pipe. Costs such as

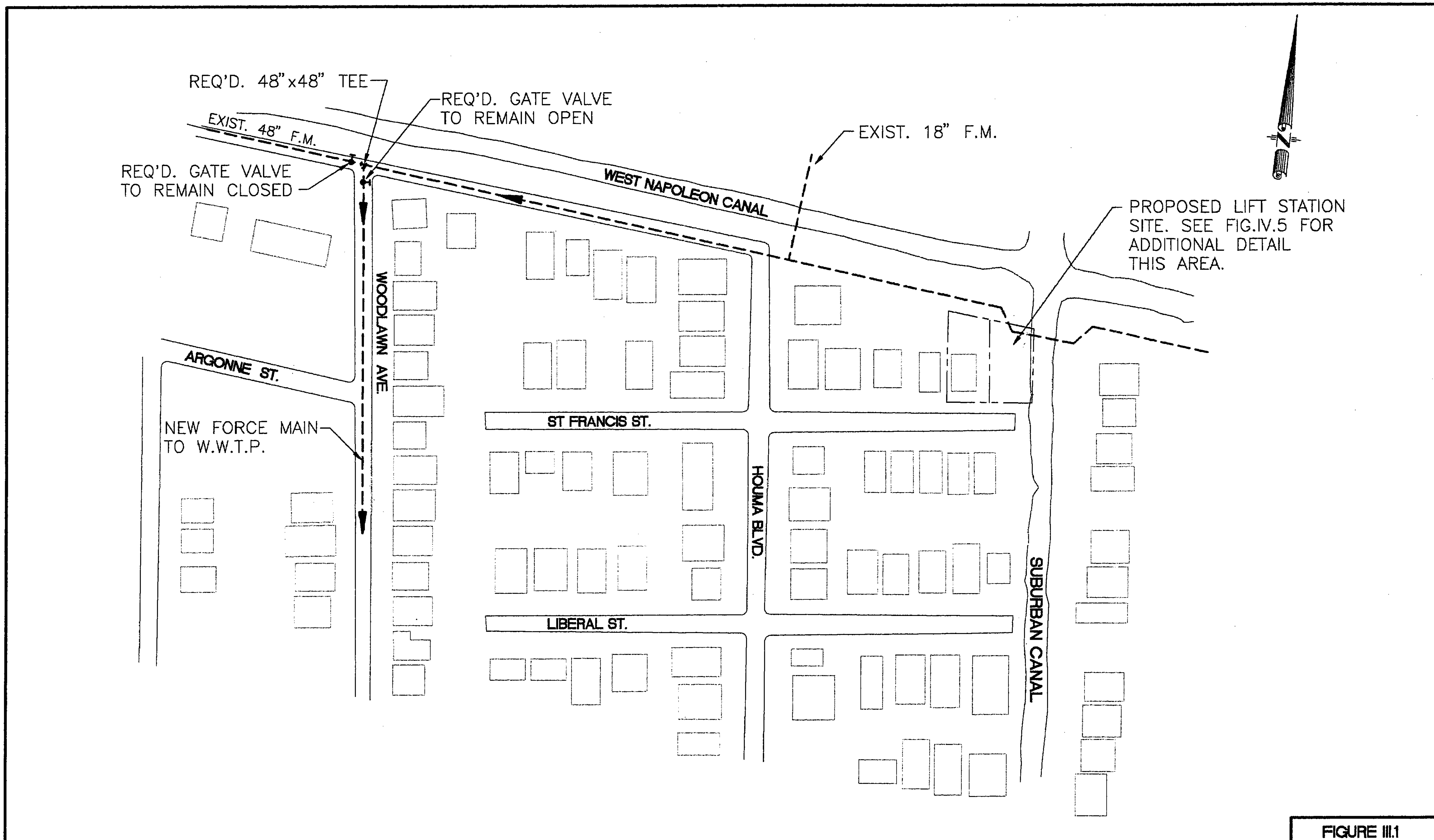

FIGURE III.1

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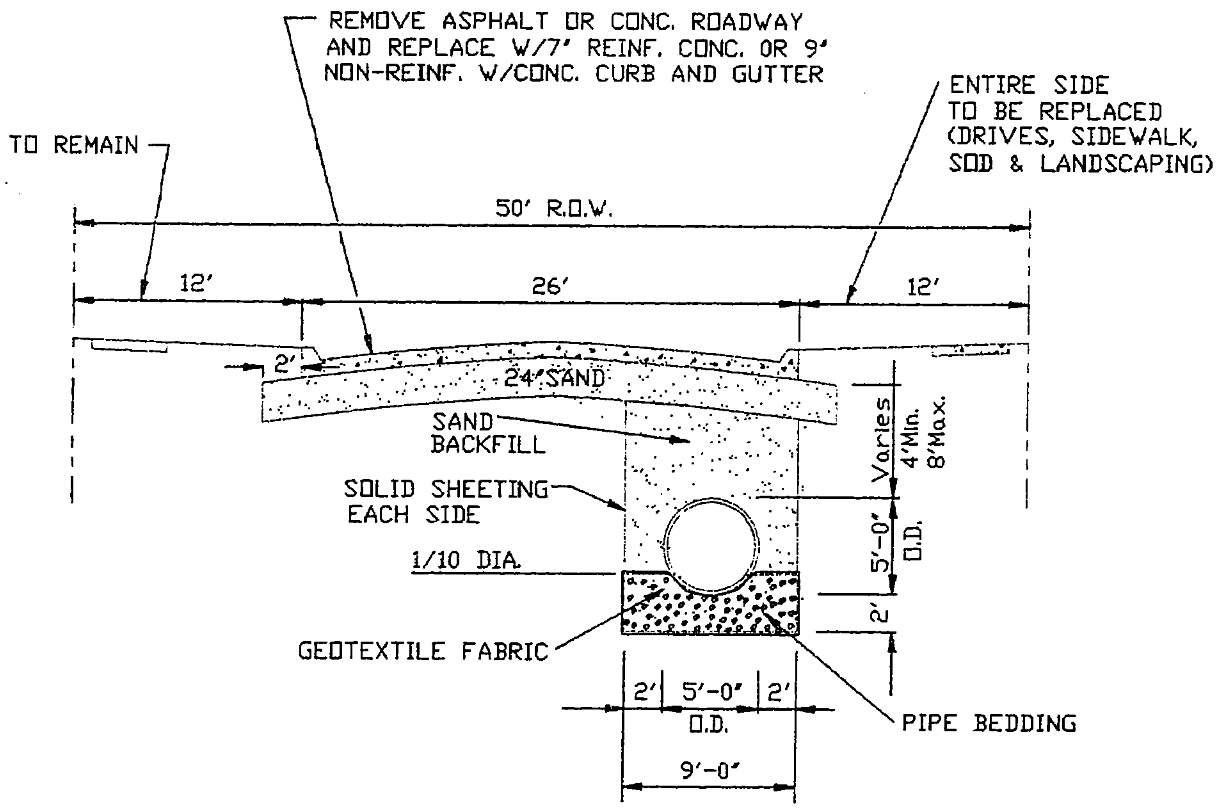
PROJECT SITE LAYOUT



TYPICAL SECTION

INSTALLATION OF 48"ϕ FORCE MAIN

FIGURE III.2

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TYPICAL SECTION
48"ϕ FOFCE MAIN

mobilization and site restoration would not increase, while the costs of excavation, bedding material and backfill would only represent minor increases. The cost to install an additional line in the future which could provide equivalent capacity would be significantly greater than the incremental cost associated with a slightly larger force main installed initially and would result in additional disruptions in the established neighborhoods. It is also questionable whether another favorable corridor could be found in the future to install a new main. The estimated cost of installing the 60 inch pipe is only 6.67% greater than the previously estimated cost of the 48 inch size, and it is hereby recommended. Hydraulic calculations for the 48 inch and 60 inch sizes are presented in Figure IV.5.

D. TYPES OF PUMPS

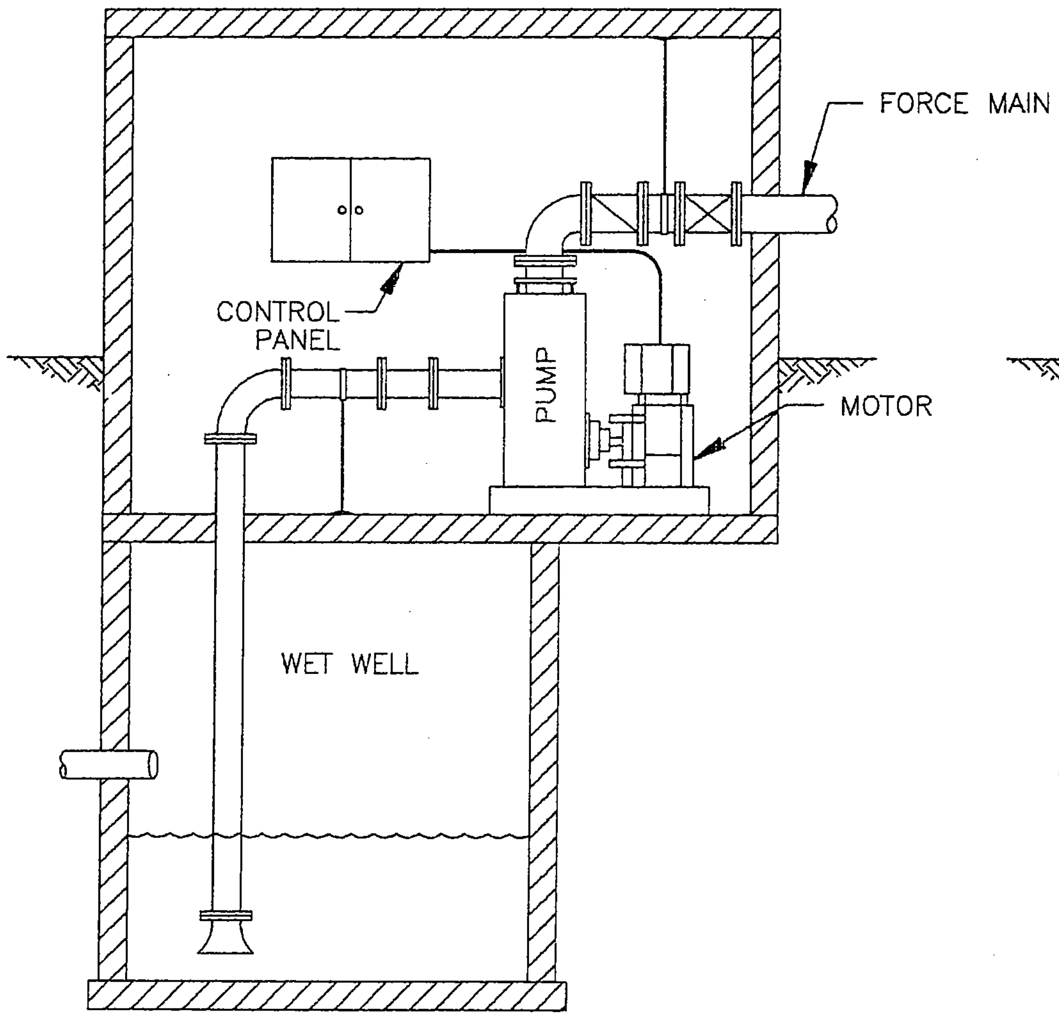
Under the stormwater only station scenario the proposed size of the station has been reduced to 20,000 gpm. This is consistent with the volume which can generally be anticipated to be extracted from the canals at this location in a 24 hour period (See Section IV A). It is also sized so as not to overburden the system should a 48 inch force main be selected to be constructed.

The 20,000 gpm capacity can best be achieved by installing five 4,000 gpm pumps, which can be operated either one at a time or with all five simultaneously as the canal levels allow. It is not recommended that a spare pump be installed as the quantity of water transmitted by this station is not a critical function of the overall success of the project. One pump could be removed for maintenance for a short period of time without drastically affecting the operation of the station. It is recommended that this station be designed for raw sewage applications in the event the need arises in the future to tie new or existing lift stations to this station. Also, should this program be discontinued after the demonstration phase, this station could be used by the Sewage Department, if necessary, to eliminate overflows.

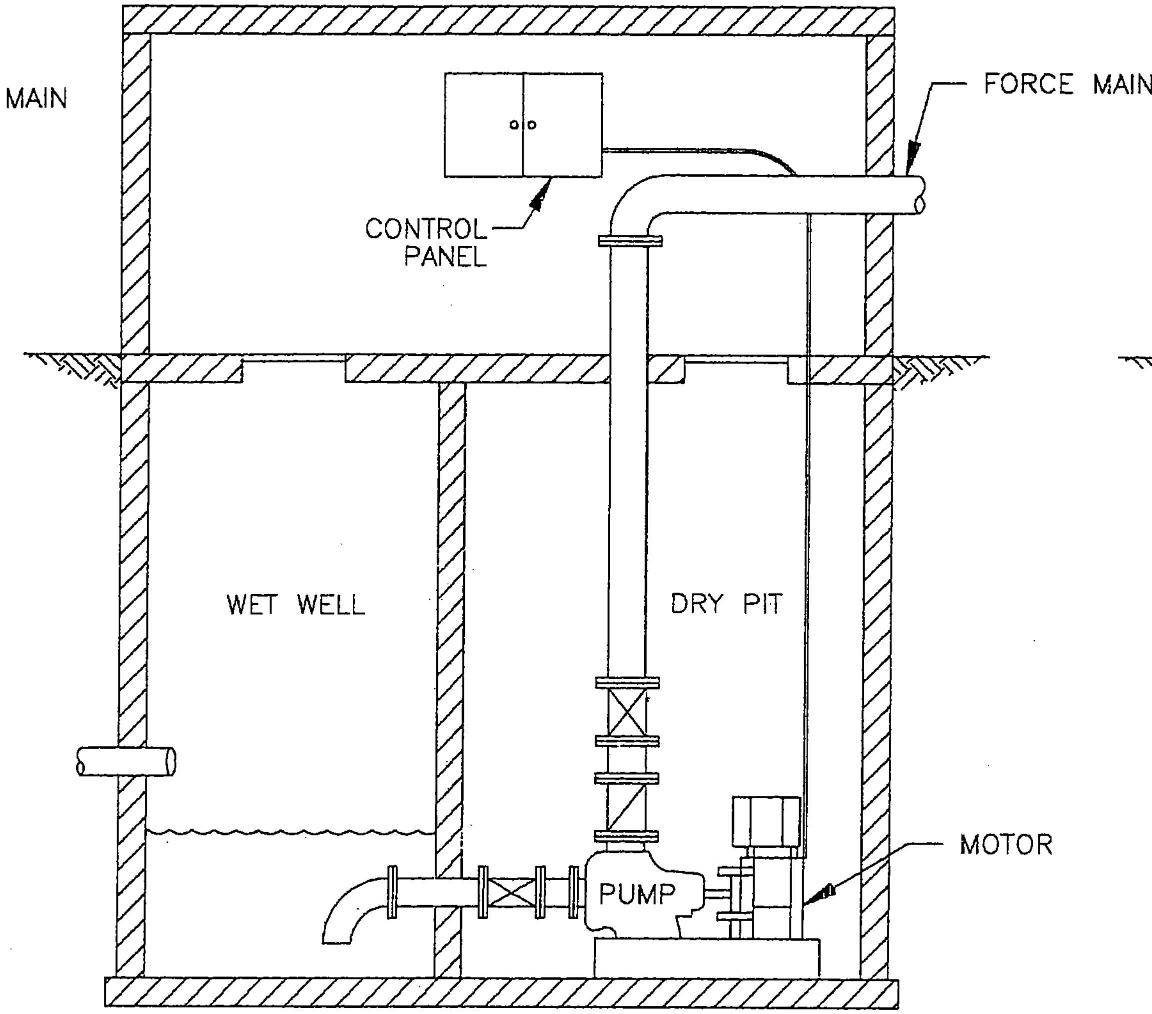
There are three viable alternative pump types for this size station: 1) Vertical Turbine Solids Handling (VTSH), 2) self-priming and 3) dry pit. Each type of pump has its own unique advantages and disadvantages. A basic schematic view of each type of pump station is shown on Fig. III.3

The VTSH pump is an above-ground pump which can handle large capacities, total heads, and size of solids. The primary disadvantage is that they are virtually a sole-source pump with Fairbanks-Morse Corporation being the only known manufacturer of a turbine pump that can handle raw sewage. The equipment costs for this type of station may be greater at the time of original purchase but this difference is easily offset when compared to the dry pit type due to a reduced deep excavation size. This pump also offers an advantage in that all electric components and pump motors are located above ground and will, therefor, not be damaged in the event of a major flood or power outage.

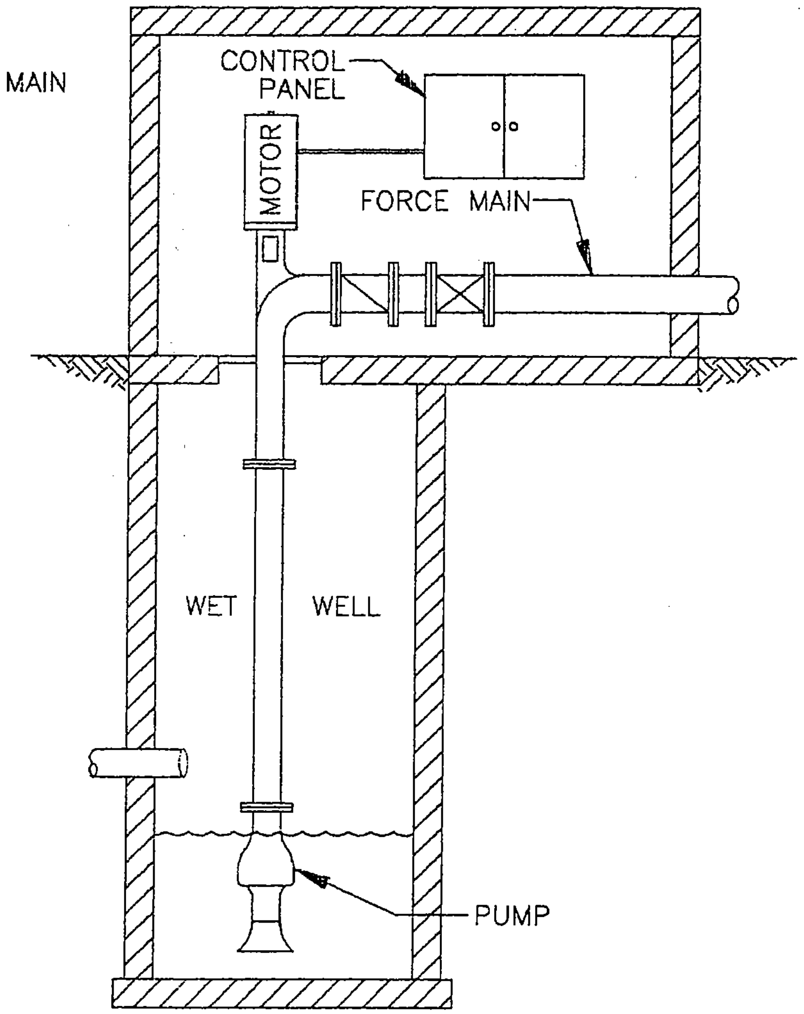
The self-priming pump is also an above ground pump but typically handles much smaller capacities. The above ground pumps also allow for a reduced construction cost since a larger deep dry pit is not required. The self-priming alternate could be implemented by either using the largest pump available from Gorman-Rupp or additional smaller pumps. This type of pump is not typically as efficient as the other two alternatives, and as with the VTSH, a sole source



TYPICAL SELF-PRIMING PUMP STATION



TYPICAL DRY PIT PUMP STATION



TYPICAL VTSH PUMP STATION

FIGURE III.3

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COMPARISON OF PUMP TYPES

situation may arise if another manufacturer cannot supply the larger size pumps this station will require. This type of pump is limited by its' suction lift capabilities and may result in the pumps needing to be recessed below ground to accommodate this shortcoming.

The dry pit pump would typically be the standard choice for a sewage pump with these types of capacities, and many models are available, but an increased construction cost is required because of the depth of the dry pit. This type of pump is a dependable, efficient pump with a long history of service in the raw sewage industry.

This is an excellent application for a VTSH installation and it is recommended that this pump type be utilized. The next best alternative would be the traditional wet pit/dry pit type station. An alternative design for each type of station could be accomplished to allow for competitive bids. It is recommended that two designs, one for a VTSH and one for a dry pit station be included in the bid plans.

E. FORCE MAIN MATERIAL

As previously discussed, three sizes of force main have been considered: 48 inch, 54 inch, and 60 inch. Four types of material are available for pressure pipes of this size: ductile iron, prestressed concrete, fiberglass and steel. As with the pumps, each type has its own unique advantages and disadvantages. Estimated prices have been obtained from suppliers of all four materials and all would appear to be very competitive. Ductile iron and prestressed concrete have been the materials of choice for sewage force main in the past, but numerous problems have occurred recently with the prestressed pipe. Fiberglass is a relative newcomer to the sewage market, while steel requires field welded joints which typically slows down the installation process. It is recommended that the force main be bid with at least two alternate types specified and that an epoxy interior lining and exterior coating be specified for each type.

F. FORCE MAIN ROUTE ALTERNATIVES

Several force main routes from the lift station site to the WWTP were considered in an effort to arrive at the most cost effective solution. The conceptual phase route, which parallels West Napoleon Avenue to North Woodlawn Avenue, then proceeds south along North Woodlawn Avenue to the Southern Pacific Railroad tracks, then along the railroad right-of-way to the WWTP remains the most viable alternative. Two alternate routes, or portions of routes, were considered. One alternate considered placing the force main within the Suburban Canal right-of-way to West Metairie Avenue, thence along West Metairie Avenue and Anthony Street to North Woodlawn Avenue then continuing along the previously discussed route. The other alternative involved altering the southern portion of the conceptual phase route. This alternative considered utilizing the former Kansas City Southern Railroad right-of-way paralleling Airline Highway. It would utilize this right-of-way between North Woodlawn Avenue and St. Peter's Ditch, thence proceed south within the St. Peter's Ditch right-of-way to the WWTP. The alternative routes considered are shown on Figure III.4.

The route along the Suburban Canal was perceived as an alternative to disrupting the residential neighborhoods along the North Woodlawn Avenue route. The disadvantages to this route however, are numerous. For one, the Suburban Canal between West Napoleon Avenue and

West Metairie Avenue has insufficient right-of-way which precludes it from being able to be widened to its' required drainage cross section. Due to this insufficient right-of-way and access difficulties, construction of a force main through this reach would be both difficult and costly. Future improvements required for drainage purposes were considered which also increased the cost. This would require deep steel sheeting to be left in place. Additionally, as a direct result of canal bank stability problems, Jefferson Parish is involved in numerous lawsuits with residents along the Suburban Canal. In conversations with contractors familiar with this type of work, it was revealed that they would typically increase their bid by approximately $5,000.00 per residence along the canal project to offset anticipated damages and allow for detailed pre-construction video surveys.

Once constructed, the force main would be difficult to access for required maintenance. In addition, the Suburban Canal ends at West Metairie Avenue hence a route through residential neighborhoods is ultimately required. This portion of the force main route crosses West Metairie Avenue and proceeds west on Anthony Street to North Woodlawn Avenue then continues south on North Woodlawn Avenue as originally proposed. The estimated cost for constructing the force main along the Suburban Canal is approximately $4,600,000.00 greater than the recommended alternative for the 48 inch diameter size. The typical section depicting the force main installation in this reach is shown on Figure III.5.

An alternate route between Airline Highway and the WWTP was also considered. The former right-of-way for the KCS Railroad parallels Airline Highway on the south and is an unobstructed parcel of land with no existing underground utilities. It was anticipated that the force main could be constructed within this right-of-way between North Woodlawn Avenue and St. Peter's Ditch, which is located on the east side of the Garden of Memories Cemetery. A jack and bore would be required beneath Clearview Parkway. The force main would continue south within the right-of-way of St. Peter's Ditch to the WWTP.

Two potential problems were identified with this alternative. The KCS right-of-way is owned by Robco, Inc., a corporation which has sold right-of-ways to Jefferson Parish in the past. The acquisition of servitude through this area was perceived to be quite costly based on previous right-of-way purchases. The other perceived problem was the St. Peter's Ditch right-of-way. This right-of-way is owned by the Garden of Memories Cemetery and was granted for drainage purposes only. It is doubtful that it would be expanded for sewage purposes, especially since the proposed route would bisect a newly developed portion of the cemetery. For these two reasons this alternative was abandoned.

The selected route would be entirely constructed within existing Jefferson Parish rights-of-way with two exceptions. A permit will be required from Louisiana DOTD for crossing Airline Highway. Rights-of-way will need to be purchased from three railroad companies for the east/west portion of the force main south of Airline Highway. All Owners have been contacted and made aware of this proposed project and no objections are foreseen with either of these situations.

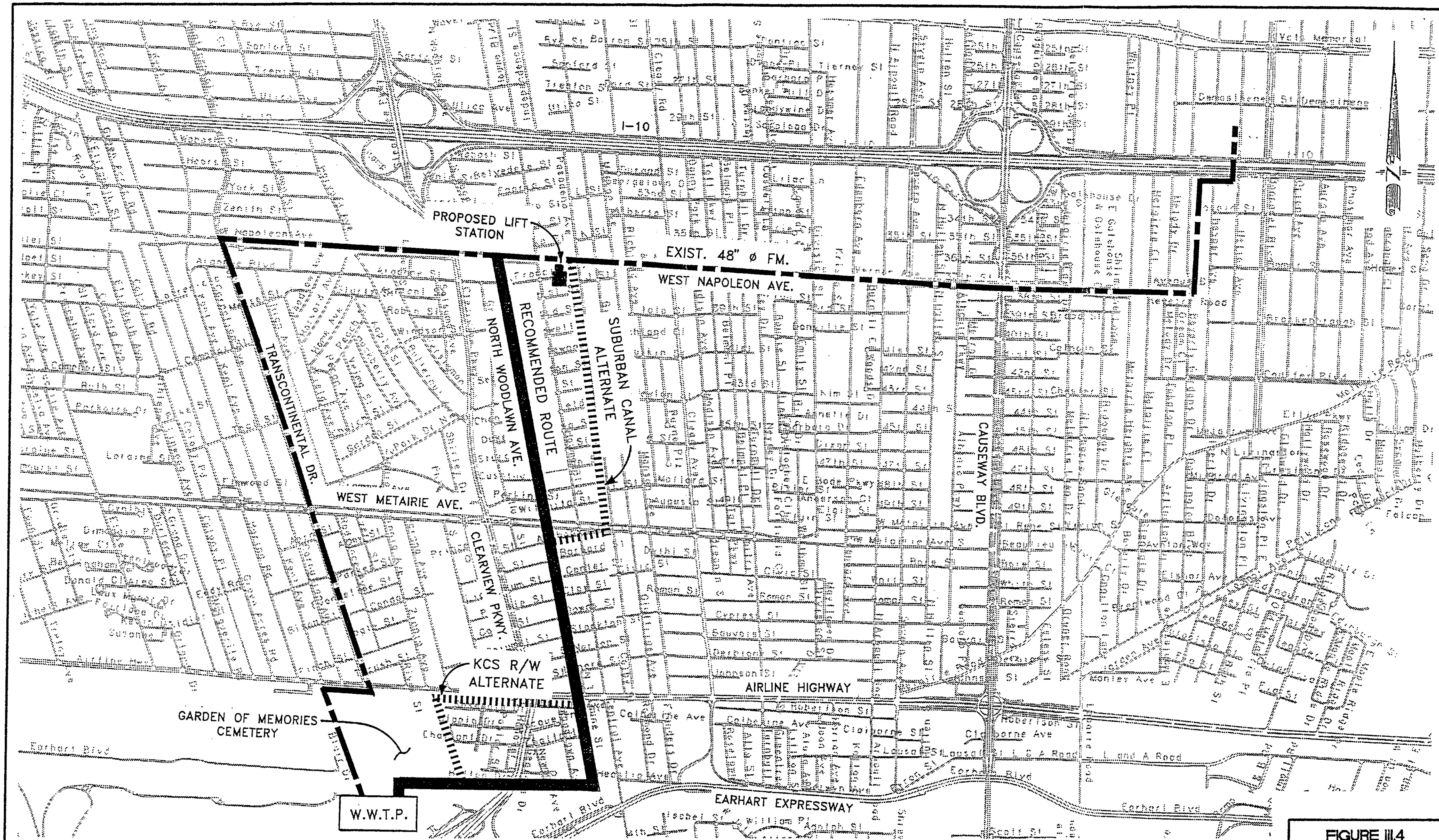

FIGURE III.4

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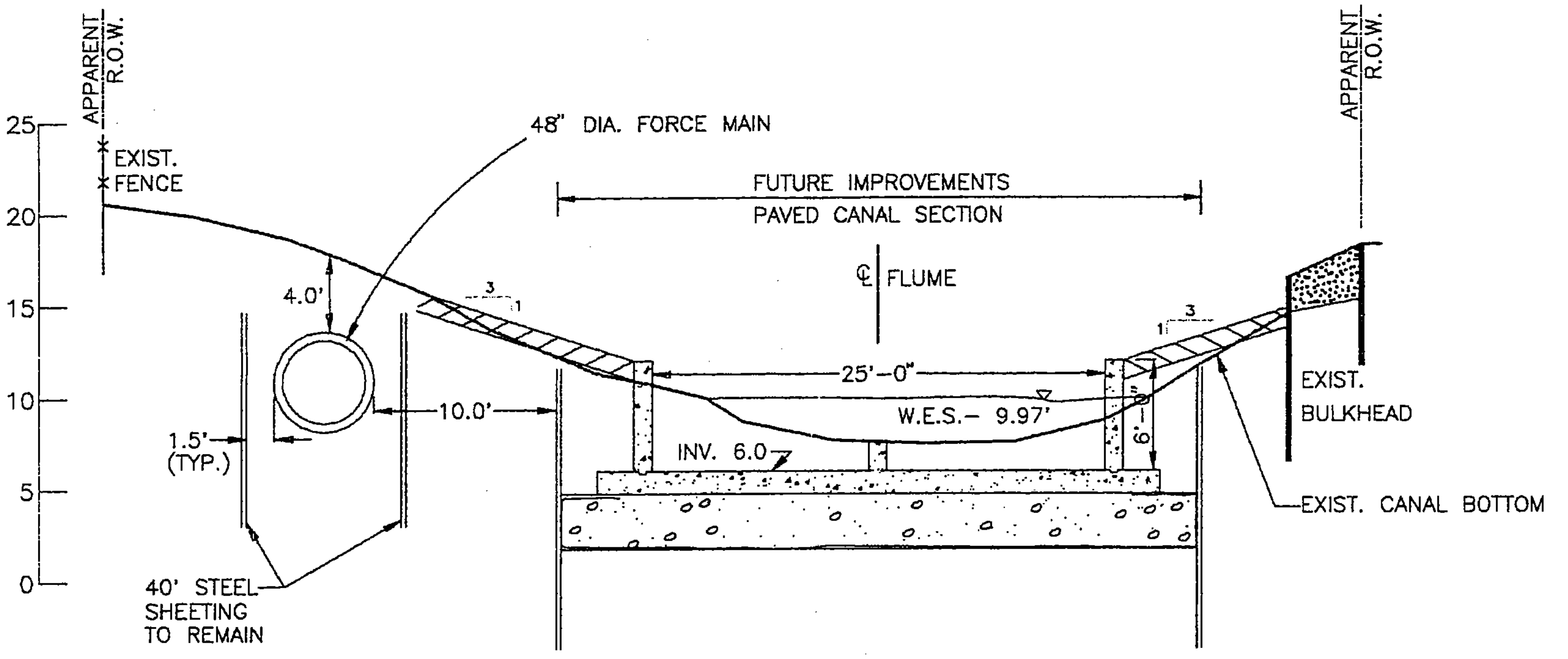
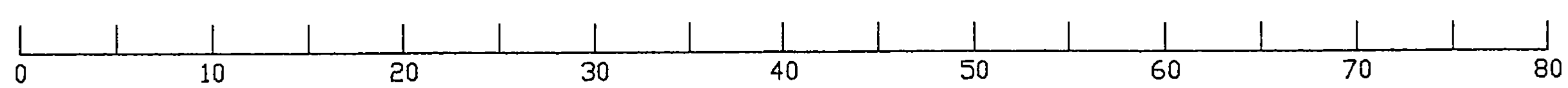
FORCE MAIN ROUTE ALTERNATIVES



TYPICAL SECTION 48" DIA. FORCE MAIN
ALONG SUBURBAN CANAL
(LOOKING NORTH)

SCALE : 1" = 10.00'



NOTE:

APPARENT R.O.W. SHOWN WAS TAKEN FROM JEFFERSON PARISH PLANS. ACTUAL FIELD CONDITIONS INDICATES LESS R.O.W. IS ACTUALLY PRESENT.

FIGURE III.5

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JEFFERSON PARISH STORMWATER DISCHARGE DEMONSTRATION PROJECT

TYPICAL SECTION - FORCE MAIN INSTALLATION ALONG SUBURBAN CANAL

SECTION IV
OPERATIONAL REQUIREMENTS

A. DRAINAGE PUMP STATION OPERATIONS

1. CURRENT OPERATING PROCEDURES

Discussions were held with Mr. Ross Ketchum, Director of Drainage Pump Stations, Mr. Doan Modianos, expert consultant to Jefferson Parish, Mr. Norman St. Martin, Superintendent of East Bank Pump Stations, and Mr. Prat Reddy, Director of Drainage to determine typical pump station operating practices and satisfactory canal drawdown levels. In conjunction with this, canal storage volumes were computed and compared with historical rainfall data.

Typical pump station operation is for the drainage canal level to be maintained at approximate elevation 9.0 Cairo Datum (C.D.). When a significant rainfall event is anticipated, the canals are pumped down one to two feet to provide additional storage. When a rainfall event occurs, the pumps at each individual station are turned on once a rise in the canal level is experienced at the station. Should the canal level be lowered and the anticipated rain not occur, backflushing from Pump Stations No. 1 and No. 4 is performed to raise the canals to their static level. Experience has proven that these canal elevations provide an effective drainage plan while also prohibiting drawdown of the surrounding water table elevation and canal bank sloughing. See Figure IV.1 for illustration of the East Bank major drainage network.

Using these operational parameters, estimates were made to quantify the type of event that could be accommodated with the proposed lift station without adversely affecting the drainage pump operations. Since the proposed lift station is located on the Suburban Canal at a central location it was assumed that the only area which would be impacted by the new lift station would be the Suburban Canal and the immediate adjacent portions intersecting the east/west canals. The storage basin used for these computations is depicted on Figure IV.2. Isolating this area equates to a total storage volume of approximately 20,000,000 gallons for a two foot differential in canal level. This 20,000,000 gallons also represents an approximate ¼ inch rainfall across the entire Suburban basin when a 50% runoff factor is assumed. Figure IV.3 presents this data in tabular form. Various pump station sizes were considered and a 20,000 gpm station was selected as the design flow. This size station allows for maximum stormwater removal within the restraints of not over-burdening the existing 48 inch sewage force main. Under this scenario the two foot storage of stormwater within the Suburban Canal storage basin could be extracted in a 17 hour period.

2. PROPOSED OPERATIONAL PLAN

Using these figures, the operational plan for the lift station was developed and discussed with Jefferson Parish officials. When a rain event is forecast, the proposed lift station would begin pumping the existing stored stormwater out of the canals to obtain two feet of storage. This would bring the canal levels to approximate elevation 7.0 (C.D.). Once

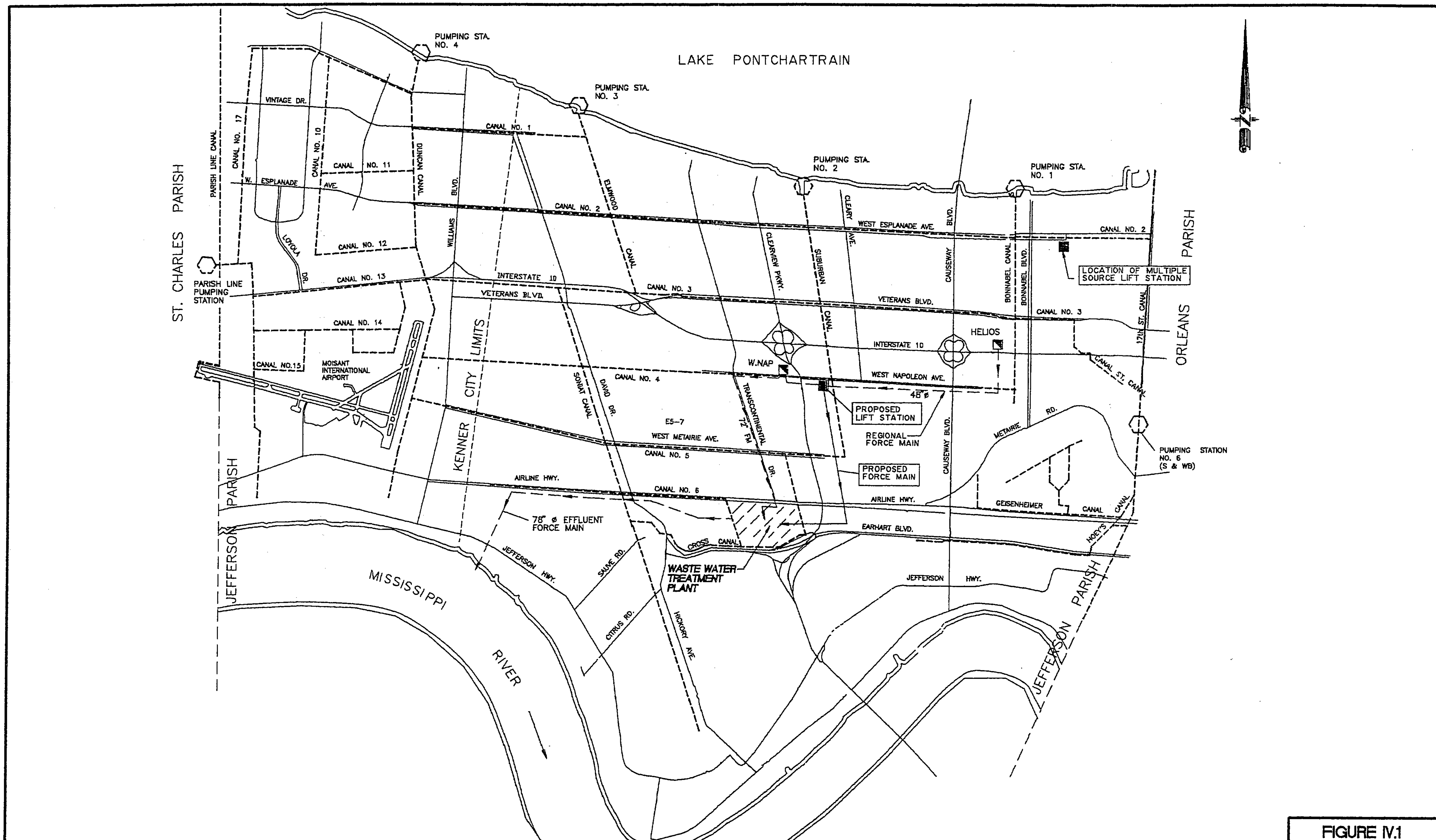

FIGURE IV.1

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MAJOR DRAINAGE NETWORK

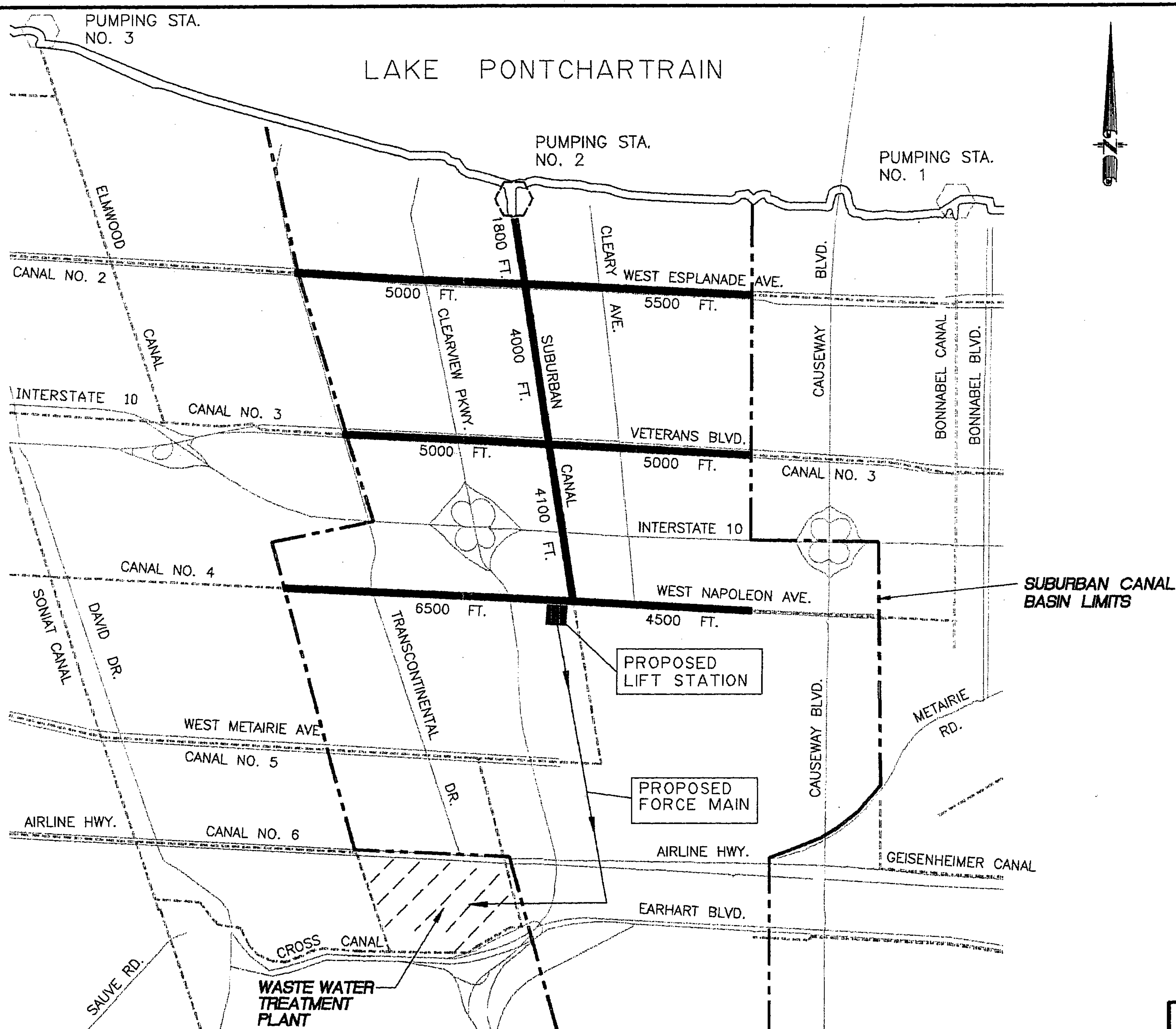

FIGURE IV.2

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JEFFERSON PARISH STORMWATER DISCHARGE DEMONSTRATION PROJECT

SUBURBAN CANAL STORAGE BASIN

STORMWATER QUALITY DEMONSTRATION PROJECT
PROJECTED PUMPING VOLUMES

CANAL	SUBURB. CANAL SIDE	AVG. WIDTH (FT.)	APPROX. LENGTH (FT.)	DEPTH (FT.)	TOTAL VOLUME CU. FT.	TOTAL VOLUME GALLONS
WEST ESPLANADE (C2)	WEST	25	5000	2	250,000	1,870,250
WEST ESPLANADE (C2)	EAST	30	5500	2	330,000	2,468,730
VETERANS BLVD. (C3)	WEST	25	5000	2	250,000	1,870,250
VETERANS BLVD. (C3)	EAST	25	5000	2	250,000	1,870,250
WEST NAPOLEAN (C4)	WEST	20	6500	2	260,000	1,945,060
WEST NAPOLEAN (C4)	EAST	25	4500	2	225,000	1,683,225
SUBURBAN – C2 TO STATION		75	1800	2	270,000	2,019,870
SUBURBAN – C3 TO C2		60	4100	2	492,000	3,680,652
SUBURBAN – C4 TO C3		45	4100	2	369,000	2,760,489

CANAL	SUBURB. CANAL SIDE	STORED VOLUME (GAL.)	PUMP STATION SIZE *14,000 GPM*	*16,000 GPM*	*18,000 GPM*	*20,000 GPM*
			PUMPING DURATION (HRS)			
WEST ESPLANADE (C2)	WEST	1,870,250	2.2	1.9	1.7	1.6
WEST ESPLANADE (C2)	EAST	2,468,730	2.9	2.6	2.3	2.1
VETERANS BLVD. (C3)	WEST	1,870,250	2.2	1.9	1.7	1.6
VETERANS BLVD. (C3)	EAST	1,870,250	2.2	1.9	1.7	1.6
WEST NAPOLEAN (C4)	WEST	1,945,060	2.3	2.0	1.8	1.6
WEST NAPOLEAN (C4)	EAST	1,683,225	2.0	1.8	1.6	1.4
SUBURBAN – C2 TO STATION		2,019,870	2.4	2.1	1.9	1.7
SUBURBAN – C3 TO C2		3,680,652	4.4	3.8	3.4	3.1
SUBURBAN – C4 TO C3		2,760,489	3.3	2.9	2.6	2.3
		TOTAL	24.0	21.0	18.7	16.8

FIGURE IV . 3

an event begins, the lift station would continue operating to attempt to "capture" as much of the first flush stormwater as possible. Since this project is a demonstration of the ultimate future plan, the station is not large enough to capture all of the first flush stormwater. Therefore, as the rain event progresses the canal level will rise.

It is anticipated that in a ¼ to ½ inch event over the entire basin, the canal level will not rise beyond elevation 10.0 (C.D.), the approximate current "on" elevation for the drainage pumps. The drainage pumps in this situation will not have to be operated. Assuming the rain stops before elevation 10.0 is reached, the lift station will then continue pumping the stored runoff down to elevation 7.0. This would be achieved within an 18 to 24 hour period. Once this occurs, and assuming another storm event is not forecast, the canals would be backflushed with lake waters to raise the canal level back up to the static 9.0 elevation. At this point the new lift station can again be operated in order to "cleanse" the canals. Once elevation 7.0 is again reached the backflushing cycle will again commence (assuming no other rain event is anticipated). This pumping and backflushing scenario would occur over a three to four day period.

It is recommended that once the second backflushing is complete the cycle be stopped to wait for the next storm event in order to avoid circulating and treating only lake water. Historical data has shown that approximately 80% of the rainfall events in Jefferson Parish are of the ¼ to ½ inch magnitude and typically occur three to four days apart. Therefore, while the lift station operation will not be effective in all rain events, it would be effective in the majority of events and will significantly decrease the amount of stormwater pumped into the lake from the Suburban Canal Pump Station.

B. STORMWATER AFFECT ON SEWERAGE FORCE MAIN OPERATIONS

As previously discussed the existing regional force main is a combined discharge manifold from seven large sewage lift stations. Figure IV.4 illustrates the force main size, route and lift station capacities. These capacities represent peak wet weather flows. Typically, average dry weather flow will be on the order of one-third of these peak flows, leaving two-thirds of the capacity available for stormwater in dry weather. This was the basis for computing the size of the proposed stormwater lift station and force main.

Preliminary hydraulic calculations were performed to determine the affect pumping stormwater would have on the seven regional force main stations. Figure IV.5 presents the existing design capacities and total heads for each of the seven stations and the affect of the new force main for both the 48 and 60 inch sizes. The 54 inch size is neglected as the incremental cost-benefit ratio is not as great for the 54 inch when compared to the 48 inch as the 60 inch is when compared to either the 54 inch or 48 inch. It can be seen from these figures that a substantial increase in station capacities will be achieved at the three stations downstream of the new force main tie-in, while no impact is felt on the four upstream stations when a new 48 inch is used. For a 60 inch force main, substantial increases at all lift stations are achieved. This provides additional pumping capacities which will allow the stations to keep up with wet weather flows and eliminate overflows.

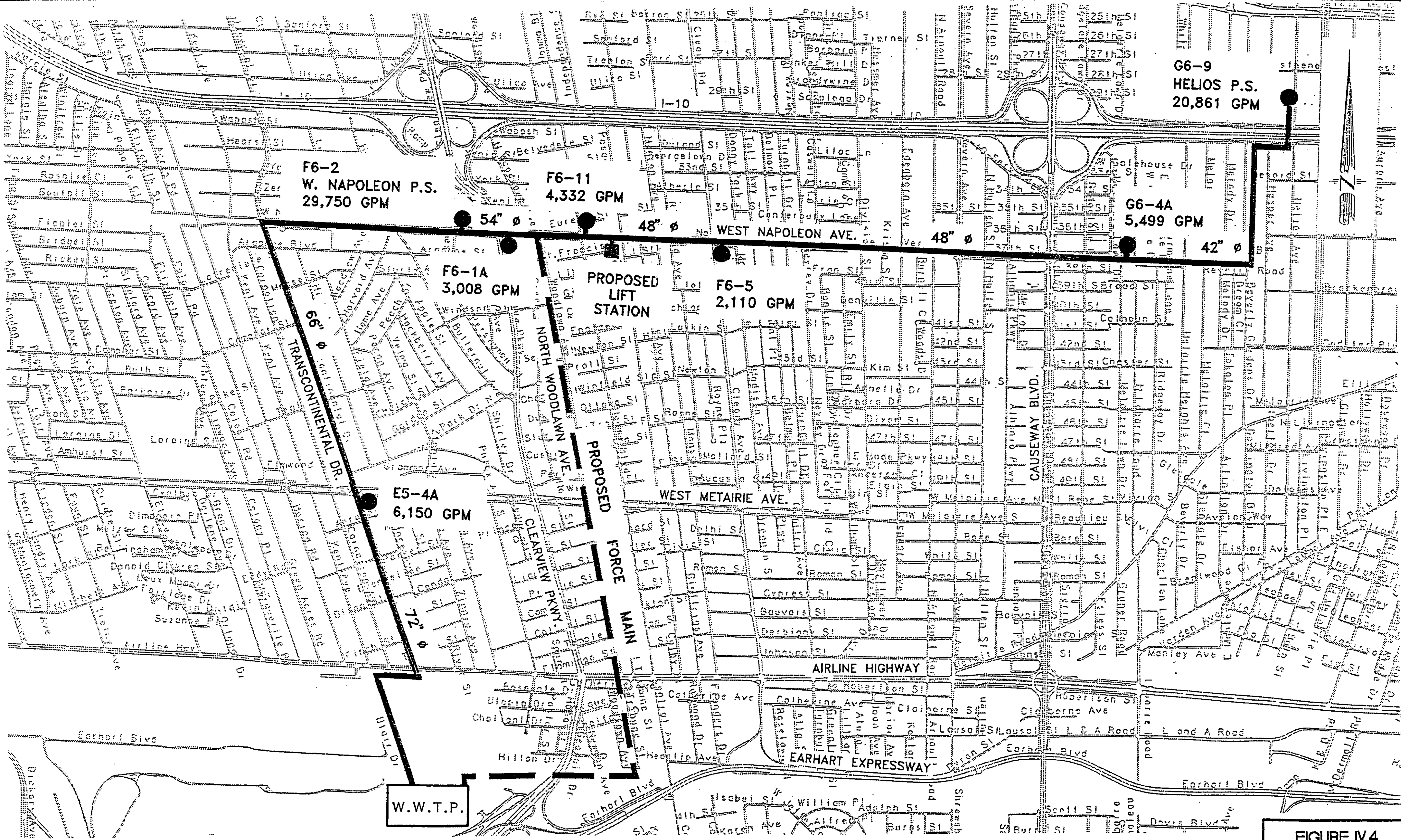

FIGURE IV.4

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MAJOR SEWERAGE NETWORK

FIGURE IV.5 : SUMMARY OF LIFT STATION HYDRAULICS

LIFT STATION NO.	LOCATION	NO. OF PUMPS	EXISTING CONDITIONS			WITH NEW 48" FORCE MAIN (PEAK FLOW)					WITH NEW 60" FORCE MAIN (PEAK FLOW)				
			DESIGN CAPACITY PER PUMP (GPM)	TOTAL CAPACITY OF STATION (GPM)	TOTAL DYNAMIC HEAD (FEET)	CAPACITY PER PUMP (GPM)	TOTAL CAPACITY OF STATION (GPM)	TOTAL DYNAMIC HEAD (FEET)	PUMP IN EFFICIENCY RANGE ?	PERCENT INCREASE IN FLOW	CAPACITY PER PUMP (GPM)	TOTAL CAPACITY OF STATION (GPM)	TOTAL DYNAMIC HEAD (FEET)	PUMP IN EFFICIENCY RANGE ?	PERCENT INCREASE IN FLOW
G6-9	FORMER HELIOS TREATMENT PLANT	5	5160	20,861	96	5000	20,000	97	YES	0	6000	24,000	86	YES	15.0
G6-4	GALLERIA	4	1833	5499	86	1825	5475	88	YES	0	2450	7350	76	YES	33.7
F6-13	CLEARY & WEST NAPOLEON (SOUTH)	2	2110	2110	72	PUMP	CURVE	NOT	AVAILABLE	0	PUMP	CURVE	NOT	AVAILABLE	?
F6-11	HOUMA & WEST NAPOLEON	4	1444	4332	75.4	1425	4275	77	YES	0	1900	5700	68	YES	31.6
F6-1	CLEARVIEW & WEST NAPOLEON	3	1500	3008	59	2200	4400	51	YES	46.3	2200	4400	51	YES	46.3
F6-2	FORMER W.NAPOLEON TREATMENT PLANT	5	7400	29,750	65	10,200	40,800	54	YES	37.1	10,200	40,800	54	YES	37.1
E5-4	WEST METAIRIE & TRANSCONTINENTAL	4	2050	6150	87	2250	66750	84	YES	9.8	2250	6750	84	YES	9.8

Another benefit with the installation of the new force main will be achieved should a break ever occur in either the existing or new force main, as the installation of gate valves will allow flows to be diverted away from the break. This would be a significant benefit to the quality of water in Lake Pontchartrain, as well as the open canal system, since if a break would occur under the present layout, the only available method of disposing of the large amounts of raw sewage would be through the stormwater system. Therefore, the construction of the new force main provides numerous benefits to the quality of the lake water as it allows the entire northern half of the unincorporated East Jefferson sewage system to perform more efficiently, thereby reducing the amount of fecal coliform in Lake Pontchartrain. The benefits are even more pronounced with the installation of a 60 inch line in lieu of a 48 inch.

C. LIFT STATION

The preliminary recommendation is to build a 20,000 gpm lift station with sewage pumps that would be gravity fed from an inlet in the Suburban Canal. It is recommended that either vertical turbine solids handling pumps or dry pit centrifugal pumps be utilized. The pump requirements for either type of station are virtually identical. The difference between the two types of stations is primarily physical as the dry pit necessitates a larger and deeper station size. Utilizing either type of pump, the recommendation is to use five (5) 4000 gpm pumps with 100 horsepower, 1180 rpm motors. The efficiency of each pump would be approximately 80%. A backup pump would not be provided.

Canal improvements for the intersection of the Canal No. 4 and the Suburban Canal are currently under design to provide for the construction of a triple cell box culvert. This work will be constructed under the West Napoleon Roadway Improvement Project which would also construct a divided four lane roadway along West Napoleon Avenue. It is projected that construction on this project will not begin until at least mid-1996.

The lift station would be designed to be fed by a 54 inch round or arch equivalent reinforced concrete suction pipe with the inlet located at the southern end of the proposed box culverts. A grated inlet would be provided across the canal invert at this point and an automatically operated bar screen would be installed within the suction line at the canal bank. It is recommended that a concrete paved canal bottom be provided along the Suburban Canal upstream of the inlet location for a minimum of 100 feet to help reduce the amount of sedimentation entering the inlet.

The invert elevation at the southern end of the proposed box culvert will be elevation 3.5 (C.D.). Providing for a totally submerged inlet, the bottom wet well invert for the lift station would be set at approximate elevation 0.0 (C.D.). As previously discussed, the pumps would typically be operated between elevations 7.0 and 10.0. This configuration provides proper submergence for the pump suction inlets. The pumps would be manifolded together into a common header with a 36 inch discharge line hot-tapped into the existing 48 inch prestressed concrete force main. Although a check and gate valve will be provided on each pump discharge, an additional check valve should be provided on the 36 inch discharge line prior to the tie-in with the 48 inch line to positively prohibit the back flow of raw sewage through the pumps and into the stormwater canals. Figure IV.6 provides a footprint of the proposed lift station and suction and discharge lines.

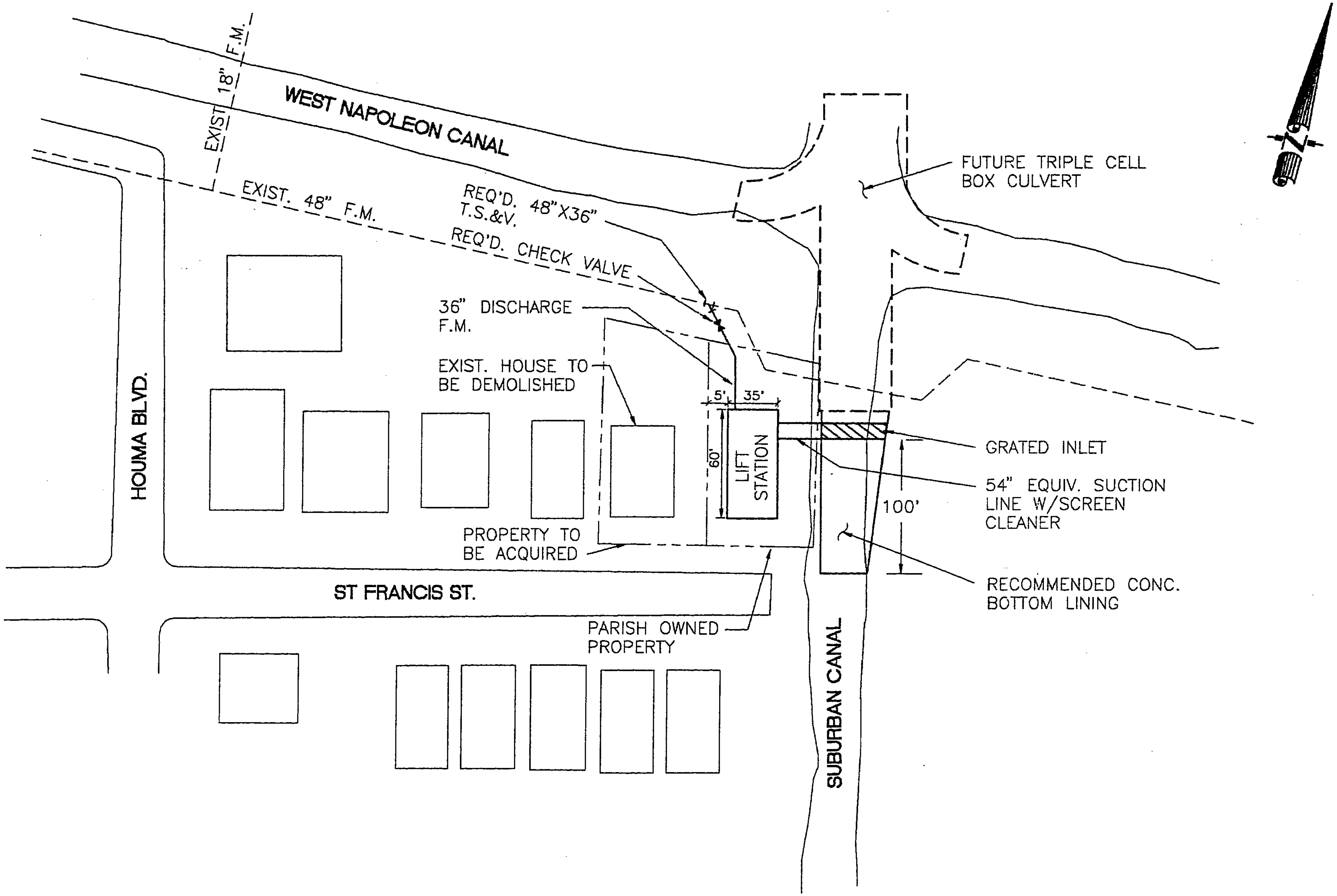

FIGURE IV.6

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JEFFERSON PARISH STORMWATER DISCHARGE DEMONSTRATION PROJECT

LIFT STATION LAYOUT

The lift station could be physically constructed within the confines of the property which is presently owned by the Jefferson Parish Department of Sewerage. There is, however, a need for additional right-of-way for access of construction equipment and operations. Sufficient room must be provided for excavation and storage purposes. In addition, the size of this station and its pumping equipment warrants a "buffer" between the station and adjacent residential properties to allow for maintenance access, exterior storage and to minimize negative impacts on the neighborhood. Therefore, it is recommended that the property adjacent to the proposed site be purchased and the existing residential structure demolished. The acquisition and demolition cost is estimated at $150,000.00.

D. OPERATION OF MULTIPLE SOURCE STATION

As part of this demonstration project, it is recommended that the existing Sewer Lift Station (H8-3A), at West William David and the south side of West Esplanade Avenue, be developed as a representative site for the Multiple Source Station Concept.

In the Multiple Source Station Concept, small east bank lift stations at various locations along the major drainage canal system would be used to convey stormwater to the WWTP.

Based on the results obtained at this station during this demonstration project, the concept can be evaluated for application at the various other locations.

At Lift Station H8-3A, stormwater can be collected from a defined sub-catchment area of the Parish drainage system. This area is roughly bounded by the levees at Lake Pontchartrain and the 17th Street Canal on the north and east and Veterans Highway and Papworth Avenue on the south and west, and is not connected with the other Parish canal systems.

In this isolated area, drainage nonpoint source pollution can be studied and pollutant effects from known point sources at specific locations, such as sanitary sewer overflow points, car washes, gas stations, mechanic shops, etc., can be analyzed. The pollutant stream can be analyzed from point of origin to point of discharge into the canal.

The existing sewer lift station has a wet weather design capacity of 3,120 gallons per minute. It contains two 42 horsepower submersible pumps, each with a design capacity of 1,560 GPM. The pumps can handle solids up to 3". It is anticipated that during dry weather periods, there would be approximately 2,000 GPM of surplus pumping capacity.

The construction at this site will include a temporary earthen dam, constructed so that it can be removed on short notice in the event of adverse effects on the canal's ability to accommodate stormwater. Just east of the dam, a sump will be created in the bottom of the canal, approximately 2½' deeper than the existing canal bottom and a screened intake constructed in the center of the sump. The intake elevation would be 1½' lower than the existing bottom of the canal (see Figure IV.7).

Water from the canal will be pumped into the existing wet well. A control panel will monitor the water level in the canal and the existing sanitary wet well and regulate the amount of stormwater being pumped. When the sanitary wet well is receiving high sanitary flows, storm water would not be introduced from the canal. This typically occurs during wet weather conditions.

Sensors will determine the surface water elevation in the canal and in the sanitary wet well. The control panel will regulate the amount of flow being pumped from the canal to utilize the surplus capacity.

The temporary dam elevation will be 9.00 C.D. The dam may impact the water level stages in Canal No. 2 east of the dam. The design flow at this location is anticipated to be less than 300 cubic feet per second during a ten year rain event. During the preliminary design phase, canal cross-sections east of the proposed temporary dam will be obtained. A detailed hydraulic analysis will be performed on this canal reach to asses the impacts of the proposed temporary dam on canal water surface profiles. The results of this analysis will be used to modify the features required to assure that the drainage east of the dam is not impaired. Detailed analysis may indicate that this sampling alternative may be feasible only for a short time period due to impacts on local drainage.

There are many suitable drainage pumps for this site. The recommended stormwater pump is an axial flow propeller pump, with a single electric drive unit operating the pump mechanism with flexible hydraulic hoses. The electric motor and hydraulic system would be installed on a 72" x 42" concrete slab on the canal bank. This unit is portable and can be moved to other locations and used for other purposes.

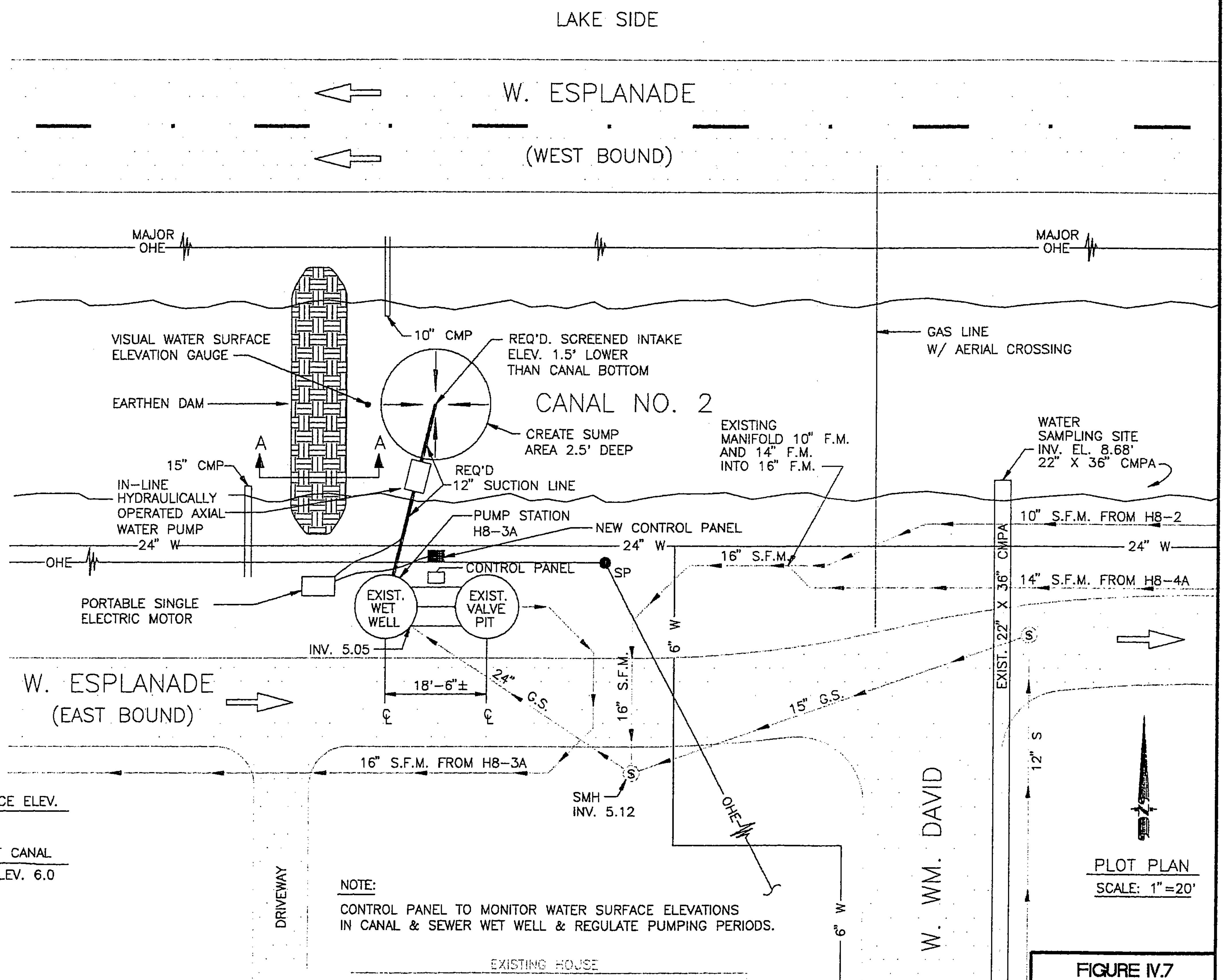
A 12" discharge would be installed into the wet well and would include a backflow preventer flap gate to prevent sanitary sewer from backflowing into the canal.

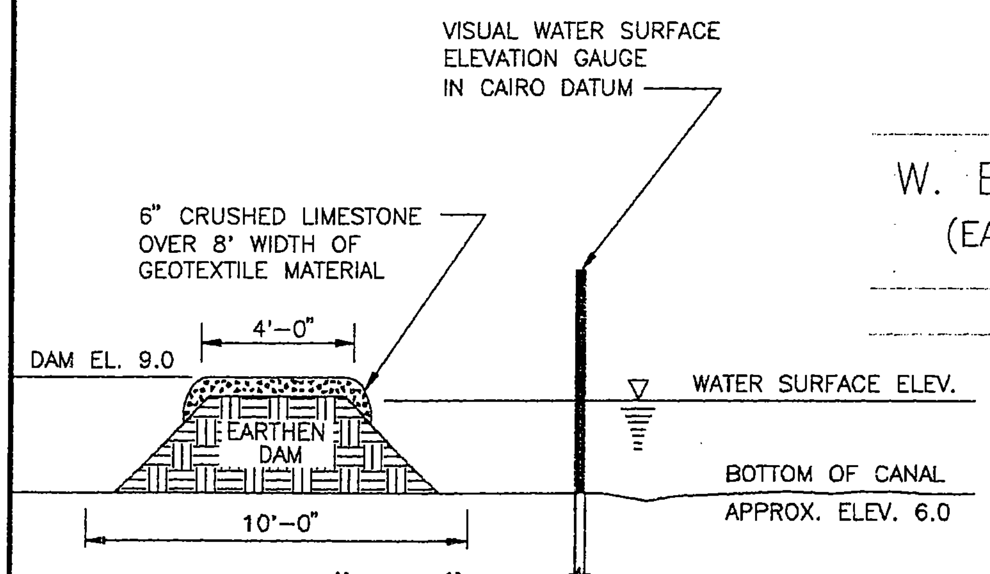
The stormwater pump will operate between the water surface elevations 6.00 C.D. (the approximate invert of the canal), at the low water stage, and 12.00 C.D. the proposed invert of the discharge at the high water stage.

At the elevation of the invert of the stormwater discharge line, a check valve will be activated to prevent flooding of the sanitary wet well by high waters in the canal. That is, if for some reason the canal water would be an elevation as high as the discharge line invert, water will be prevented from inundating the wet well through gravity flow.

EXISTING STATION HAS TELEMETRY

PUMPING STATION DATA TABLE				
PUMP STATION NUMBER				H8-3A
NUMBER OF PUMPS				3
TYPE OF DRIVE				C.S.
DESIGN CAPACITY PER PUMP,GPM				1560
TOTAL DYNAMIC HEAD, FT.				53
MIN. EFF. AT DESIGN CAPACITY,%				68
DESIGN SPEED, MAX. R.P.M.				1200
MAX. HORSEPOWER PER PUMP, H.P.				42
MIN. SHUT-OFF HEAD, FT.				67
MIN. SIZE SOLIDS, IN.				3
MIN. DISCHARGE SIZE, IN.				10
FLOW,GPM	1500	2000	2500	3000
HEAD,FT.	15.0	19.0	25.0	31.5





SECTION "A-A"
EARTHEN DAM

FIGURE IV.7

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JEFFERSON PARISH STORMWATER DISCHARGE DEMONSTRATION PROJECT

MULTIPLE SOURCE STATION PLAN

SECTION V
PROJECTED UPGRADES TO EAST BANK WWTP

A. INTRODUCTION

The East Bank Wastewater Treatment Plant (WWTP) is a 33.46 MGD activated sludge plant constructed in 1987 and commissioned in 1988. This plant processes domestic, commercial, and industrial flows together with the infiltration/inflow (I/I) generated in the collection system of East Jefferson Parish including the City of Harahan. A critical design parameter for the plant is its ability to handle peak hourly and peak daily flows well in excess of the average daily dry weather flow to the plant. The high peak hourly and peak daily flows are attributable mainly to the large I/I flows generated during wet weather periods. The ratio between peak daily flows and average daily dry weather flows have reached as high as 5:1 or higher. Based on past available influent data, the East Bank WWTP receives average daily flows (including wet weather flows) of approximately 35 MGD with peak daily flows of up to 145 MGD. Previous operating experience and preliminary analysis of the plant operations indicates that the design of the plant for excessive peak flows has resulted in excess capacity within the plant to treat average daily flows.

The purpose of the proposed stormwater treatment project is to transport mainly "first flush" stormwater to the East Bank WWTP for treatment prior to discharge to the Mississippi River. Thus, a critical factor in the stormwater treatment project is establishing the present capacity of the East Bank WWTP to treat additional stormwater flows and pollutant loadings. In addition, it must be established if the type of pollutants within the stormwater will affect or interfere with the normal operation of the plant's existing treatment units. An evaluation of the existing East Bank WWTP to handle the proposed stormwater flows along with necessary upgrades are discussed in this section.

B. PROJECTED QUALITY AND QUANTITY OF STORMWATER

B.1 QUALITY OF STORMWATER

The proposed construction and operation of a combined 22,000 gallon per minute stormwater pumping station and related force main to divert stormwater runoff to the East Bank WWTP will significantly increase the influent pollutant loadings at the plant. The 22,000 gpm includes 20,000 gpm from the new single source station and 2,000 gpm from the multiple source site. In order to evaluate the effect of these increased flows on the plant it was necessary to estimate the concentration of various pollutants in the stormwater that may be diverted to the plant.

The primary goal of the stormwater demonstration project is to capture and treat the initial "first flush" of stormwater received in the Jefferson Parish drainage canal system. The initial "first flush" of stormwater could be described as the first few hours of runoff collected in the canal system after the start of a rainfall event. For rainfall events of short duration and/or low intensity, the "first flush" could be represented by the total runoff produced by the storm event. In the event of longer duration and/or higher intensity storms the "first flush" would be represented by the first runoff collected in the storm drainage system after the start of the rainfall event. The reason for attempting to capture and treat the "first flush" of stormwater is that this initial flow of water has been found to have the highest concentration of pollutants. The "first flush" generally represents the first runoff from streets, parking areas, and industrial sites as well as the flushing of stagnant waters from within the subsurface as well as the surface drainage structures. Since the rainfall runoff provides a cleansing of these areas it is logical to assume that the "first flush" of runoff would remove the highest concentration of these pollutants.

To estimate the concentration of pollutants in the stormwater, results of wet weather field screening of the "first flush" from several storm events were reviewed. The field screening test data utilized was obtained from the May, 1993 NPDES Stormwater Permit Application prepared for Jefferson Parish by Montgomery Watson, Inc. The permit application included test results for five different canal locations with six storm events sampled for each location. "First Flush" data from additional sampling performed at the Elmwood Canal during the initial stages of a rainfall event, was also obtained and utilized.

In evaluating the test data only the pollutants which were detected in concentrations above the minimum quantification level for the test method in at least one runoff sample was included for consideration. After gathering the test data (which consisted of over 30 individual samples) and determining which pollutants were to be evaluated, a numeric average was determined for each of the pollutants of concern. Table V-1 lists each of the pollutants of concern and gives the numeric average for the quality of the stormwater runoff anticipated.

TABLE V-1 POLLUTANT CONCENTRATIONS OF "FIRST FLUSH" STORMWATER

POLLUTANT	ANTICIPATED CONCENTRATION (mg/l)
Biological Oxygen Demand (BOD_5)	18.33
Total Suspended Solids (TSS)	115.69
Ammonia (NH_3)	0.673
Cadmium (Cd)	0.002
Chromium (Cr)	0.007
Copper (CU)	0.035
Cyanide (CN)	0.144
Lead (Pb)	0.032
Mercury (Hg)	0.0002
Nickel (Ni)	0.012
Silver (Ag)	0.004
Zinc (Zn)	0.195
Phenol	0.008
Arsenic (As)	0.004
Beryllium (Be)	0.001
Selenium (Se)	0.0002
Chlorine	0.029
Oil and Grease (O&G)	18.822
Chemical Oxygen Demand (COD)	82.429
Thallium (TL)	0.001
Antimony (Sb)	0.003

The pollutant concentrations found in Table V-1 are indicative of wet weather flow or "first flush" data. If stormwater is to be pumped and treated on a steady basis at the East Bank WWTP, it is anticipated that a portion of this flow would be considered dry weather flow. The pollutant concentrations in dry weather flow could be anticipated to be lower than the values found in the "first flush" flow. Thus, the demands on East Bank WWTP capacity may also be lower. However, insufficient data is available at this time to establish dry weather flow concentrations. Thus, a conservative assumption is to utilize "first flush" data to establish the pollutant concentrations of all stormwater flows that will be diverted to the East Bank WWTP.

B.2 QUANTITY OF STORMWATER

The proposed construction and operation of an influent stormwater pumping station and associated force main to divert stormwater runoff to the East Bank WWTP would significantly increase the influent volume at this facility. The operation of this pump station would increase flow to the East Bank WWTP by 31.7 MGD if operated on a continuous basis. For rainfall events, of short duration and/or low intensity, the treatment of stormwater could continue for several days in order to treat additional stormwater stored within the existing canal system. However, for the case of longer duration and/or higher intensity storms, treatment would be limited to the time until increased wastewater flows caused by I/I would make treating the additional stormwater at the East Bank WWTP impractical.

A review of the hourly flow measurements taken at the East Bank WWTP and rainfall data gathered by the Jefferson Parish Geological Information System (GIS) Department revealed that rainfall events as small as one half of an inch have a large impact on the influent wastewater flows at the treatment plant. The impact of rainfall on plant flows are shown in Figures V.1 through V.4. These figures compare increases in flow to specific rainfall events ranging in quantities from 0.5 inches to several inches. The plant flows are displayed as "Increase in Plant Flows" and represent the increase in plant flow above average flow conditions for that time of the day. Changes in flows for storms of 0.54", 0.88", 1.24", and 2.48" are displayed. The figures indicate that even storm events of one half inch can double to triple plant influent flows due to I/I within a maximum of one hour of the rainfall event.

It is apparent the East Bank WWTP would not be capable of handling an additional 31.7 MGD of stormwater flow under the present I/I flow conditions experienced during rainfall events. More importantly, the system designed to convey the stormwater to the East Bank WWTP would be fully utilized to pump I/I flows and would not be able to also pump stormwater flows. Based on this information the feasibility of treating the "first flush" of stormwater with excess treatment capacity at the East Bank WWTP does not appear to be practical at all times.

In reviewing scenarios for delivering stormwater to the East Bank WWTP, the conclusion was reached that, since the capacity of the wastewater treatment plant is already taxed by the increase in wastewater flows caused by I/I, the ability of the plant to treat stormwater delivered to the plant on a continuous basis would be evaluated. This would represent the continuous pumping of canal water to the East Bank WWTP at the rate of 31.7 MGD during both dry and wet weather flows when allowed by flow conditions. In addition, preliminary operational plans include recharge of the drainage canals by backflushing the canals with waters from Lake Pontchartrain or treated effluent from the East Bank WWTP. Under this scenario it will be necessary for the East Bank WWTP to handle increased average daily flows. This scenario represents the most critical condition for successful operation of the plant facilities. Other scenarios, which represent isolated rainfall events, would allow delivery of the "first flush" flows to the plant simultaneously with I/I flows.

C. EFFECT OF STORMWATER ON TREATMENT PLANT OPERATION

C.1 GENERAL

The intent of pumping the stormwater to the East Bank WWTP is to remove pollutant concentrations from Lake Pontchartrain and treat the stormwater prior to discharge to the Mississippi River. However, depending on the concentration of pollutants within the stormwater, the proper operation of the plant may be upset by this additional loading. The increase in flows to the WWTP may require an increase in hydraulic capacity and treatment capacity which is discussed in Subsections E and F herein. However, even with an increase in hydraulic or treatment capacity the pollutants within the stormwater could still adversely affect plant performance. The three major ways the pollutant concentrations could affect the plant performance are exceedance of its permit limits due

FIGURE V.1: INCREASES IN EAST BANK WWTP FLOWS
March 27, 1994 - 0.54" Rainfall Event

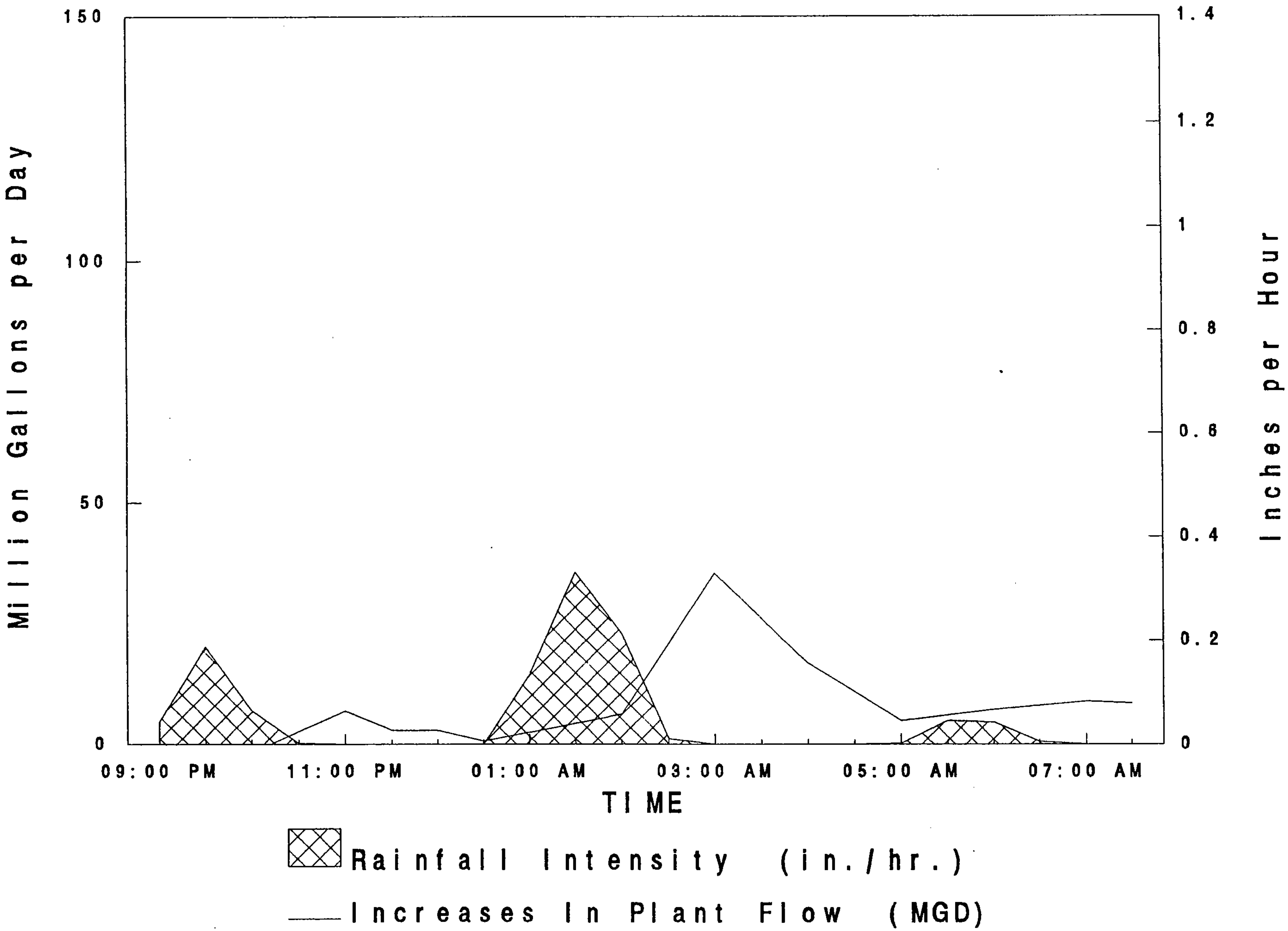


FIGURE V.2: INCREASES IN EAST BANK WWTP FLOWS
January 17, 1994 - 0.88" Rainfall Event

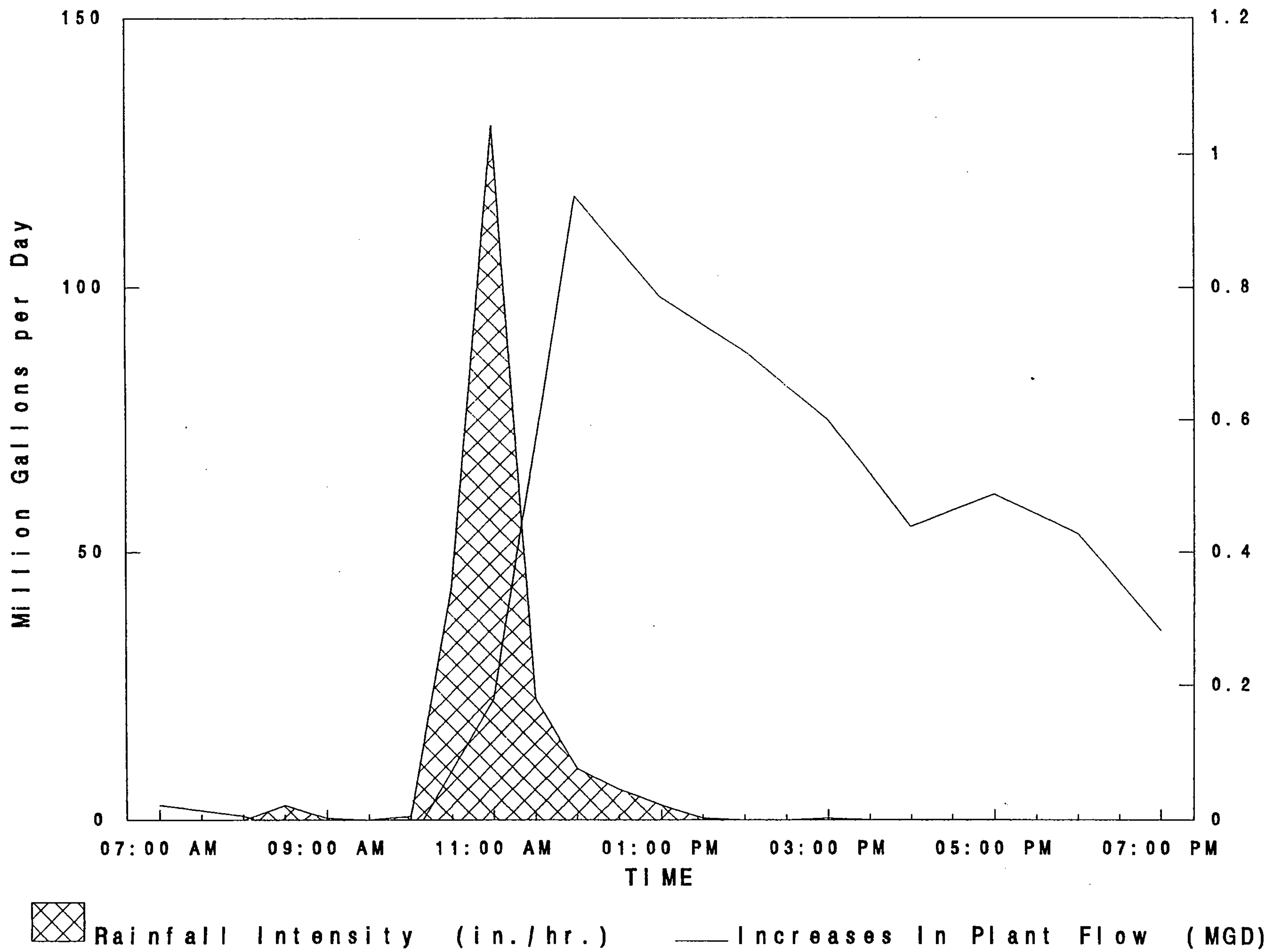


FIGURE V.3: INCREASES IN EAST BANK WWTP FLOWS
March 9, 1994 - 1.24" Rainfall Event

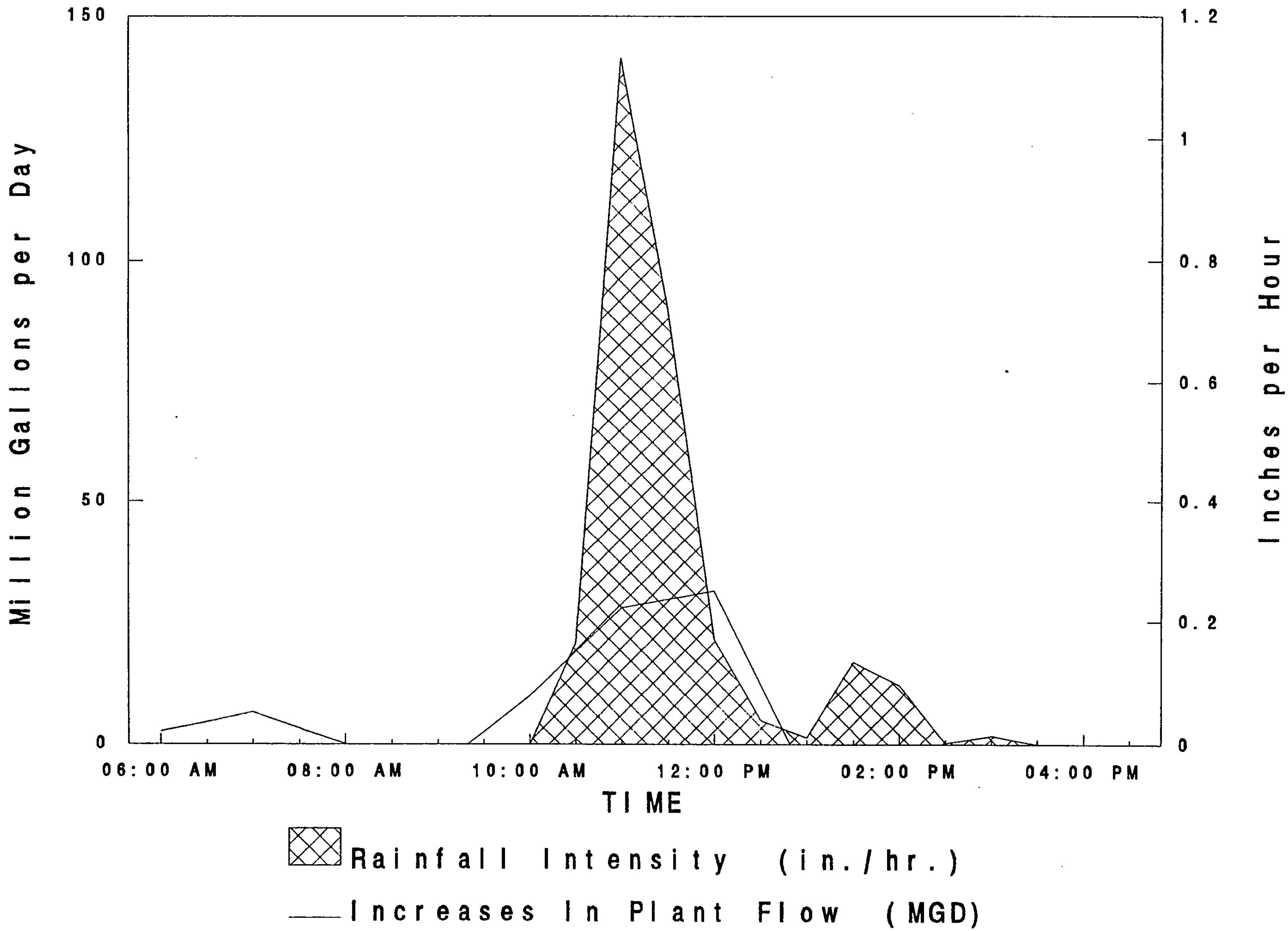
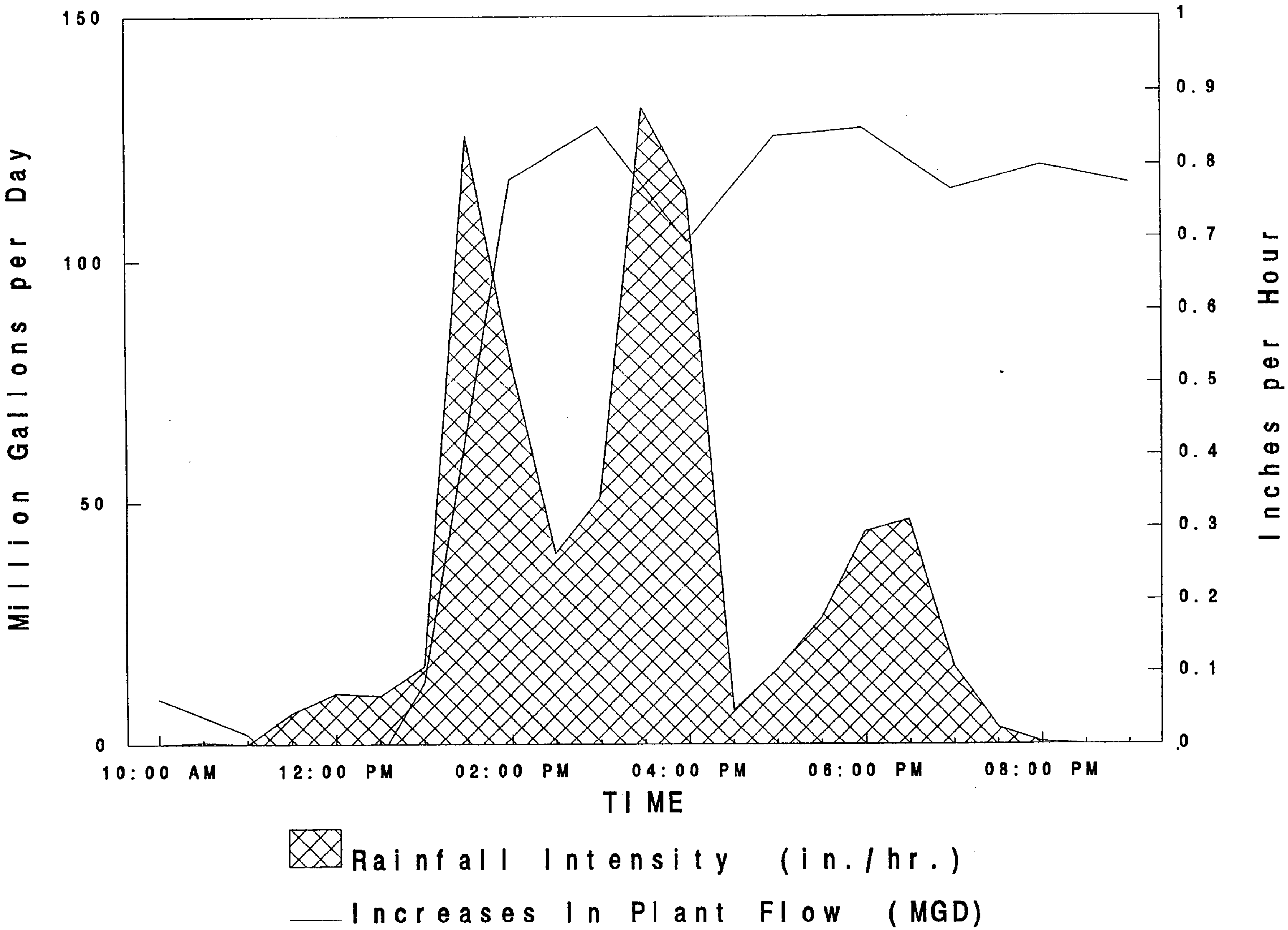


FIGURE V.4: INCREASES IN EAST BANK WWTP FLOWS
March 1, 1994 - 2.48" Rainfall Event



to the pass through of untreated pollutants, inhibition of biological treatment plant processes, and the capture and concentration of pollutants in the treatment plant's sewage sludge thus preventing its disposal or reuse.

In order to determine the effect of stormwater on the East Bank's WWTP performance the PRELIM Local Limits Computer Model Version 3.0 was utilized. This program was prepared by Science Applications International Corporation for the U.S. Environmental Protection Agency Office of Water Enforcement and Permits. The PRELIM (Pretreatment Limits) program was prepared to assist municipalities with the development of technically based local limits (TBLL's). The TBLL's are utilized to establish limits on the amount and concentration of pollutants which various industries can discharge into the WWTP collection system in order to assure the proper performance and operation of the WWTP. The proper performance of the plant is assured by back calculating allowable influent pollutant loadings based on inhibition, sludge quality, and effluent water quality criteria. Water quality criteria are utilized to assure that pass-through of pollutants do not result in toxic effluent. Inhibition criteria are utilized to assure that a concentration of a pollutant does not reach the plant which will interfere with its biological treatment processes. Sludge criteria is utilized to assure that the concentration of pollutants removed through clarification which become concentrated in the plant's sewage sludge do not affect the disposal or reuse options for the sludge. The allowable industrial pollutant loading is based upon the most restrictive of the preceding three criteria. The computations executed by the PRELIM computer program have been adopted from the local limits derivation procedures presented in Appendix L of EPA's 'Guidance Manual for POTW Pretreatment Program Development' (October, 1983).

The Prelim program allows the input of the following data:

Average Design Influent Flows for the WWTP
Total Industrial Flow to the WWTP
Total Domestic Flow to the WWTP
Sludge Flows To Digester/Disposal
Percent Solids Of Disposed Sludge
Influent Monitoring Data
Industrial User Monitoring Data
Domestic Monitoring Data
Treatment Unit Removal Efficiencies of Pollutants
Biological Process Inhibition Levels
Applicable Sludge Disposal And/or Water Quality Criteria/Standards

The Prelim program will utilize default values if data is not available for each of these items. For the evaluation of the stormwater's effect on the performance of the East Bank WWTP several sources of data were provided. The projected average daily flow (including wet weather flow) to the plant of 39.6 MGD was input as the domestic flow to the plant. The concentration of pollutants in this flow was obtained from influent monitoring data over the past three years. Therefore, all existing domestic sources and industrial sources to the plant were represented by this flow and pollutant concentrations. The quantity (31.7 MGD) and concentration of pollutants (Table V-1) for the stormwater to be treated were input as the sole industrial flow for the prelim program. Default values were utilized for the removal efficiencies of the primary treatment units since no data was available to calculate these removals. Secondary removal efficiencies were established by dividing influent mass loading data by effluent mass loading data (for the same day) for the plant for the past three years.

The biological treatment process inhibition levels utilized were default values provided by the Prelim program for activated sludge processes. The only exception was the value utilized for zinc of 0.45 mg/l due to the plant's past ability to handle flows with these zinc concentrations without upset. The water quality standards utilized were based on the Louisiana Department of Environmental Quality (LDEQ) Water Quality Standards (LAC 33:IX.1113). The most conservative of the acute and chronic criteria for each pollutant were utilized. (A 1 to 10 dilution of effluent is assumed for acute criteria and a 1 to 100 dilution of effluent is assumed for chronic criteria). Water Quality Standards for the parameters of BOD_5 and TSS were based on the plant's NPDES permit limits of 30 mg/l and 30 mg/l, respectively. Criteria utilized for sludge disposal were based on the allowable sludge pollutant concentrations for land disposal as defined by the Toxicity Characteristic Leaching Procedure. Criteria for sludge reuse were based on pollutant concentrations as set forth in CFR 503.

C.2 EFFLUENT QUALITY

The concentration of stormwater pollutants input into the Prelim program are displayed in Table V-1. It was assumed these values would be conservative since they have been considered "first flush" values. The first Prelim run (Appendix B) utilized the Water Quality Criteria shown in Table V-2 and the TCLP Sludge Criteria shown in Table V-3 and inhibition values as previously mentioned.

TABLE V-2 WATER QUALITY CRITERIA FOR EAST BANK WWTP EFFLUENT **(MISSISSIPPI RIVER)**

POLLUTANT	CRITERIA (MG/L)
Biological Oxygen Demand	30.00
Total Suspended Solids	30.00
Ammonia	6.80
Arsenic	0.050
Beryllium	0.053
Cadmium	0.010
Chromium	0.016
Copper	0.019
Cyanide	0.022
Lead	0.032
Mercury	0.000012
Nickel	1.40
Phenol	0.005
Selenium	0.260
Silver	0.001
Zinc	0.120

As shown on page B-6 of this Prelim run in the table labelled "Comparison of Headworks Loading Limits" the Pass-Through or Water Quality limits are the most stringent (allow the lowest headworks loading) of the three influent criteria for ammonia, chromium, copper, phenol, and beryllium. A comparison of Actual Industrial Pounds/Day (stormwater pollutant loading) to Allowable Industrial Pounds/Day (additional pollutant loading the plant can effectively treat) indicates the Actual Industrial Pounds/Day are an order of magnitude less than the Allowable for these pollutants (see page B-7 of Appendix B). Thus, the treatment of 31.7 MGD of stormwater will not cause the plant to exceed or even approach the effluent quality criteria for these pollutants. For all the remaining pollutants, sludge criteria and inhibition were the critical limits for plant performance. Again, the actual industrial loading was significantly lower than the allowable for the remaining pollutants. Thus, based on the above mentioned rationale, the treatment of 31.7 MGD of stormwater will not result in exceedance of sludge criteria and effluent quality criteria.

C.3 INHIBITION OF TREATMENT PROCESSES

The Prelim program with the input parameters as described previously and as shown in Appendix B indicated that the East Bank WWTP plant would not suffer from biological process inhibition problems due to the treatment of stormwater. As shown on page B-6 of this Prelim run, the inhibition limits are the most stringent of the three influent criteria for cyanide, nickel, and zinc. On page B-7 of Appendix B, a Comparison Of Actual Industrial Loadings (stormwater pollutant loadings) to Allowable Industrial Loadings indicates that the actual loadings are one third or less of the allowable loadings for these pollutants. For all the remaining pollutants, sludge criteria and water quality criteria were the critical limits for plant performance. Again, the actual industrial loadings were significantly lower than the allowable loadings for these remaining pollutants.

An additional Prelim program was also run to determine if the conversion of the primary clarifiers to additional aeration basins would result in inhibition of the activated sludge process. The removal of primary clarification from the Prelim program would result in increasing pollutant concentrations to the activated sludge process. This in turn would result in a greater potential for inhibition to occur in this process. This was achieved by assuming 0% primary removal within the Prelim program while inputting all other parameters as in Prelim Run #1 (Appendix B). The results of Prelim Run #2 are shown in Appendix C. As shown on page C-6 of this Prelim run, the inhibition limits are the most stringent of the three influent criteria for arsenic, cyanide, lead, nickel, and zinc. On page C-7 of Appendix C, a comparison of actual industrial loadings (stormwater pollutant loadings) to allowable industrial loadings indicates that the actual loadings are one quarter or less of the allowable loadings for these pollutants. The only exception is for cyanide with an actual loading of approximately one half of the allowable loading. For all the remaining pollutants, sludge criteria and water quality criteria were the critical limits for plant performance. Again, the actual industrial loading was significantly lower than the allowable for these remaining pollutants. This indicates that, even without the benefit of primary clarification for pollutant removal, the treatment of 31.7 MGD of stormwater would not cause inhibition problems due to increased pollutant loadings.

C.4 FOOD/MICROORGANISM RATIO

The efficient operation of the activated sludge process is dependent upon certain quantities of BOD_5 within the influent stream to the plant. If the concentration and loading of this pollutant in the influent becomes too low or too high the activated sludge process will not function properly. The activated sludge process utilizes micro-organisms to breakdown or reduce the incoming BOD_5 and other pollutants to carbon dioxide, water, and new micro-organisms or biomass. In the activated sludge process, the ability of the organisms to break down BOD_5 is dependent upon the food to micro-organism ratio. The food to micro-organism (*f/m*) ratio is defined as follows:

$$f/m = \text{Organic loading rate/ Volume of sludge x Concentration of sludge}$$
$$= (\text{Influent Flow x } BOD_5)/\text{ Volume of sludge x X}$$

Where X = Mixed liquid suspended solids(MLSS) in the aeration tank(mg/l)

The *f/m* ratio range for proper operation of a conventional activated sludge process is 0.2 to 0.4 [Design parameters for activated sludge processes, MOP, Vol 1, ed. 8 WEF]. When the *f/m* ratio is higher than 0.4 the micro-organism are in the exponential growth phase. With excess food the rate of metabolism is at a maximum with large BOD_5 removal achieved. However, under these conditions the micro-organisms do not flocculate upon removal from the aeration basins but are generally dispersed and do not readily settle through secondary clarification. In addition, since food is in excess, not all the organic material will be utilized so the remainder will pass out in the final effluent. This condition can result in high TSS and BOD_5 concentrations in the plant effluent most likely in excess of effluent limits.

Low *f/m* ratios put the micro-organisms into a food -limited environment. The micro-organisms may first experience a high rate of metabolism when first encountering an incoming wastewater with a low BOD_5 content, but once food becomes limited the rate of micro-organism metabolism will rapidly decline until the micro-organisms are in an endogenous respiration phase which results in cell lysis and resynthesis. This will cause bulking to occur. The micro-organisms will no longer flocculate upon removal from the aeration basin and will not settle through secondary clarification. This condition will adversely affect the operational efficiency of the activated sludge system, resulting in exceedance of its BOD_5 and TSS discharge limits.

It is anticipated that the addition of stormwater to the East Bank WWTP may result in conditions where the *f/m* ratio in the activated sludge processes will be below typical design parameters. Stormwater flows of 31.7 MGD with a first flush BOD_5 concentration of 18 ppm combined with 39.6 MGD wastewater will result in low BOD_5 loading and hence a *f/m* ratio to the aeration basin of 0.28-0.30 day^{-1}. This is within the operational range, however, during the early morning hours when the wastewater flow will be very low (15-20 MGD) the addition of 31.7 MGD of stormwater will result in low *f/m* ratios of 0.12-0.17 day^{-1}, well below the acceptable operational range. If additional aeration basin volume is added as proposed in subsection F, the ratio was found to decrease further. This would result in *f/m* ratios of 0.20 to 0.27 day^{-1} and 0.10 to 0.14 day^{-1} for wastewater flows of 39.6 MGD and 15 MGD, respectively, and a stormwater flow of 31.7 MGD. In order to maintain the *f/m* ratio within an acceptable operational range during low wastewater and high stormwater flows the provision of additional BOD_5 loadings to the East Bank WWTP must be pursued. Options to solve this problem are discussed in Subsection E below.

C.5 SALT CONTENT OF STORMWATER

The proposed continuous pumping of water from the drainage canals to the East Bank WWTP may cause drawdown of these canals during dry weather periods. To allow pumping to continue and to further flush the canals, it has been proposed to allow water from Lake Pontchartrain to flow into the canals. If this is allowed to occur, water with an elevated salt content may enter the canals and may eventually reach the East Bank WWTP. Water with a high salt content may adversely effect the operation of the plant.

Salinity can exhibit two main effects on wastewater treatment processes. An increase in salinity causes a depression in the oxygen content of water. Also, high salinity waters can be toxic to the microorganisms utilized in the activated sludge process. The depression in oxygen levels in stormwater proposed to be pumped to the plant is expected to be insignificant. Literature indicates that at 25°C and a salt concentration of 10,000 ppm, the oxygen saturation value is reduced to 7.56 ppm. In general, DO values at Lake Pontchartrain are typically around 8 ppm. These DO levels should not inhibit the biological activity of the wastewater treatment processes at the East Bank WWTP. Furthermore, any decrease in the overall DO would be more than compensated by the aeration system provided in the plant's activated sludge aeration basins.

The mean salinity of the Lake Pontchartrain is approximately 4 parts per thousand measured at the water quality measurement stations provided near the Lake Pontchartrain Causeway. There are three active water quality stations maintained by Louisiana Department of Environmental Quality LDEQ near this location. Monthly sampling at the three stations was initiated in January, 1994. The toxic effect of salinity on the activated sludge process utilized at the East Bank WWTP is dependent upon the salt content of the incoming salt water. It is very difficult to predict the effect of salt contents on the activated sludge process due to the scarcity of related literature. Even if adequate literature on the toxic effect of salt water on the activated sludge process was available it is difficult to predict at this time, the exact salt content which would be experienced at the East Bank WWTP. Although the salt content of the water allowed to flow into the drainage system is expected to be in the 4 parts per thousand range, other operational parameters must be considered. These considerations include a determination of how often inflow of lake water would incur, and the degree to which it would be diluted by stormwater and wastewater. Also, the sensitivity of activated sludge systems to the toxic effects of salt content can vary widely between activated sludge systems.

At this time, it is difficult to determine how the East Bank WWTP would be effected by salt water from Lake Pontchartrain reaching the plant through the stormwater pumping system. However, prior to designing the proposed stormwater treatment system for the Parish, it is recommended that pilot scale studies be performed on the activated sludge system to analyze its sensitivity to various salt concentrations including a maximum of at least 4 parts per thousand. Flow modeling to establish the maximum salt concentration expected may also be necessary.

D. DISPOSAL AND REUSE OF SEWAGE SLUDGE

D.1 INTRODUCTION

The treatment of stormwater will result in some pollutants from the stormwater entering the plant's sewage sludge. Dependent on the pollutant concentrations in the stormwater, the resulting pollutant concentrations in the sewage sludge may affect the method by which the Parish can dispose or reutilize its sludge. At present, the Parish is landfilling the sewage sludge from the East Bank WWTP. However, due to recent increases in disposal costs, sludge reuse may become more cost effective in the near future.

Thus, if stormwater treatment at the East Bank WWTP affects sludge quality it may affect sludge disposal or reuse options thereby increasing the Parish's costs for sludge handling.

D.2 EXISTING SLUDGE DISPOSAL PRACTICES

Jefferson Parish's East Bank WWTP presently disposes of its sewage sludge at the Kelven Landfill in Waggaman, Louisiana. The Kelven Landfill is owned and operated by Jefferson Parish and is permitted to receive municipal solid waste and sewage sludge. In 1988, the Parish performed a "Sludge Disposal Alternative Study" that indicated that landfill disposal was the most cost effective method of disposal for the sludge produced by the East Bank WWTP as well as the three other Jefferson Parish WWTP. However, the cost of sludge disposal in 1993 was $5.17/ton and has increased to $12.65/ton in 1994. Future sludge disposal costs are expected to exceed $20.00/ton within the next two years. (See D.3.a below)

The only present restrictions on landfill disposal of sewage sludge is that the material must pass the Paint Filter Test and must be tested by the Toxicity Characteristic Leaching Procedure (TCLP). The Paint Filter Test establishes if the sludge contains free liquid. If it does, it can not be landfilled. The TCLP establishes whether a waste material is to be considered a hazardous waste or nonhazardous waste affecting the disposal method utilized. To date the sewage sludge from all of Jefferson Parish's four major WWTPs has consistently passed these tests allowing it to be disposed of as a nonhazardous solid waste. Designation as a hazardous waste would greatly increase sludge disposal costs.

D.3 FUTURE SLUDGE DISPOSAL AND REUSE OPTIONS

D.3.a Land Disposal

The East Bank WWTP can continue to landfill its sludge at the Kelven landfill as long as the landfill remains open and properly permitted or until regulatory changes forbid the disposal of sewage sludge in landfills. The existing landfill is anticipated to remain properly permitted and open for an additional twenty years. This is dependent on a 270 acre expansion of the landfill being permitted under existing Subtitle D regulations. The present landfill phases will be filled within approximately one and one half years. The permit for the 270 acre Phase III expansion is anticipated to be granted. However, the newly implemented design requirements for Subtitle D landfills are also anticipated to substantially increase landfill disposal costs. Thus, in the future, landfill disposal costs could become prohibitive and alternative disposal or reuse methods for sewage sludge may be considered.

In order to continue land disposal of the East Bank sewage sludge as solid waste it must continue to pass the Paint Filter Test and the TCLP. The treatment of stormwater should not effect the ability of dewatered sludge to pass the Paint Filter Test. However, the increase in total pollutant loading to the East Bank WWTP due to stormwater treatment may increase pollutant concentrations in the sewage sludge and effect its ability to pass the TCLP.

The ability of sewage sludge to pass the TCLP with the addition of stormwater flows to the East Bank WWTP was assessed through the use of the Prelim program. This was accomplished by inputting as sludge criteria in Prelim Run #1 (Appendix B) twenty times the maximum contaminant concentrations that define a waste as hazardous according to the TCLP. The maximum contaminant concentrations were multiplied by twenty since the TCLP dilutes solid wastes by a factor of twenty with leaching solution during the testing procedure and prior to contaminant analysis. The concentrations utilized in the Prelim Run #1 are shown in Table V-3. (The TCLP tests for 40 pollutants, only the compounds anticipated in the wastewater or stormwater to be treated are listed in Table V-3).

The results of this Prelim Run#1 are shown in Appendix B. As shown on page B-6 of this Prelim run, the sludge criteria are the most stringent of the three influent criteria for arsenic, cadmium, lead, mercury, silver, and selenium. On page B-7 of Appendix B comparison of actual industrial loadings (stormwater pollutant loadings) to allowable industrial loadings indicates that the actual loadings are an order of magnitude less the allowable loadings for these pollutants. For all the remaining pollutants, inhibition and water quality criteria were the critical limits for plant performance. The actual industrial loadings were significantly lower than the allowable for these remaining pollutants. This indicates the stormwater treatment will not result in production of sewage sludge which will fail the TCLP and must be disposed as hazardous waste.

TABLE V-3 : TCLP SEWAGE SLUDGE POLLUTANT CONCENTRATION LIMITATIONS

POLLUTANT	TCLP CONCENTRATION LIMIT (mg/l)	PRELIM INPUT (mg/l)
Arsenic	5.0	100.0
Cadmium	1.0	20.0
Chromium	5.0	100.0
Lead	5.0	100.0
Mercury	0.2	4.0
Selenium	1.0	20.0
Silver	5.0	100.0

D.3.b Land Application

If land disposal of sewage sludge becomes cost prohibitive in the future land application or beneficial reuse of sewage sludge may become the Parish's most cost effective alternative. In order to land apply or reuse sewage sludge, some combination of the requirements outlined in 40 CFR 503, Subpart B must be met. The regulations establish three criteria which must be met to allow land application or beneficial reuse of sewage sludge. These criteria include pollutant concentration limits, pathogen reduction requirements and vector attraction reduction requirements.

Methods to meet pathogen and vector attraction requirements typically involve methods to reduce the sludge's pathogen concentration (typically measured as fecal coliform) and percent volatile solids. A typical method to achieve these reductions is aerobic or anaerobic digestion.
To qualify for land application or beneficial reuse the pollutants in the sludge must meet specific pollutant concentrations (Tables 1 and 3 of Subpart B; Tables

V-4 and V-5, respectively): To be land applied under any basis the sludge must first meet the concentrations in Table 1 of Subpart B. To be potentially land applied without any site restrictions or as a "exceptional quality sludge" the sludge must meet the requirements in Table 3 of Subpart 3.

TABLE V-4 : 40 CFR PART 503, SUBPART B
TABLE 1 ALLOWABLE SEWAGE SLUDGE
POLLUTANT CEILING CONCENTRATIONS

POLLUTANT	CEILING CONCENTRATION (mg/kg)
Arsenic	75
Cadmium	85
Chromium	3000
Copper	4300
Lead	840
Mercury	57
Molybdenum	75
Nickel	420
Selenium	100
Zinc	7500

The sludge produced by the East Bank WWTP is tested for all the pollutants in Tables 1, 2, and 3 of the Subpart B regulations except Molybdenum. Quarterly metals data for 1991, 1992, and 1993 for the sludge produced from the East Bank WWTP indicated that only once during the three years (1991) did the sludge not meet all of the Table 1 and Table 3 requirements (excluding Molybdenum). In July of 1991, the East Bank plant exceeded the Table 3 limit for Lead. Therefore, to meet the pollutant requirements, Jefferson Parish may have to monitor for lead more frequently as well as add molybdenum to its quarterly analysis.

In addition to the pollutant requirements, pathogen percent reduction and vector attraction reduction requirements must be met. However, it is probable for the sludge from the East Bank WWTP to meet the pathogen reduction or vector attraction reduction requirements, sludge digesting or other sludge stabilizing equipment would have to be installed at the plant.

TABLE V-5: 40 CFR PART 503, SUBPART B TABLE 3 ALLOWABLE SEWAGE SLUDGE POLLUTANT CONCENTRATIONS

POLLUTANT	MONTHLY AVERAGE CONCENTRATIONS(mg/kg)
Arsenic	41
Cadmium	39
Chromium	1200
Copper	1500
Lead	300
Mercury	17
Molybdenum	18
Nickel	420
Selenium	36
Zinc	2800

The treatment of stormwater may affect the ability of the sludge to be land applied or beneficially reused in the future due to potential increases in contaminant concentrations. To determine if the treatment of stormwater at the East Bank WWTP would have this affect on the sewage sludge the parameters shown in Table V-5 or Table 3 above were input into the Prelim Run#3 as the sludge criteria.

The results of Prelim Run #3 are shown in Appendix D. As shown on page D-6 of this Prelim run the sludge criteria are the most stringent of the three influent criteria for arsenic, cadmium, copper, lead, nickel, and selenium. On page D-7 of Appendix D comparison of actual industrial loadings (stormwater pollutant loadings) to allowable industrial loadings indicates that the actual loadings are at least an order of magnitude less than the allowable loadings for these pollutants. For all the remaining pollutants, inhibition and water quality criteria were the critical limits for plant performance. The actual industrial loadings were significantly lower than the allowable loadings for these remaining pollutants. This indicates that stormwater treatment at the plant will not cause the sewage sludge to exceed contaminant concentrations which would prevent the future potential use of sewage sludge for land application or beneficial reuse.

D.3.c Surface Disposal

Surface disposal units (sludge lagoons, monofils) have different sludge requirements based on the presence or absence of a liner and leachate system. If a surface disposal site uses a liner and a leachate collection system, there are no pollutant concentration limits. It is assumed that pollutants leaching from the solids will be collected in the leachate and treated accordingly. Liners are required to meet specific hydraulic conductivity requirements.

If the disposal site does not have a liner and leachate collection system, the sludge must meet pollutant concentration levels for three pollutants, arsenic, chromium, and nickel. The levels vary with distance of the active unit's boundary from the property line of the site. Based on the most stringent concentration limits for the three pollutants (30, 200, and 210 mg/kg, for arsenic, chromium, and nickel respectively), the East Bank WWTP has met the pollutant limits since 1991.

It is not anticipated that the Parish will pursue this option in the future for sewage sludge disposal. However, the Parish has operated sludge lagoons in the past, and the potential for sewage sludge to be disposed in this manner does exist. Under a worst case scenario (an unlined disposal cell, without leachate collection and within 25 meters of a property line) the sewage sludge would be expected to meet arsenic, chromium, and nickel concentration limits of 30, 200, and 210 mg/kg, respectively to be surface disposed. The previous Prelim runs to evaluate land disposal and land application options indicate stormwater treatment should not produce sewage sludge which exceeds these concentration levels. If it does, surface disposal still can still be utilized if a liner and leachate collection system is provided for the disposal cell.

D.3.d INCINERATION

Incineration is an option for Jefferson Parish in terms of sludge disposal. Pollutant limits as well as emission standards for specific substances must be met to utilize this disposal method. Emission limits on the incineration of sewage sludge for beryllium and mercury are 10 grams per 24 hours and 3200 grams per 24 hours, respectively (40 CFR 61). Limits for the other pollutants, lead, arsenic, cadmium, chromium, and nickel, are site specific limits based on the performance characteristics of the incinerator, emission dispersion modeling, and sludge feed rates. Sewage sludge incinerators must also meet a monthly average limit for total hydrocarbon emission of 100 ppm. The monthly limit is corrected for zero percent moisture and an oxygen content of seven percent. The equation for calculating the limit is located in 40 CFR 503, Subpart E. The limit is intended as an indicator of the control toxic organic compound emissions.

Since all but two of the pollutant limits are based on site specific conditions and the total hydrocarbon limit is established by emissions testing, Jefferson Parish would have to test the East Bank sewage sludge in a permitted sewage sludge incinerator to determine if the sludge is suitable for this alternative disposal method. Since the Parish has no operating experience with this sludge disposal method, it is difficult to determine how changes if any in sludge quality would affect this disposal option. Also, it is anticipated that incineration costs will exceed future land disposal costs precluding incineration as a viable disposal option.

E. EVALUATION OF EXISTING TREATMENT CAPACITY

E.1 EXISTING FLOWS AND POLLUTANT CONCENTRATIONS

Based upon influent flow data measured for the East Bank WWTP for the past five years the average flow to the plant has been 35.3 MGD (Figure V.5). The average flow for the last two years (January, 1992 through February, 1994) has been 35.5 MGD (Figure V.6). The average BOD_5 and TSS concentrations to the East Bank WWTP for January, 1989 through February, 1994 has been 122 mg/l and 130 mg/l, respectively. It should be noted both these values are below the typical BOD_5 and TSS concentrations anticipated for a domestic wastewater stream of 200 mg/l and 200 mg/l, respectively. The main reason for this occurrence is the dilution of wastewater flows to the East Bank WWTP due to I/I flows. The initial design of the East Bank WWTP was based upon flows of 33.46 MGD and a BOD_5 and TSS concentrations of 180 mg/l. The corresponding BOD_5 and TSS loadings utilized during plant design were 38,000 lbs/day. The present conditions at the plant indicate that average flows to the plant have increased above 33.46 MGD while the concentration of BOD_5 and TSS to the plant have decreased. Even BOD_5 and TSS concentrations representative of dry weather flows (less than 35 MGD), exhibit low values of 133 mg/l and 144 mg/l, respectively. At the same time, the BOD_5 and TSS loadings to the plant have remained close to the initial design parameters. The average BOD_5 and TSS loadings for the last two years (January 1992 through December 1993) were 33,400 lbs/day and 36,000 lbs/day, respectively.

E.2 REVIEW OF UNIT OPERATIONS

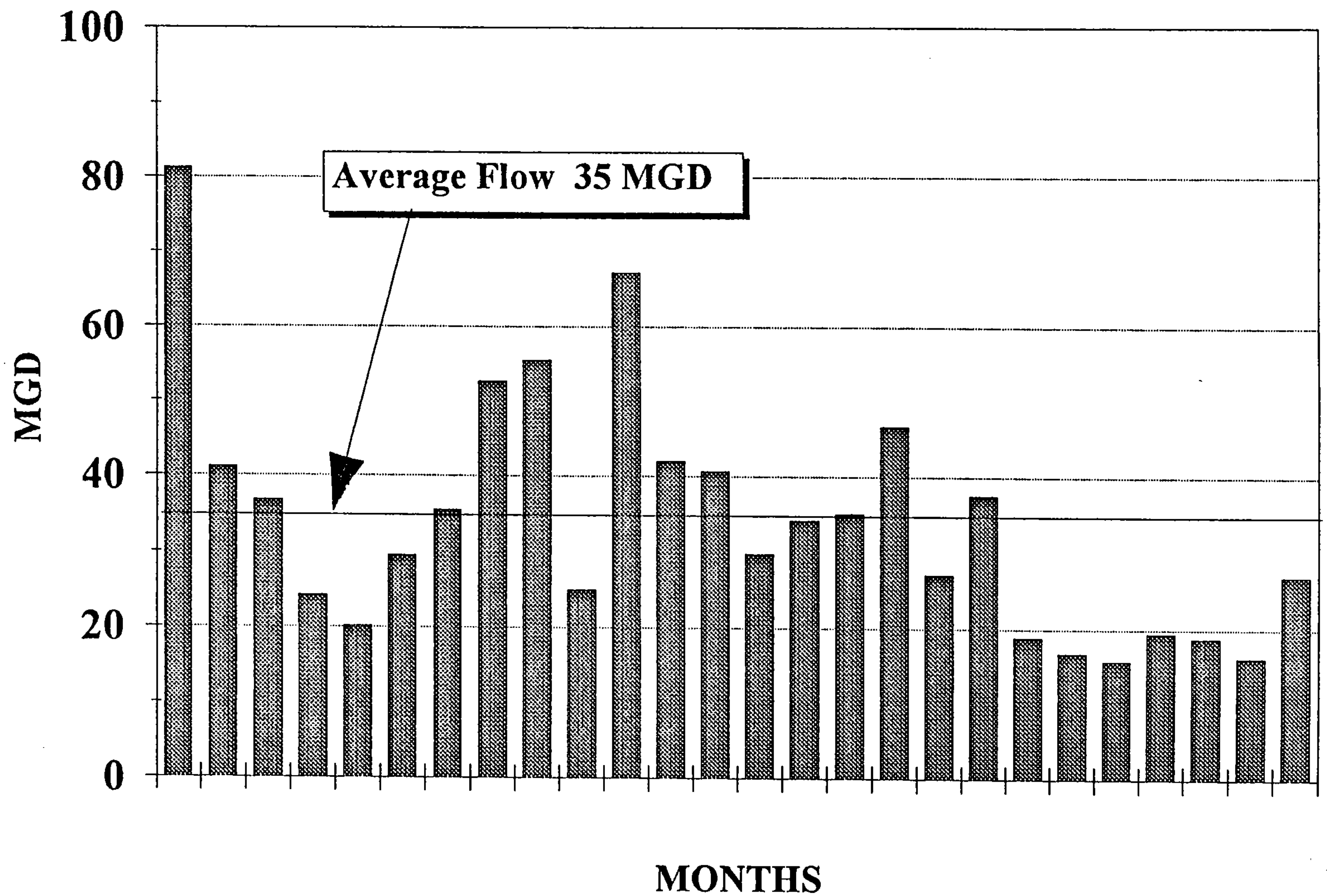
The unit operations include preliminary treatment including screening and grit removal by two bar screens and two aerated grit chambers, followed by six primary clarifiers. Secondary treatment consists of four conventional activated sludge aeration basins followed by eight circular clarifiers. Disinfection is accomplished by applying chlorine to the secondary clarifier effluent channel upstream of the effluent pump station wet well.

Treatment of the solids generated by the liquid phase treatment train consists of dissolved air flotation thickening of the activated sludge, transfer of sludge to sludge holding tanks, dewatering of the sludge utilizing belt filter presses, and disposal of the dewatered sludge at the Kelven Landfill. A flow schematic and site layout of the existing treatment train is shown in Figures V.7 and V.8, respectively.

Odor control facilities are also provided at the following locations: Headworks (screening and grit chamber), primary clarifier, dissolved air flotation, and filter press building. All channels, grit chambers, and primary clarifiers are covered and vented through an HOCL scrubber unit in order to prevent the emission of odors. The dissolved air flotation and filter press building are also enclosed and vented through an HOCL scrubber unit to prevent emission of odors. The East Bank WWTP also has the capacity to inject chlorine into the wastewater at several locations within the plant to provide additional odor control. These locations include the headworks, primary clarifier, and dissolved air flotation units. The equipment to provide chlorine injection at these locations has been installed but has not been operated since the construction of the plant.

Figure V.5 : INFLUENT FLOW EBWWTP

JANUARY 92 TO FEBRUARY 94 (26 MONTHS)



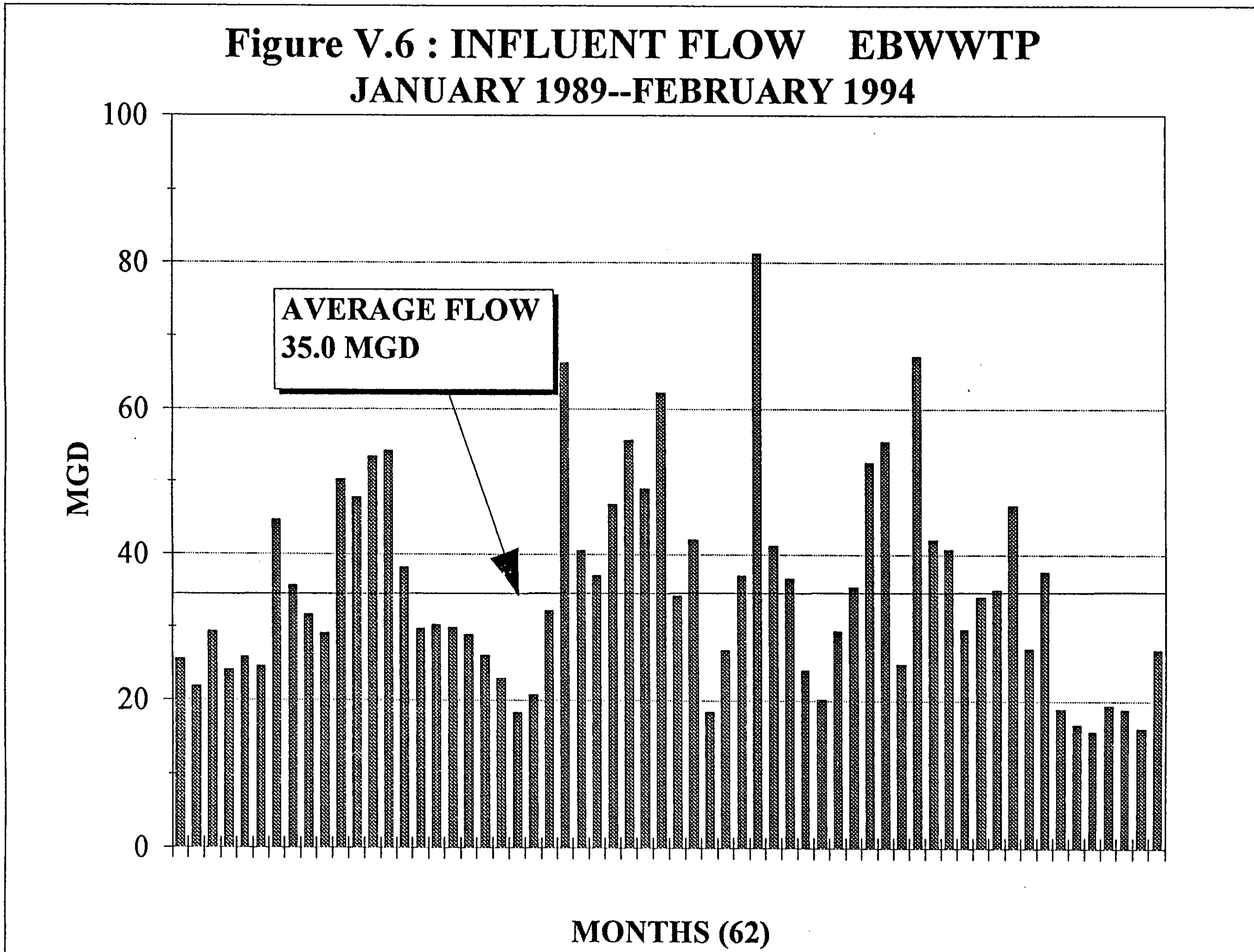


Figure V.6 : INFLUENT FLOW EBWWTP
JANUARY 1989--FEBRUARY 1994
100
80
60
40
20
0
MGD
AVERAGE FLOW
35.0 MGD
MONTHS (62)

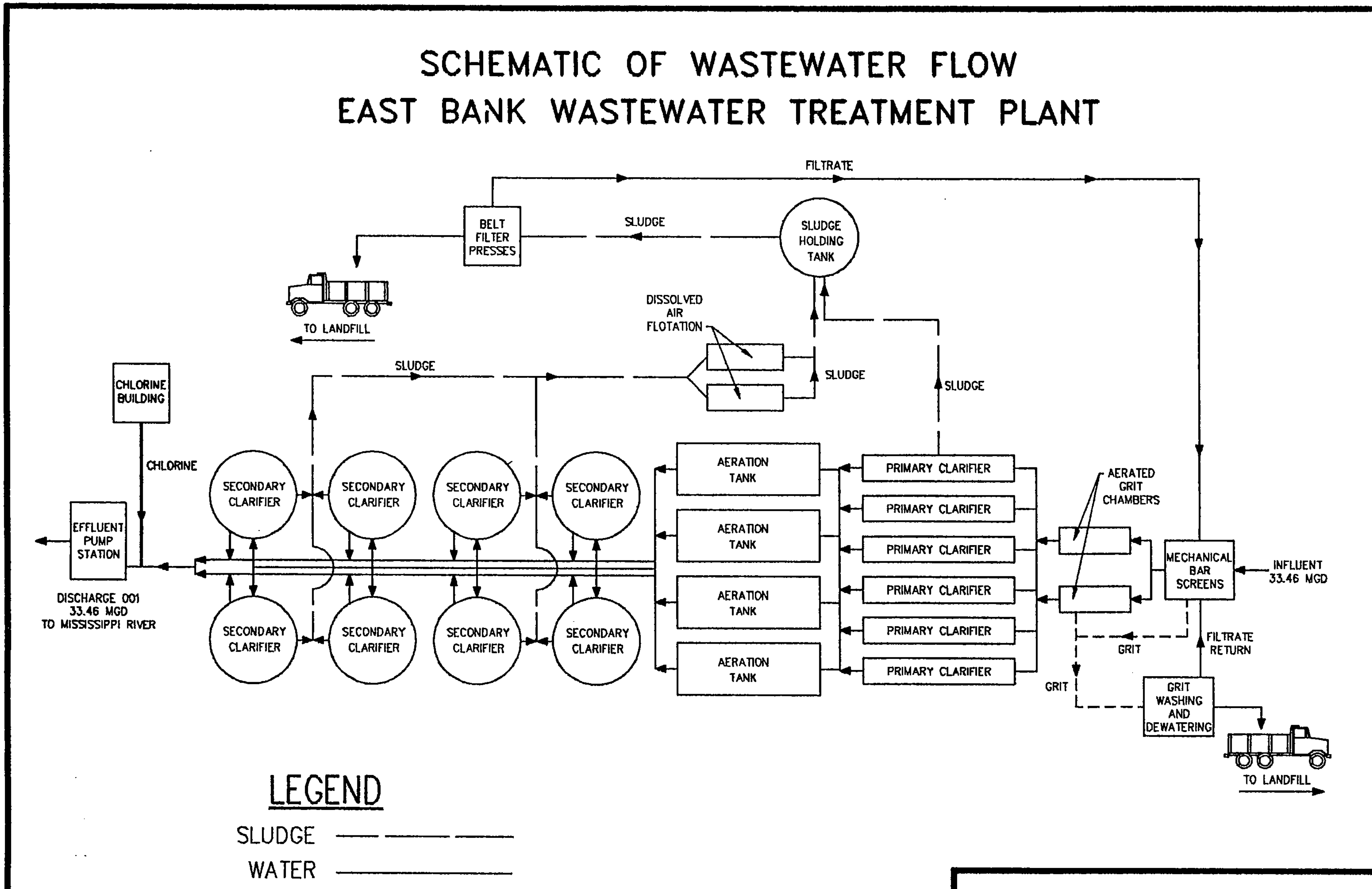


FIGURE V.7

EAST BANK WWTP

SCHEMATIC OF WASTEWATER FLOW

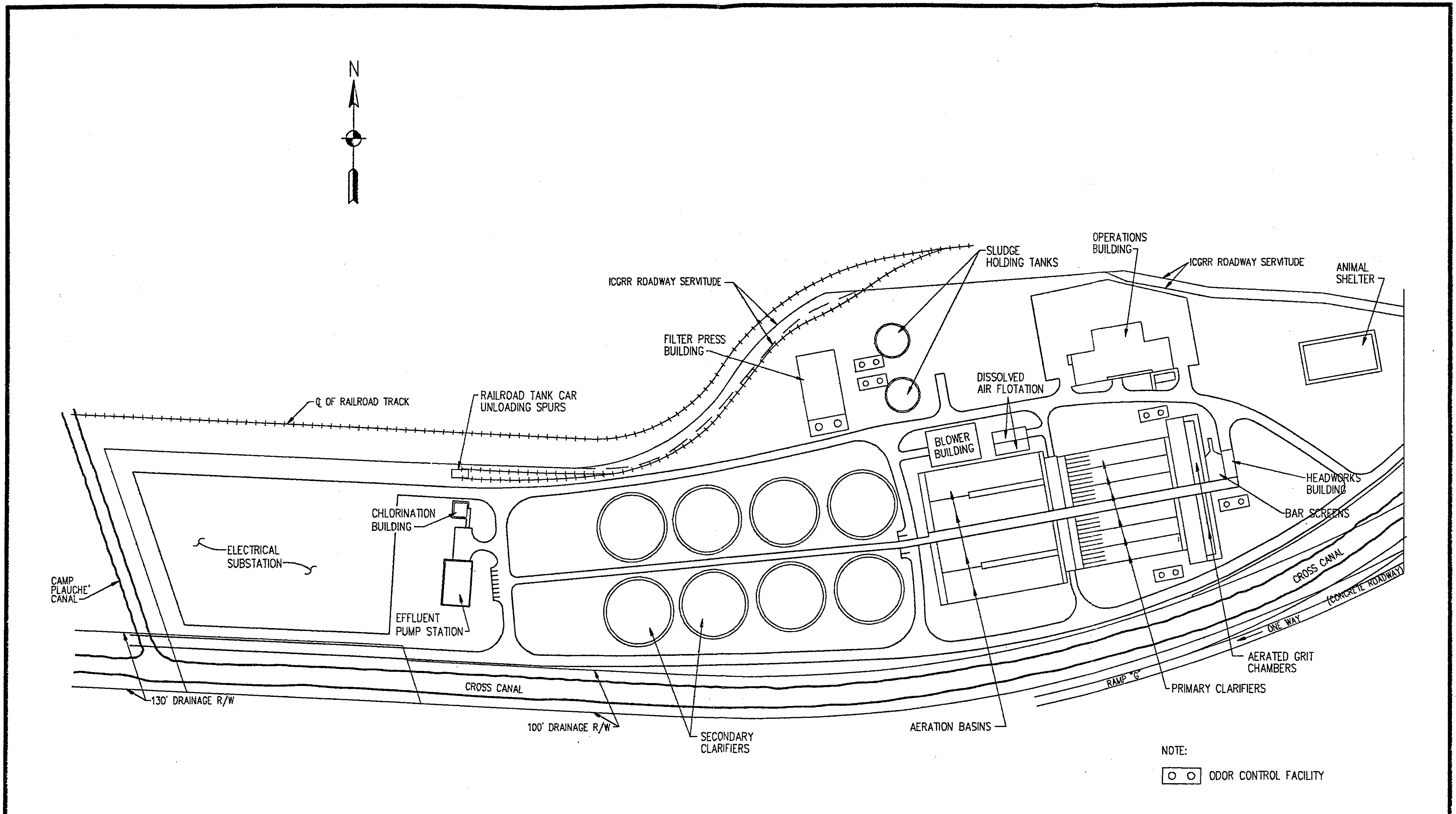


FIGURE V.8
EAST BANK WWTP
PLANT LAYOUT

E.2.a Preliminary Treatment

Preliminary treatment consists of three barscreens followed by two aerated grit chambers. The objective of screening is to remove coarse solids such as rags, debris and other materials from influent wastewater for the protection of downstream process equipment. All three screens are mechanically cleaned bar screens. During normal operation, two of the screens always remain in operation. Screenings from the units are discharged by means of a conveyor belt system to containers located within the building which houses the plant's headworks. The three screens are installed at an 80 degree angle from horizontal.

The objective of grit removal is to remove the heavier inorganic materials such as sand, gravel, cinders, and other heavy solid materials. Grit removal is provided to protect downstream mechanical equipment from excessive wear and to reduce the formation of deposits in pipelines, channels, and basins. Each grit chamber is 152 feet long by 21 feet wide. The chambers are outfitted with longitudinal baffles to prevent short-circuiting. Each chamber is fitted with ten 7.5 hp centrifugal grit pumps which provide grit removal from the chambers.

E.2.b Primary Treatment

Raw wastewater contains suspended particulates heavier than water; these particles tend to settle by gravity under quiescent conditions. Primary clarifiers at the East Bank WWTP are used to remove readily settleable solids prior to subsequent treatment by the activated sludge process and secondary clarification. Primary clarification reduces the organic and solids loading to these processes. The East Bank WWTP has six rectangular concrete tanks for primary sedimentation with room available for adding two more tanks in the future. Each tank is 40-ft wide by 235-ft long.

E.2.c Secondary Treatment

Secondary treatment at the plant consists of four conventional activated sludge aeration basins followed by eight circular clarifiers. The activated sludge system is used for carbonaceous and nitrogenous BOD_5 removal. Each aeration basin is 70-ft wide by 270-ft long by 15-ft deep. The design of the activated sludge system provides flexibility for several alternative modes of operations including

> conventional mode, step feed mode, sludge reaeration mode and the provision of a contact stabilization system. Currently the activated sludge process at the East Bank WWTP is operated in the conventional mode.

All basins are equipped with a ceramic type fine bubble air diffuser system having a minimum clean water oxygen transfer efficiency of about 25 %. The air supply system for the aeration basins consists of five 640 hp centrifugal blowers with 15,000 cfm capacity each. Each blower unit is provided with high efficiency intake air filters. Four of the five units provide air to the aeration basins.

Secondary clarification separates MLSS or activated sludge from the treated wastewater prior to discharge and thickens it before being returned to the aeration basin or wasted to the dissolved air flotation units. The secondary clarifiers consist of eight 150-ft diameter reinforced concrete units each with a volumetric capacity of 1.85 million gallons. Because of the ample secondary clarification capacity, some of the clarifiers remain idle during the majority of operations.

E.2.d Disinfection

A chlorination system was provided for the East Bank WWTP upon its initial construction. This system was not utilized until spring of 1994, when disinfection was first required by the NPDES permit for the plant. The system consists of chlorine injection into the effluent channel from the secondary clarifiers approximately 400 feet upstream of the effluent pump station wet well. No chlorine contact chamber is provided for the chlorinated effluent, with chlorine contact provided in the approximately 3.5 mile long effluent force main which discharges to the Mississippi River. The major components of the East Bank WWTP chlorination system consists of two 55 ton chlorine rail car unloading spurs designed to supply liquid chlorine to three chlorine evaporators. The chlorination feed equipment at the plant consists of three liquid chlorine evaporators with capacities of 8000 lbs/day each and three chlorinators with capacities of 8000 lbs/day each which inject chlorine into solution lines which can inject chlorine at various locations in the plant. The injector utilized for the secondary effluent channel has a capacity of 4,700 lbs/day.

The East Bank WWTP presently has Total Residual Chlorine (TRC) limits for its effluent discharge to the Mississippi River. The plant is presently meeting these limits without providing dechlorination of its effluent.

E.2.e Sludge Handling

Dissolved Air Flotation

Two dissolved air flotation units are provided to thicken the solids and waste activated sludge withdrawn from the primary and secondary clarifiers at the East Bank WWTP prior to sludge dewatering. Each of the two units is 45 feet long by 20 feet wide by 10 feet deep. The purpose of the units is to thicken the activated sludge and solids prior to their transfer to the sludge holding tanks and dewatering by the belt filter presses. Through thickening the sludge requires less holding tank storage capacity and the performance of the belt filter presses is improved.

Sludge Holding Tank

The East Bank WWTP has two holding tanks to serve as a balancing reservoir for the belt filter press facilities. Each tank has the sludge holding capacity of 535,000 gallons. The filter press dewatering operations operate in a batch mode (8-12 hrs/day, 5-6 days/week. Balancing is required due to continual sludge production by the activated sludge system and the batch operation of the belt filter press dewatering operations. The holding tanks are gas tight and sludge is withdrawn from the holding tanks by progressive cavity pumps and pumped to the filter press building. Aeration of the sludge holding tanks is provided to prevent the development of anaerobic conditions.

Sludge Dewatering System

The purpose of the belt filter press facilities is to dewater the activated sludge prior to land disposal and to transform the sludge into a firm cake. Four 2-meter wide belt filter presses are presently provided for sludge dewatering at the East Bank WWTP. These belt filter presses along with a sludge feed system, and other support systems are currently housed in a single building. The dewatering of the activated sludge by the belt filter presses is assisted by the addition of polymer. The belt filter press dewatering operations operate in a batch mode. The resulting dewatered sludge is disposed at the Kelven Landfill in Waggaman, Louisiana.

E.2.f Odor Control

Odors are the results of putrefaction of the solids in wastewaters. Many distinctive and unpleasant odors are caused by bacteria and enzymes acting on protein matter. Ammonia, hydrogen sulphide, volatile fatty acids, and mercaptans are mainly responsible for odors. Experience has shown that when H_2S is controlled, other obnoxious odors are controlled as well. H_2S production usually occurs within the collection system when wastewater is detained in the system long enough to experience anaerobic conditions. Factors contributing to the generation of odors (H_2S) are high BOD_5 in the wastewater, high sulphate in the wastewater, high sewage temperature, sluggish and stagnant flow conditions, and lack of air cover in force mains and sludge deposits. The agitation of these stagnant flows and high sulfide wastewaters upon entrance to the East Bank WWTP will cause odor problems.

Properly designed and operated systems minimize odor production to the extent that no nuisance conditions are created. At the East Bank WWTP all channels, grit chambers, and primary clarifiers are covered and vented through an HOCL scrubber unit in order to prevent the emission of odors. One HOCL scrubber is provided for the headworks and an HOCL scrubber is provided for the north and south side of the primary clarifiers. The dissolved air flotation and filter press building are also enclosed and vented through an HOCL scrubber unit to prevent emission of odors. An HOCL scrubber is provided for each of these facilities. The East Bank WWTP also has the capacity to inject chlorine into the wastewater at several locations within the plant to provide odor control. These locations include the headworks, primary clarifier, and dissolved air flotation units. The equipment to provide chlorine injection at these locations has been provided but has not been operated since the construction of the plant.

E.3 EVALUATION OF PRESENT TREATMENT CAPACITY

E.3.a **Preliminary Treatment**

1. **Bar Screen**

Type : Mechanically Cleaned Bars
Number : 3
Bar Size : 1/2" x 2-1/2" (deep)
Spacing : 3/4"
Width of Screen : 8'
Incline : 80 degrees from horizontal
Operating Depth : 5-1/2'
Channel Cross Section Area : 5-1/2' x 8' = 44.8 ft^2
Effective Channel Area : 0.6 x 44.8 = 26.88 ft^2

Design Criteria (WEF Manual of Practice No.8)
Max. Approach Velocity = 3.0 ft/sec
Max. Design Flow = 3.0 ft/sec x 44.8 ft^2 = 134.4 ft^3/sec = 86.8 MGD

Both bar screens operating together should be able to handle 173.6 MGD.

2. **Grit Chamber**

Type : Aerated Horizontal Channel with Rectangular Cross-Section
Number : 2
Dimensions : Width : 21'
Length : 152'
Depth : 12.5'

Design Criteria (WEF Manual Of Practice No. 8)
Cross Sectional Area = 262.5 ft^2
Grit Chamber Volume = 39900 ft^3
Max. Flow Detention Time = 3 minutes
Minimum Length to Width Ratio = 2.5 to 1 (7.2 to 1 utilized)
Max. Design Flow = 39900 ft^3/3 min = 221.7 cfs = 143 MGD

Each grit chamber is capable of treating a peak hourly flow of 143 MGD. The combined capacity of the aerated grit chambers is 286 MGD.

E.3.b **Primary Treatment**

Primary Clarifiers

Type: Rectangular with helical scum removal
Number of Units: 6
Dimensions: Length: 235 ft
Width: 40 ft
Depth: 11 ft
Surface Area per Unit: 9400 ft^2
Total Surface Area: 56400 ft^2

Volume: 103,400 ft^3 (per clarifier)
Weir Length: 400 ft (per clarifier)

Detention Time: Sufficient detention time for contact between solids particles is necessary for flocculation and effective sedimentation. Typical detention times for primary sedimentation tank are summarized in Table 10.4, p.458, WEF Manual of Practice No. 8.

According to the analysis presented in Table V-6 below the existing primary clarifiers should be capable of treating average daily, peak daily, and peak hourly flows of 56 MGD, 85 MGD, and 141 MGD, respectively.

TABLE V-6 EXISTING PRIMARY CLARIFIER DESIGN CRITERIA

	LIMITING CRITERIA	RATED CAPACITY
Average Daily Flows	1000 gpd/ft^2	56 MGD
Peak Daily Flows	1500 gpd/ft^2	85 MGD
Peak Hourly Flows	2500 gpd/ft^2	141 MGD
Weir Loading (At Average Daily Flows)	23333 gpd/ft	56 MGD
Weir Loading (At Peak Daily Flows)	35000 gpd/ft	85 MGD
Weir Loading (At Peak Hourly flow)	58750 gpd/ft	141 MGD

E.3.c **Secondary Treatment**

1. **Activated Sludge Aeration Basins**

Type : Conventional
Sludge Recycle rate : 50 % to 70 %
Dimensions: Width : 70 '
Depth : 15 '
Length : 270 '
No. of Basins : 4
Total Volume : 1134000 ft^3 , 8.48 Million Gallons

Hydraulic Loading
Detention Time
At Peak Daily Flow V/Q = 2.0 hrs
Peak Daily Flow = 8.48 million Gallons/2.0 hrs =102 MGD
At Average Daily Flow: V/Q = 4.00 hrs*
Average Daily Flow: 8.48 Million Gallons/4.0 hrs = 51 MGD
* (According to Design of Municipal Wastewater Treatment Plants Volume I {WEF Manual of Practice No. 8}, 1992 p. 627 : For Conventional Plug Flow Basin V/Q should equal 4 - 8 hrs
Hydraulic Loading: Average flows and peak daily flows of the existing aeration basins based on hydraulic detention time criteria is determined to be 51 MGD and 102 MGD, respectively.

Organic Loading
Design Criteria (WEF Manual of Practice No. 8)
Organic Loading Capacity = 37.3 BOD_5/1000 ft^3 day x 1134000 ft^3 = 42298 lbs/day
Number of Blowers: 5 at 15,000 icfm each
Blower discharge pressure: 23 psia
Air Requirements Average Loading: 28,351 SCFM, approx. 31,501 icfm
Air Capacity: 75,000 icfm.

(Air Requirement calculations shown in Appendix E, calculation assumed an influent NH_3 concentration of 25 mg/l which provided the majority of the oxygen demand. The concentration experienced is most likely much lower since current plant operation indicates the need for only one 15,000 icfm blower)

2. **Secondary Clarifiers**
Type : Circular with Scum Removal
Number of Units: 8
Diameter = 150'
Side Water Depth = 14'
Surface Area = 17675 ft^2 (per clarifier)
Total Volume = 247,450 ft^3, 1.85 MG
Weir Length : 418' per clarifier

The existing secondary clarifier capacity was determined according to standards outlined in WEF Manual of Practice No. 8. According to the analysis performed in Table V.7 the existing secondary clarifiers should be capable of treating average daily flows, peak daily flows, and peak hourly flows of 85 MGD, 100 MGD, and 141 MGD, respectively.

TABLE V-7 EXISTING SECONDARY CLARIFIER DESIGN CRITERIA

	LIMITING CRITERIA	RATED CAPACITY
Average Daily Flows	600 gpd/ft^2	85 MGD
Peak Daily Flows	1000 gpd/ft^2	141 MGD
Peak Hourly Flows	1500 gpd/ft^2	212 MGD
Weir Loading (At Average Daily Flows)	25,419 gpd/ft	85 MGD
Weir Loading (At Peak Daily Flows)	30,000 gpd/ft	100 MGD
Weir Loading (At Peak Hourly flow)	42,165 gpd/ft	141 MGD

E.3.d **Disinfection**

Chlorine Injector Capacity : 4,700 lbs/day
Anticipated Chlorine Dosage : less than 2-3 mg/l (based upon present operating data)
Demand at Average Daily Flow : 36 MGD x 3 mg/l x 8.34 = 901 lbs/day
Demand at Peak Daily Flow : 56 MGD x 3 mg/l x 8.34 = 1401 lbs/day
Demand at Peak Hourly Flow : 145 MGD x 3 mg/l x 8.34 = 3,628 lbs/day

Capacity should be adequate for all flows experienced unless changes in wastewater characteristics occur. Assuming a dosage of 3 mg/l, system can handle a maximum flow of 188 MGD.

E.3.e **Sludge Treatment**

Belt Press Dewatering:
Type : Belt filter press
Number : 4
Width : 2 meter
Present Operating Schedule : 45-50 hrs/week
(assuming 3 units in operation)
Estimated Sludge Flow : 487 tons/week dewatered

Polymer addition utilized : yes
Typical Cake Solids Concentration : 23 %
Dry Solids Produced : 487 x 23 % = 112 tons/week

Sludge Dewatering Capacity :
Total Capacity: 4 presses x 8 hrs/day X 5 days/week x 1 dry ton/hr/.23 dry ton/wet ton = 695 wet tons/week
Capacity of 3 belt filter presses = 522 wet tons/week

E.3.f **Dissolved Air Flotation**

Surface Area = 45 ft x 20 ft = 900 ft^2 (per unit)
Assume solids loading rate = 0.8 lbs/hrs/ft^2
Existing Sludge withdrawal rate = 70 gpm primary, 300 gpm secondary
Sludge concentration = 0.75%

Anticipated solids loading to DAF unit
= 370 gallons/min. x 60 min/hr. x 0.75 % x cf/7.48 gallons x 62.4 lbs/cf
= 1390 lbs/hr

Solid loading capacity/effective flotation area
= 0.8 lb/hr/ft^2 x 1800 ft^2
= 1440 lbs/hr > 1390 lbs/hr

The existing DAF units are capable of treating the existing sludge produced.

HYDRAULIC LOADING CAPACITY

DAF thickeners are generally designed hydraulically to operate in the range of 0.5 to 2 gpm/ft^2. Assuming the hydraulic loading rate for the EBWWTP DAF thickeners to be 0.6 gpm/ft^2, the hydraulic loading capacity of the existing DAF thickeners

= 0.6 gpm/ft^2 x 1800 ft^2 x 2
= 1080 gpm = 1.6 MGD

The existing DAF thickeners have sufficient hydraulic loading capacity to handle increase in sludge quantity due to additional stormwater flows.

E.4 SUMMARY OF EXISTING PLANT CAPACITY

The existing capacity of the East Bank WWTP is summarized in Table V-8 below. Based upon this summary, the plant is capable of adequately handling well in excess of the 35 MGD average flows presently experienced at the plant. From this analysis it appears the critical units in establishing additional plant capacity are the primary clarifiers and aeration basins. Based on limiting factors, the overall plant capacity should be rated at 51 MGD, 85 MGD, and 141 MGD, for average daily, peak daily, and peak hourly flows, respectively. The average daily flow is restricted by the aeration basin hydraulic loading capacity. The peak daily flow is restricted by the primary clarifier capacity. As shown in Table V-8 the capacity will be exceeded with the addition of stormwater to the East Bank WWTP. The need to upgrade the plant will be discussed in Subsection F herein.

The existing solids train treatment capacity is sufficient to handle the present quantities of sludge produced. However, as shown in Table V-8 the capacity will be exceeded with the addition of stormwater to the East Bank WWTP. The need to upgrade will be discussed in Subsection F below.

TABLE V-8 SUMMARY OF EXISTING PLANT CAPACITIES

		AVERAGE DAILY FLOW	PEAK DAILY FLOW	PEAK HOURLY FLOW	AVERAGE DESIGN FLOW FOR PLANT UPGRADE
LIQUID TRAIN					
BAR SCREEN (3)		------	------	174 MGD	71.2 MGD
GRIT CHAMBER				286 MGD	71.2 MGD
PRIMARY CLARIFIER		**56 MGD**	**85 MGD**	**141 MGD**	**71.2 MGD**
AERATION BASIN	DETENTION TIME	**51 MGD**	**102 MGD**		**71.2 MGD**
	ORGANIC LOADING	42,300 LBS/DAY	56,397 LBS/DAY		47,900 LBS/DAY
SECONDARY CLARIFIER		85 MGD	100 MGD	141 MGD	71.2 MGD
DAF UNIT (MAXIMUM SLUDGE)		0.55 MGD			
HOLDING TANK (MAXIMUM SLUDGE)		**1.07 MG (2.7 Days of Storage)**			**1.4 MGD (2.5 Days)**
BELT PRESS UNIT		**24 Dry Ton/Day**			**30 Dry Ton/Day**

F. NECESSARY UPGRADES TO ACCOMMODATE STORMWATER TREATMENT

The need to upgrade the existing wastewater treatment facilities at the East Bank WWTP was based on several criteria. The primary criteria was the ability of the existing WWTP to handle increases in average daily flow due to the daily treatment of stormwater flows. The anticipated average daily flow of 71.2 MGD was based upon the present average flow to the plant of approximately 36 MGD plus 10% of 36 MGD to allow for future flow increases plus the anticipated stormwater flow of 22,000 gpm or 31.6 MGD. The 10% increase was based upon the Bureau of Economic Analysis Regional Projections projection in 1985 that the population of the New Orleans would increase by 14% between from 1990 to 2015. This was adjusted downward to 10% to account for growth between 1990 and 1994 and the fact that the 1990 census indicated negative growth trends for Jefferson Parish between 1980 and 1990. No increase in peak daily or hourly flows were anticipated. The ability of the WWTP to handle the stormwater flows should also consider how these flows change the characteristics of the wastewater which will be treated by the plant. The two primary considerations in this area is the increase in solids loading (TSS) and BOD_5 loading to the plant and the decrease in the BOD_5 concentration of the wastewater. The increase in solids loading could affect the effectiveness and efficiency of the bar screens and grit removal operations at the plant. Also, the increased solids loading may increase the sludge produced by the plant possibly requiring upgrades in sludge handling facilities. The change in BOD_5 concentration may also effect the efficiency and proper operation of the aeration basins at the plant.

As shown in Table V-8 upgrades would be necessary within the primary clarifiers and aeration basins to handle increases in average flows. Upgrades in sludge holding tanks and belt filter press units may also be necessary due to increases in solids and organic loadings.

F.1 PRELIMINARY TREATMENT

F.1.a Bar Screen

The existing capacity of the bar screens should be sufficient to handle peak hourly flows of 156 MGD. No upgrade is needed in order to handle the increase in average daily flows to 71.2 MGD. However, the addition of stormwater will add a waste stream with a TSS concentration of 116 mg/l or an additional TSS loading of approximately 30,571 lbs/day.

The TSS due to the storm flows may consist largely of construction debris, sticks, leaves, duckweed, bottle caps, cigarette butts, rocks and other fecal matter. This is evidenced by the corresponding low BOD_5 content of the incoming stormwater of 18 mg/l. This indicates that the TSS consists primarily of nonbiodegradable materials.

This presents two potential problems. If a large increase in fine sized in-organic solids occur, it may be beneficial to convert the existing bar screens with 3/4" bar openings to continuous screening devices with openings of 1/4" diameter openings. This is especially true if the primary clarifiers are removed from operation, which currently facilitate greater protection of aeration basin equipment. In addition, an increase in solids loading on the bar screen may require an upgrade of its solids removal system. The extent of these problems and need to upgrade facilitates depend on the composition, quantity, volume and characteristics of screening to be removed. Exact quantifications of these variables are not available at this time.

In order to establish the need for necessary upgrades a sieve analysis should be performed on the TSS associated with the stormwater. It is recommended this be performed during the preliminary design phase of this demonstration project.

Projected Costs

The projected cost to upgrade the existing bar screens to continuous fine screening devices (if necessary) is estimated at $350,000.

F.1.b Aerated Grit Chamber

The aerated grit chambers have sufficient capacity to handle the projected wastewater and stormwater peak flows of up to 280 MGD allowing for a five minute detention time. Longer detention times improve grit removal and provide additional preaeration as well as remove smaller grit particles.

As with the bar screens, no upgrade is necessary to handle the anticipated increase in average daily flows of up to 71.2 MGD. However, an increase in TSS loading may require upgrades in the grit removal equipment associated with the aerated grit chambers. Again, determination of these upgrades would require a sieve analysis of the TSS characteristics of the stormwater which should be performed in the preliminary design phase.

Projected Costs

Upgrade costs included in projected costs to upgrade bar screens.

F.2 PRIMARY TREATMENT

Based on the existing capacity of the primary clarifier as evaluated in Subsection E, the primary clarifiers will not be able to handle an increase in average daily wastewater flows to 71.2 MGD. If the primary clarifiers were to remain in service at their existing capacity, short circuiting would be experienced with an increased TSS and BOD_5 loading to the existing aeration basins. This, in turn, could exceed the BOD_5 loading capacity of the existing aeration basins. However, based on the determinations made herein, the complete removal of the primary clarifiers will not result in BOD_5 over loading of the aeration basins. In fact, as stated below, the removal of the primary clarifiers has been considered as an option for improving the *f/m* ratio within the aeration basins when stormwater is treated.

Projected Costs

The projected costs to remove the equipment from the primary clarifiers in order to convert them to aeration basins is included in the upgrade costs for secondary treatment.

F.3 SECONDARY TREATMENT

The critical operational unit in upgrading the existing plant capacity to treat average daily flows of 71.2 MGD is the existing aeration basins. Design criteria indicate that these aeration basins are not adequate for both the organic and hydraulic loading at an average design flow of 71.2 MGD (Table V-8). The existing aeration basin capacity can be improved by converting the primary clarifiers into aeration basins and by providing diffused aeration to some or all of the primary clarifiers. It is anticipated that four of the six primary clarifiers would have to be converted to meet the organic and hydraulic loading demands. The baffles and existing equipment within the remaining clarifiers could be removed to convert these clarifiers to bypass channels.

The activated sludge system is intended primarily for removal of carbonaceous BOD_5. However, it is anticipated that nitrification will occur during the warm summer months. Nitrogenous BOD_5 removal will be achieved by extending the detention time. With additional aeration capacity the detention time for average flow and peak daily flow will be 4 hrs. and 2 hrs, respectively after upgrade. The hydraulic and organic capacity of the aeration basin can also be improved by adding one aeration basin to the existing four aeration basins without primary clarifier conversion.

Of the two options, converting the primary clarifiers would be more cost effective at an estimated cost of $1,600,000 compared to the cost of adding one additional aeration basin at an estimated cost of $2,590,000. In addition, conversion of the primary clarifiers to aeration basins would increase the BOD_5 loading to the aeration and increase the *f/m* ratio.

However, primary clarification removal may result in carryover of deleterious material to the aeration basin and therefore affect operations. The decision on whether the primaries should be converted or an additional aeration basin be added will be determined in the preliminary design.

As discussed, a low *f/m* ratio gives rise to bulking of activated sludge. Bulking sludge settles more slowly than normal sludge due to the presence of filamentous organisms interfering with settlement and subsequent compaction of activated sludge. As calculated in Subsection C.4 the *f/m* ratio will be 0.10 to 0.11 day^{-1} during wastewater flows of 15 MGD anticipated from 2:00a.m. to 4:00a.m. and stormwater flows of 31.7 MGD which is well below the operational range for conventional activated sludge *f/m* ratio (0.20 - 0.40 day^{-1}). During higher wastewater flows 25 MGD-30 MGD and stormwater flows of 31.6 MGD the *f/m* ratio would be within the 0.2-0.40 day^{-1} operational range.

Due to the low *f/m* ratio it is expected that bulking of sludge will occur. This will result in much thinner sludge being returned to the aeration basin with a low MLSS which will lead to difficulty of maintaining the operational MLSS (2200 - 3000 ppm). A solution for this problem will be either controlling the height of sludge blanket by wasting more sludge than normal or adding additional food sources during these events. Wasting more sludge than normal will result in the MLSS concentration in the aeration tank rapidly declining and causing gradual reduction in treatment efficiency. Adding additional food sources during these situations can solve the problem of the low *f/m* ratio. There are two potential solutions for providing additional BOD_5 loading to the plant at low wastewater flows.

The first solution would be to allow any new significant industrial user that is anticipated to be located within the plant's collection system to discharge high strength wastewater during low wastewater and high stormwater flow periods. This could be achieved by either controlling the industrial discharge to the existing collection system or providing separate piping from the industry to the WWTP and providing discharge controls at the WWTP. Another, option would be to allow existing significant industrial contributors to increase their BOD_5 loading to the collection system. Jefferson Parish could alter their existing pretreatment program to allow for greater BOD_5 discharges. These industrial contributors will be excellent sources of BOD_5 for the WWTP, when needed. It is important to note that these additional BOD_5 sources will be needed mostly under special circumstances. Both of these options should be further evaluated during the preliminary design phase.

The clarification of the activated sludge will use eight circular clarifiers. These units will remove the biomass produced by treatment of the wastewater and stormwater in the activated sludge aeration basin. The main design criteria for secondary clarifiers are based on overflow rates. The evaluation of the performance of the secondary clarifiers was based on overflow rates for average daily, peak daily, and peak hourly flows. The design overflow rates utilized were 600 gpd/ft^2, 1000 gpd/ft_2, and 1600 gpd/ft^2, respectively. Based on these overflow rates it can be concluded that the secondary clarifiers at the WWTP have sufficient capacity to handle increases in the average daily average flows due to an additional 22,000 gpm (31.7 MGD) of stormwater.

F.4 DISINFECTION

As shown in Subsection E.3 above, the ability of the existing disinfection system to handle peak daily flows of 145 MGD has been demonstrated. The ability of the system to handle such flows are based on current wastewater characteristics. The addition of stormwater to the wastewater is anticipated to reduce the fecal coliform concentration of the wastewater stream (treated wastewater is anticipated to have fecal coliform counts in excess of 50,000 colonies/dl while post 1990 data has indicated stormwater fecal coliform concentrations of 982 colonies/dl). Thus, the concentration of chlorine which must be added to the plant effluent for disinfection should decline. Since the peak daily flows and peak hourly flows will not change despite the addition of stormwater flows, the existing chlorination system is considered adequate to handle the addition of stormwater flows to the treatment plant.

The addition of stormwater flows may result in the need to provide dechlorination of the plant's effluent prior to discharge to the Mississippi River. Presently, the plant is meeting its TRC limit of 0.426 mg/l. However, when the NPDES permit for the plant is modified due to its increase in average daily flow from 33.46 MGD to 71.2 MGD the TRC limit is expected to be changed to approximately 0.19 mg/l. The plant is presently operating fairly close to the existing TRC limit of 0.426 mg/l. With the corresponding decrease in detention time within the effluent force main with the increase in average daily flow it will be difficult to provide adequate disinfection and meet a TRC limit of 0.19 mg/l.

If a dechlorination system is to be provided, previous studies have recommended the best option is to provide a liquid sodium bisulfate injection facility near the effluent force main's discharge at the Mississippi River.

Projected Costs

Based on East Bank Wastewater Treatment Plant Disinfection Study prepared by Hartman Engineering, Inc. in August, 1993, the cost to provide dechlorination for the East Bank Plant was estimated at $53,000.

F.5 SLUDGE HANDLING

The projected sludge waste rate to the belt filter presses due to the treatment of an additional 22,000 gpm of stormwater will be approximately 30 dry tons/day. This is an increase over the present estimated sludge waste rate of approximately 17 tons/day. The projected volume of sludge will be 700,000 gals/days (assuming 1% solids).

The belt filter presses available to dewater the sludge are not capable of handling this increase in sludge loading within the present operating schedule. The belt filter presses are presently operated 40-45 hrs/week, five days a week. This schedule is governed by landfilling operations. The Kelven landfill is open for a certain number of hours in a day and certain days in a week. Based upon these operating restrictions, an additional two meter belt filter press capable of a 2000 lbs/hr dry solids throughput would be necessary to provide adequate capacity. Also, an additional truckbay and conveyor system would have to be added to the dewatering facility to assure sufficient truck loading capacity during the 40-45 hour operational week. To increase efficiency and maximize space utilization within the existing dewatering facility the use of five centrifuges to replace the existing belt filter presses should also be considered as an option. The costs for the proposed upgrades are indicated below.

Additional capacity would not be necessary if the operational hours could be extended. However, this would increase operational costs, and would require the storage of dewatered sludge on the plant grounds during time periods when the landfill is not operational. The storage of dewatered sludge on plant grounds may further increase operational costs and could result in vector and odor control problems.

The increase in sludge loading may also require an upgrade to the sludge holding tank and diffused air flotation facilities. The present volumetric capacity of the sludge holding tanks is 1,200,000 gallons. At the present, sludge production of approximately 396,000 gallons/day occurs. Providing a conservative assumption of sludge containing 1% solids this would result in approximately 2.7 days of storage capacity. If sludge production increases to 700,000 gallons/day only 1.5 days of storage capacity may be available. During weekend periods when the filter presses are inoperable for two days, the storage capacity may be exceeded preventing sludge from being wasted from the secondary clarifiers. This could, in turn, affect plant performance. Thus, it is recommended an additional 535,000 gallon holding tank be provided to assure an approximate two and one half day storage capacity (approx. 2.3 days).

The increase in solids loading to the dissolved air flotation units is anticipated to be from 1,390 lbs/hr to approximately 2,500 lbs/hr. This will exceed the units capacity resulting in sludge with a lower percent solids, a lower dewatered percent solids, and requiring greater holding tank capacity. Polymer can be added to improve the efficiency of the dissolved air flotation units. The expected increase in efficiency would be as follows:

Solids Loading Capacity/Effective Flotation Area=
$= 1.5$ lbs/hr/ft^3 x 1800 ft^2 = 2700 lbs/hr

Thus, polymer addition should adequately increase the efficiency of the dissolved air flotation basins to handle increased loadings. The cost to provide polymer addition should be minimal.

<u>It should be noted that the need to provide all the anticipated upgrades in sludge handling equipment is based heavily upon the increased TSS loading to the East Bank WWTP. It is also based upon the assumption that the majority of the TSS loading will not be removed by the plant headworks. A sieve analysis must be performed to establish the amount of TSS which will be removed as sludge and what portion will be removed as grit. This will establish if the following mentioned upgrades are necessary. This analysis, as previously stated, will be performed during the preliminary design phase.</u>

<u>Projected Costs</u>

The costs for the upgrades as discussed in this section are as follows:

Sludge Holding Tank:	$ 300,000
AND	
One Belt Filter Press (Plus Building Upgrades):	$ 413,000
OR	
Five Centrifuges (Plus Building Upgrades):	$1,800,000

F.6 ODOR CONTROL

It is anticipated that the treatment of stormwater flows will impact the existing odor control facilities. If an impact does occur it would be expected at the headworks of the plant. However, the BOD_5 content of the stormwater should result in low biological activity within the stormwater resulting in low sulfide production. Also, the provision of the proposed 22,000 gpm pump station should increase both the flow rates of stormwater and wastewater to the East Bank WWTP. This increase in flow rate should reduce the amount of H_2S produced in the existing wastewater stream. Thus, a need to upgrade the odor control facilities is not anticipated at this time.

G. ANTICIPATED PLANT UPGRADES

The capacity of the East Bank WWTP following the implementation of the upgrades proposed in Section F is summarized in Table V-9. The critical upgrades in this table are the conversion of the primary clarifiers to aeration basins. If this option is not selected than additional aeration capacity must be provided.

The estimated costs to upgrade the existing wastewater treatment facilities to handle increases in average daily flow due to the addition of stormwater flows will be dependent on the upgrades selected and if the preliminary design establishes all the proposed upgrades are indeed necessary. The estimated construction cost range is shown in Appendix A as **$2,925,250 TO $4,462,850.** Prices used to prepare this estimate were based upon estimates from equipment vendors, local contractors and Means Construction Cost Data 1993. Costs were increased to reflect anticipated construction costs in the year 1995.

TABLE V-9 SUMMARY OF UPGRADED EAST BANK WWTP CAPACITY

		AVERAGE DAILY FLOW	PEAK DAILY FLOW	PEAK HOURLY FLOW
BAR SCREEN (3)		100 MGD	125 MGD	210 MGD
PRIMARY CLARIFIER		Bypass	Bypass	Bypass
GRIT CHAMBER (2)		146 MGD	175 MGD	290 MGD
AERATION BASIN	DETENTION TIME	71 MGD	110 MGD	
	ORGANIC LOADING	49,900	57,000	
SECONDARY CLARIFIER		85 MGD	141 MGD	212 MGD
DAF UNIT (MAXIMUM SLUDGE)		0.75MGD		
HOLDING TANK (MAXIMUM SLUDGE)		1.6 MG (2.3 days)		
BELT PRESS UNIT (DRY TON)		32 MGD		

H. PROJECTED TREATMENT PLANT IMPROVEMENTS FOR FUTURE FLOW

As an additional consideration of the stormwater demonstration project it has been mentioned that an additional 26,000 gpm of stormwater may be directed to the East Bank WWTP for treatment for a total stormwater flow of 48,000 gpm (69.1 MGD). This would increase the total anticipated average daily flow at the East Bank WWTP from 71.2 MGD to 108.5 MGD. The plant upgrades necessary in addition to those mentioned in Subsection G are summarized in Table V-10. It is anticipated that the additional flows would require extensive aeration basin and secondary clarifier upgrades at a minimum. Also, it is projected that valuable information will be learned from the construction and operation of this demonstration project. This experience may affect the necessary improvements for further upgrades and therefore, costs have not been included herein.

I. EPA STORMWATER AND WASTEWATER PERMITTING REQUIREMENTS

I.1 STORMWATER PERMITTING

Since Jefferson Parish has yet to be issued a draft stormwater permit, the Parish can amend the Stormwater Management Plan submitted with Part II of their permit application to indicate the treatment to be provided by the demonstration project. Amending the existing stormwater management plan to include the new stormwater treatment plans will prevent the Parish from having to reapply.

I.2 WASTEWATER PERMITTING

A major modification to the existing NPDES permit for the East Bank WWTP, Permit No. LA0066630, must be filed if major changes to the treatment plant processes or flows occur. The necessary changes in plant processes and in the average daily flow of the East Bank WWTP to treat stormwater flow will warrant a major modification to the plant's NPDES permit. The major modification to the NPDES permit should include:

1. The capacity of the treatment plant expansion.
2. The manner in which treatment will be achieved.
3. The source of additional flow to the plant.
4. The amount of flow to the plant.
5. Sample results taken from the stormwater.

In addition to modifying the NPDES permit for the East Bank plant, the LWDPS permit (No. WP1153) issued by the Louisiana Department of Environmental Quality must also be modified to reflect the addition of stormwater to the treatment plant and treatment plant changes.

TABLE V-10 TREATMENTS UNIT UPGRADES NECESSARY TO TREAT AN ADDITIONAL STORMWATER FLOW OF 26,000 GPM OR AN AVERAGE DAILY FLOW OF 108.5 MGD.

Anticipated influent loading

BOD_5 = 59 mg/l
TSS = 126 mg/l
NH_3 = 9.2 mg/l
Flow = 108.5 MGD

As indicated

BAR SCREEN	No upgrade needed
GRIT CHAMBER	No upgrade needed to treat hydraulics flows. Necessity of further upgrade will depend on O&G, grit quantity, and grit size.
PRIMARY CLARIFIER	Bypassed
AERATION BASIN	The organic loading capacity of the aeration basins after the first upgrade will be sufficient to handle additional stormwater flows. In order to handle this flow hydraulically, two additional aeration basins (8.48 + 4.24 MGD) and conversion of the remaining two primary clarifiers (4.64 MGD) to the aeration basins will be needed. This upgrade will provide detention time of 3.84 hours at the flow of 108.5 MGD. **Further dilution of BOD_5 concentrations may make *f/m* ratios too low for proper functioning of the activated sludge process.**
SECONDARY CLARIFIER	Addition of two more 150' dia. clarifiers to the existing clarifiers will be able to handle the additional stormwater flow
DAF UNIT	Based on the DAF thickeners solid loading rate, additional capacity will be needed. One additional unit, 45 ft long x 20 ft wide x 10 ft deep, will be sufficient for this purpose.
BLOWER	Existing blowers have sufficient capacity to handle the increase in daily flow.
HOLDING TANK	No upgrade needed
BELT PRESS	Three additional 2 meter belt filter presses may be needed.

SECTION VI
COORDINATION WITH AGENCIES

A. CORPS OF ENGINEERS

The design team has closely coordinated their efforts with the Corps of Engineers (COE) throughout the evaluation phase period. Meetings have been held on a regular basis to review the status of the various project activities.

The COE's Technical Report on the project was issued in March 1994 with the Environmental Assessment issued in April 1994. As of this writing, comments on both of these documents are still being received by the District Office and are being addressed. The Project Cooperation Agreement (PCA) between the Department of the Army and Jefferson Parish will be executed upon final approval of these documents.

B. JEFFERSON PARISH

The work has also been closely coordinated with various departments of Jefferson Parish. Included in many of the progress meetings and subsequent discussions have been representatives of the Sewage, Drainage, Drainage Pump Stations, Public Works and Environmental Departments. Input from all representatives has occurred throughout this period and all formal comments have been included or addressed.

C. LOUISIANA DOTD

Interviews have been held with representatives of the LaDOTD permit division at Bridge City relative to the jack and bore at Airline highway. Requirements of the DOTD have been obtained relative to this work and will be addressed in the preliminary and final design phases of the project.

D. RAILROADS

Inquiries were sent to officials of the Southern Pacific Railroad, New Orleans Public Belt Railroad and the Illinois Central Railroad. The railroad right-of-way within the project limits between Tribune Street and the WWTP is owned by Illinois Central on the north and New Orleans Public Belt on the south. The Public Belt portion is leased to Southern Pacific.

It is proposed to place the required force main beneath an existing swale ditch between the second and third northern most tracks of the Illinois Central property. A jack and bore beneath the first two tracks would be required at Tribune Street. The proposed route would then follow this swale ditch to the existing St. Peter's Ditch right-of-way at which point the force main would proceed south. It is proposed to open cut the force main in this area beneath the remaining tracks by utilizing the existing trestle crossings. The force main would exit the southernmost railroad right-of-way at St. Peter's Ditch then turn west and proceed parallel to the railroad yard to the WWTP. Installation, permit and fee requirements have been received from the three railroads involved and no objection to the project has been expressed.

SECTION VII
CONCLUSIONS AND RECOMMENDATIONS

This evaluation phase has expanded upon the concept of the stormwater treatment plan conceived in a conceptual phase as a method of improving the water quality of Lake Pontchartrain and thereby improving the quality of life for everyone living within the metro New Orleans region. The original concept plan was based on the broad application of engineering principals. In the evaluation phase, engineering effort was applied to develop and evaluate feasible alternatives.

It can be concluded that with a few modifications the conceptual plan can be implemented and constructed within the allotted budget. Implementation of the plan after construction will require intense coordination with the operations of Department of Drainage Pump Stations and possibly some alternations to their typical policy and procedures used in operating the pump stations.

The following list provides an item by item recommendation for implementing the plan to most effectively demonstrate the ability to improve the quality of stormwater runoff into Lake Pontchartrain:

1. Construct a 20,000 gpm lift station at the intersection of the Suburban Canal and West Napoleon Canal (Canal No. 4). The station shall be a stormwater only pump station furnished with five (5) 4000 gpm sewage pumps. The station will be fed by a 54 inch equivalent suction pipe with an automatically operated screen cleaner for stormwater influent.

2. Construct a 60 inch force main for direct transmission of the stormwater to the WWTP. The force main should be connected to the existing regional force main and provided with two gate valves, one on each branch, for isolation of the regional force main as applicable.

3. Construct a small (approximately 5 cfs) drainage pump station in the West Esplanade Canal (Canal No. 2) near West William David Drive with discharge into sewage lift station H8-3A. This station would demonstrate the concept of multiple site stations utilizing existing, or improved, sewage facilities.

4. Install a pollutant monitoring station on a 30 inch arch drainage outfall pipe at West Esplanade Avenue and West William David Drive. This station could accurately provide information as to what type of pollutants are present in the underground stormwater system and therefore help to predict the improvements to lake water quality should the project be implemented on an Eastbank wide basis.

5. Perform a sieve analysis to better define the characteristics of the total suspended solids (TSS) associated with the stormwater. This is required to determine the upgrades necessary to the existing bar screens.

6. Perform pilot scale studies on the activated sludge system to analyze the sensitivity to salt concentrations. This is required due to anticipated backflushing of the canal system with Lake Pontchartrain water.

7. Convert primary clarifiers to aeration basins or provide additional aeration basins.

8. Provide a dechlorination system near the WWTP effluent force main discharge at the Mississippi River.

9. Upgrade the sludge handling facilities by providing an additional sludge holding tank and belt filter press or replace the existing belt filter press by providing five centrifuges.

10. Continue discussions with the Department of Drainage Pump Stations to coordinate implementation of the required pumping plan associated with the operation of the stormwater lift station.

The total estimated cost to implement the demonstration project is contingent upon the modifications which will be required at the WWTP, the full extent of which will not be known until further studies are performed in the preliminary design phase. The estimated range of total costs is $19,666.00 to $22,776.00. The breakdown of the estimate is as follows:

Lift Station	$ 2,000,000
Force Main	$ 9,095,800
Multiple Site Station and Monitoring Station	100,000
Treatment Plant Upgrades	2,543,750 to $ 3,880,750
Subtotal	$13,739,550 to $15,076,550
Contingencies 15% ±	2,060,450 to $ 2,261,450
Total Construction	$15,800,000 to $17,338,000
Engineering & Design (non-Federal)	1,300,000
Construction Management (non-Federal)	1,400,000
Monitoring & Admin. (Federal)	1,788,000
Lands and Damages	750,000
Relocations	200,000
TOTAL COST	$21,238,000 to $22,776,000

Detailed cost estimates for the construction items are presented in Appendix A. The cost for federal and non-federal design, administration and monitoring as well as lands, damages and relocations were extracted from the Corps of Engineers Technical Report dated March, 1994. Comparing these estimates to that shown in the Technical Report shows that even the maximum total cost for all the required treatment plant upgrades and 60 inch force main is less than 1.5% greater than the previously computed $22,444,000. This further strengthens the recommendation for the 60 inch force main since present funding is available for this construction.

The project cost is to be shared as 75% Federal cost and 25% Jefferson Parish cost. Jefferson Parish is responsible for lands, easements, right-of-way, and relocations as part of the 25% share. Monitoring and federal administration costs will be included in the 75% share. The distribution of costs is as follows:

Total Cost	\$21,238,000 to \$22,776,000
Federal Cost	\$15,928,500 to \$17,082,000
Parish Cost	\$ 5,309,500 to \$ 5,694,000

Jefferson Parish is responsible for operating and maintaining the project and furnishing any replacements as required through the life of the project. The Parish will accomplish this function through the use of existing operating personnel and maintenance facilities.

APPENDIX A
COST ESTIMATES

STORMWATER QUALITY DEMONSTRATION PROJECT
PRELIMINARY CONSTRUCTION COST ESTIMATE
48" DIAMETER SEWER FORCE MAIN

ITEM NUMBER	DESCRIPTION	ESTIMATED QUANTITY	UNIT	ESTIMATED UNIT PRICE	TOTAL PRICE
1	MOBILIZATION	LUMP SUM	LUMP	368,000.00	368,000.00
2	REMOVAL OF STRUCTURES AND OBSTRUCTIONS	LUMP SUM	LUMP	60,000.00	60,000.00
3	TRAFFIC CONTROL	LUMP SUM	LUMP	40,000.00	40,000.00
4	CONSTRUCTION PHOTOGRAPHS AND VIDEO TAPING	LUMP SUM	LUMP	25,000.00	25,000.00
5	SEISMIC MONITORING	LUMP SUM	LUMP	15,000.00	15,000.00
6	48" DIA. FORCE MAIN	12,300.00	L.F.	275.00	3,382,500.00
7	LIMESTONE BEDDING	8,200.00	C.Y.	35.00	287,000.00
8	RIVERSAND BACKFILL	45,100.00	C.Y.	9.00	405,900.00
9	WOOD SHEETING	825.00	MFBM	500.00	412,500.00
10	GEOTEXTILE FABRIC	32,800.00	S.Y.	1.50	49,200.00
11	UNCLASSIFIED EXCAVATION	65,600.00	C.Y.	4.50	295,200.00
12	REMOVE & REPLACE CONCRETE ROADWAY	22,500.00	S.Y.	45.00	1,012,500.00
13	REMOVE & REPLACE ASPHALT ROADWAY (W. NAPOLEON)	900.00	S.Y.	45.00	40,500.00
14	RIVERSAND SUBBASE	17,500.00	C.Y.	9.00	157,500.00
15	REMOVE & REPLACE CONCRETE DRIVEWAYS	1,800.00	S.Y.	36.00	64,800.00
16	REMOVE & REPLACE CONCRETE SIDEWALKS	2,850.00	S.Y.	27.00	76,950.00
17	SOD	4,000.00	S.Y.	10.00	40,000.00
18	ADJUST SEWER HOUSE CONNS.	25.00	EACH	900.00	22,500.00
19	ADJUST WATER HOUSE CONNS.	25.00	EACH	300.00	7,500.00
20	WATERLINE OFFSETS	21.00	EACH	3,000.00	63,000.00
21	CONFLICT BOX	30.00	EACH	3,000.00	90,000.00
22	CATCH BASINS	40.00	EACH	2,200.00	88,000.00
23	48" X 36" TAPPING SLEEVE & VALVE AT LIFT STATION TIE-IN	1.00	EACH	25,000.00	25,000.00
24	AERIAL CROSSING AT WEST METAIRIE (EXCLUDES PIPE)	LUMP SUM	LUMP	25,000.00	25,000.00
25	CONNECTION TO EXISTING F. M. (INCLUDES REQ'D. PLUG VALVES)	LUMP SUM	LUMP	250,000.00	250,000.00
26	JACK AND BORE AT AIRLINE HIGHWAY	LUMP SUM	LUMP	200,000.00	200,000.00
27	JACK AND BORE AT RAILROAD	LUMP SUM	LUMP	200,000.00	200,000.00
28	TIE-IN AT S.T.P.	LUMP SUM	LUMP	25,000.00	25,000.00

SUB - TOTAL	7,728,550.00
PLUS 15% CONTINGENCIES + / -	1,159,450.00
TOTAL ESTIMATED COST OF 48" FORCE MAIN	8,888,000.00

STORMWATER QUALITY DEMONSTRATION PROJECT
PRELIMINARY CONSTRUCTION COST ESTIMATE
54" DIAMETER SEWER FORCE MAIN

ITEM NUMBER	DESCRIPTION	ESTIMATED QUANTITY	UNIT	ESTIMATED UNIT PRICE	TOTAL PRICE
1	MOBILIZATION	LUMP SUM	LUMP	410,900.00	410,900.00
2	REMOVAL OF STRUCTURES AND OBSTRUCTIONS	LUMP SUM	LUMP	60,000.00	60,000.00
3	TRAFFIC CONTROL	LUMP SUM	LUMP	40,000.00	40,000.00
4	CONSTRUCTION PHOTOGRAPHS AND VIDEO TAPING	LUMP SUM	LUMP	25,000.00	25,000.00
5	SEISMIC MONITORING	LUMP SUM	LUMP	15,000.00	15,000.00
6	54" DIA. FORCE MAIN	12,300.00	L.F.	340.00	4,182,000.00
7	LIMESTONE BEDDING	8,700.00	C.Y.	35.00	304,500.00
8	RIVERSAND BACKFILL	47,600.00	C.Y.	9.00	428,400.00
9	WOOD SHEETING	825.00	MFBM	500.00	412,500.00
10	GEOTEXTILE FABRIC	34,200.00	S.Y.	1.50	51,300.00
11	UNCLASSIFIED EXCAVATION	69,200.00	C.Y.	4.50	311,400.00
12	REMOVE & REPLACE CONCRETE ROADWAY	22,500.00	S.Y.	45.00	1,012,500.00
13	REMOVE & REPLACE ASPHALT ROADWAY (W. NAPOLEON)	900.00	S.Y.	45.00	40,500.00
14	RIVERSAND SUBBASE	17,500.00	C.Y.	9.00	157,500.00
15	REMOVE & REPLACE CONCRETE DRIVEWAYS	1,800.00	S.Y.	36.00	64,800.00
16	REMOVE & REPLACE CONCRETE SIDEWALKS	2,850.00	S.Y.	27.00	76,950.00
17	SOD	4,000.00	S.Y.	10.00	40,000.00
18	ADJUST SEWER HOUSE CONNS.	25.00	EACH	900.00	22,500.00
19	ADJUST WATER HOUSE CONNS.	25.00	EACH	300.00	7,500.00
20	WATERLINE OFFSETS	21.00	EACH	3,000.00	63,000.00
21	CONFLICT BOX	30.00	EACH	3,000.00	90,000.00
22	CATCH BASINS	40.00	EACH	2,200.00	88,000.00
23	48" X 36" TAPPING SLEEVE & VALVE AT LIFT STATION TIE-IN	1.00	EACH	25,000.00	25,000.00
24	AERIAL CROSSING AT WEST METAIRIE (EXCLUDES PIPE)	LUMP SUM	LUMP	25,000.00	25,000.00
25	CONNECTION TO EXISTING F. M. (INCLUDES REQ'D. PLUG VALVES)	LUMP SUM	LUMP	250,000.00	250,000.00
26	JACK AND BORE AT AIRLINE HIGHWAY	LUMP SUM	LUMP	200,000.00	200,000.00
27	JACK AND BORE AT RAILROAD	LUMP SUM	LUMP	200,000.00	200,000.00
28	TIE - IN AT S.T.P.	LUMP SUM	LUMP	25,000.00	25,000.00

SUB - TOTAL	8,629,250.00
PLUS 15% CONTINGENCIES + / -	1,295,750.00
TOTAL ESTIMATED COST OF 54" FORCE MAIN	9,925,000.00

STORMWATER QUALITY DEMONSTRATION PROJECT
PRELIMINARY CONSTRUCTION COST ESTIMATE
60" DIAMETER SEWER FORCE MAIN

ITEM NUMBER	DESCRIPTION	ESTIMATED QUANTITY	UNIT	ESTIMATED UNIT PRICE	TOTAL PRICE
1	MOBILIZATION	LUMP SUM	LUMP	433,100.00	433,100.00
2	REMOVAL OF STRUCTURES AND OBSTRUCTIONS	LUMP SUM	LUMP	60,000.00	60,000.00
3	TRAFFIC CONTROL	LUMP SUM	LUMP	40,000.00	40,000.00
4	CONSTRUCTION PHOTOGRAPHS AND VIDEO TAPING	LUMP SUM	LUMP	25,000.00	25,000.00
5	SEISMIC MONITORING	LUMP SUM	LUMP	15,000.00	15,000.00
6	60" DIA. FORCE MAIN	12,300.00	L.F.	370.00	4,551,000.00
7	LIMESTONE BEDDING	9,100.00	C.Y.	35.00	318,500.00
8	RIVERSAND BACKFILL	50,100.00	C.Y.	9.00	450,900.00
9	WOOD SHEETING	825.00	MFBM	500.00	412,500.00
10	GEOTEXTILE FABRIC	35,500.00	S.Y.	1.50	53,250.00
11	UNCLASSIFIED EXCAVATION	77,400.00	C.Y.	4.50	348,300.00
12	REMOVE & REPLACE CONCRETE ROADWAY	22,500.00	S.Y.	45.00	1,012,500.00
13	REMOVE & REPLACE ASPHALT ROADWAY (W. NAPOLEON)	900.00	S.Y.	45.00	40,500.00
14	RIVERSAND SUBBASE	17,500.00	C.Y.	9.00	157,500.00
15	REMOVE & REPLACE CONCRETE DRIVEWAYS	1,800.00	S.Y.	36.00	64,800.00
16	REMOVE & REPLACE CONCRETE SIDEWALKS	2,850.00	S.Y.	27.00	76,950.00
17	SOD	4,000.00	S.Y.	10.00	40,000.00
18	ADJUST SEWER HOUSE CONNS.	25.00	EACH	900.00	22,500.00
19	ADJUST WATER HOUSE CONNS.	25.00	EACH	300.00	7,500.00
20	WATERLINE OFFSETS	21.00	EACH	3,000.00	63,000.00
21	CONFLICT BOX	30.00	EACH	3,000.00	90,000.00
22	CATCH BASINS	40.00	EACH	2,200.00	88,000.00
23	48" X 36" TAPPING SLEEVE & VALVE AT LIFT STATION TIE-IN	1.00	EACH	25,000.00	25,000.00
24	AERIAL CROSSING AT WEST METAIRIE (EXCLUDES PIPE)	LUMP SUM	LUMP	25,000.00	25,000.00
25	CONNECTION TO EXISTING F. M. (INCLUDES REQ'D. PLUG VALVES)	LUMP SUM	LUMP	250,000.00	250,000.00
26	JACK AND BORE AT AIRLINE HIGHWAY	LUMP SUM	LUMP	200,000.00	200,000.00
27	JACK AND BORE AT RAILROAD	LUMP SUM	LUMP	200,000.00	200,000.00
28	TIE - IN AT S.T.P.	LUMP SUM	LUMP	25,000.00	25,000.00

SUB - TOTAL	9,095,800.00
PLUS 15% CONTINGENCIES +/-	1,364,200.00
TOTAL ESTIMATED COST OF 60" FORCE MAIN	10,460,000.00

STORMWATER QUALITY DEMONSTRATION PROJECT
PRELIMINARY CONSTRUCTION COST ESTIMATE
20,000 GALLON PER MINUTE LIFT STATION

20,000 GPM X $85 / GPM =	$1,700,000.00
SCREEN CLEANER (LUMP SUM) =	$200,000.00
SUCTION INTAKE & CANAL LINING (LUMP SUM) =	$100,000.00
SUB – TOTAL =	$2,000,000.00
+ 15% CONTINGENCIES	$300,000.00
TOTAL ESTIMATED COST =	$2,300,000.00

STORMWATER QUALITY DEMONSTRATION PROJECT
PRELIMINARY CONSTRUCTION COST ESTIMATE
REQUIRED WWTP UPGRADES

SITE WORK	$ 77,750
HEADWORKS	$ 0 TO 350,000
CONVERSION OF PRIMARY CLARIFIERS	$ 1,600,000
BELT FILTER PRESS OR CENTRIFUGES	$ 413,000 TO 1,800,000
SLUDGE HOLDING TANK	$ 300,000
DECHLORINATION FACILITIES	$ 53,000
ELECTRICAL/INSTRUMENTATION	$ 100,000 TO 200,000
SUB-TOTAL =	**$ 2,543,750 TO $ 3,880,750**
CONSTRUCTION CONTINGENCIES (15%±)	$ 381,500 TO $582,100
TOTAL ESTIMATED COST =	**$ 2,925,250 TO $4,462,850**

APPENDIX B
PRELIM RUN #1

10-06-1994
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DATA AND INFLUENT LIMITS FOR EAST BANK PLANT, JEFFERSON PARISH, HARRAHAN, LA

UNIT OPERATIONS FOR EAST BANK PLANT INCLUDE:

PRIMARY CLARIFICATION
ACTIVATED SLUDGE
FLOTATION THICKENING
OTHER SECONDARY

SLUDGE DISPOSAL METHOD FOR EAST BANK PLANT IS LANDFILL

PARAMETERS USED FOR CALCULATIONS

AVERAGE TOTAL FLOW(MGD) : 71.2

SLUDGE FLOW TO DIGESTER(MGD) : .759

DOMESTIC FLOW (MGD) : 39.6

SLUDGE FLOW TO DISPOSAL (MGD) : .0311

INDUSTRIAL FLOW (MGD): 31.6

% SOLIDS : 24.44

POLLUTANT	INFLUENT SAMPLING DATA (MG/L)	NPDES EFFLUENT LIM. (MG/L)	WATER QUAL. CRITERIA* (MG/L)	SLUDGE CRITERIA (MG/L)
BOD	80.440	30.000	30.000	-1.000
TSS	151.450	30.000	30.000	-1.000
NH3	0.855	-1.000	6.800	-1.000
CD	0.002	-1.000	0.010	20.000
CR	0.041	-1.000	0.016	100.000
CU	0.050	-1.000	0.019	-1.000
CN	0.196	-1.000	0.022	-1.000
PB	0.021	-1.000	0.032	100.000
HG	0.001	-1.000	0.000	4.000
NI	0.005	-1.000	1.400	-1.000
AG	0.007	-1.000	0.001	100.000
ZN	0.169	-1.000	0.120	-1.000
PHE	0.014	-1.000	0.005	-1.000
AS	0.002	-1.000	0.050	100.000
BE	0.001	-1.000	0.053	-1.000
SE	0.000	-1.000	0.260	20.000
CHLO	0.013	0.430	-1.000	-1.000
O&G	11.134	-1.000	-1.000	-1.000
COD	147.820	-1.000	-1.000	-1.000
TL	0.009	-1.000	-1.000	-1.000
SB	0.001	-1.000	-1.000	-1.000

-1 = NO DATA PROVIDED

* DILUTION RATIO IS: 11.07584 :1

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IFORMATION ON
AST BANK PLANT

INDUSTRIES AND DISCHARGE POLLUTANT CONCENTRATIONS (MG/L)

	NAME 1
FLOW IGD)	31.600
POL	
D	18.325
TSS	115.690
NH3	0.673
[illegible]	0.002
R	0.007
CU	0.035
[illegible]	0.144
B	0.032
HG	0.000
[illegible]	0.012
[illegible]	0.004
ZN	0.195
PHE	0.008
S	0.004
[illegible]	0.001
SE	0.000
ILO	0.029
G	18.822
COD	82.429
[illegible]	0.001
B	0.003

INDUSTRIES AND POLLUTANT LOADINGS (POUNDS/DAY)

	NAME 1
POL	
OD	4829.445
SS	30489.410
NH3	177.444
D	0.448
R	1.818
CU	9.329
N	37.845
B	8.407
HG	0.053
NI	3.136
G	0.975
N	51.286
PHE	1.977
S	1.028
BE	0.264
SE	0.053
CHLO	7.722
O&G	4960.346
COD	21723.720
TL	0.158
SB	0.738

* = DEFAULT DATA USED

INFORMATION ON
EAST BANK PLANT

POLLUTANT	DOMESTIC (UNCONTROL.) CONTRIB. CONC. (MG/L)	LOAD(LBS/D)	REMOVAL RATES (%) PRIMARY	SECONDARY	TERTIARY	POLLUTANT
BOD	130.00	42934.32	19 *	91 *	-	BOD
TSS	180.00	59447.52	20	95	-	TSS
NH3	1.00	330.26	1 *	35 *	-	NH3
CD	0.00	0.76	15 *	23	-	CD
CR	0.07	22.42	1	1	-	CR
CU	0.06	20.51	22 *	81	-	CU
CN	0.24	78.50	27 *	52	-	CN
PB	0.01	4.16	57 *	76	-	PB
HG	0.00	0.23	10 *	67	-	HG
NI	0.00	0.03	14 *	32 *	-	NI
AG	0.01	3.40	20 *	90	-	AG
ZN	0.15	49.28	27 *	66	-	ZN
PHE	0.02	6.21	8 *	53	-	PHE
AS	0.00	0.03	12 *	40 *	-	AS
BE	0.00	0.03	1 *	1 *	-	BE
SE	0.00	0.03	9 *	50 *	-	SE
CHLO	0.00	0.03	25	99	-	CHLO
O&G	5.00	1651.32	1 *	1 *	-	O&G
COD	200.00	66052.80	1 *	1 *	-	COD
TL	0.01	4.92	1 *	1 *	-	TL
SB	0.00	0.03	1 *	1 *	-	SB

* - INDICATES DEFAULT DATA USED

MASS BALANCE CHECK *

POLLUTANT	CALCULATED FROM FIELD SAMPLING DATA: UNCONTROLLABLE (DOMESTIC) SOURCES (LBS/DAY)	CONTROLLABLE (INDUSTRIAL) SOURCES (LBS/DAY)	TOTAL SOURCES (COLS.2&3) (LBS/DAY)	POTW INFLUENT LOAD (LBS/DAY)	DIFFERENCE (COLS.4&5) (PERCENT)
BOD	42934.32	4829.44	47763.76	47765.92	0.00
TSS	59447.52	30489.41	89936.92	89932.22	-0.01
NH3	330.26	177.44	507.71	507.71	-0.00
CD	0.76	0.45	1.21	1.19	-1.69
CR	22.42	1.82	24.24	24.23	-0.07
CU	20.51	9.33	29.84	29.81	-0.10
CN	78.50	37.84	116.35	116.33	-0.02
PB	4.16	8.41	12.57	12.59	0.16
HG	0.23	0.05	0.28	0.30	4.38
NI	0.03	3.14	3.17	3.15	-0.70
AG	3.40	0.98	4.38	4.39	0.39
ZN	49.28	51.29	100.56	100.53	-0.03
PHE	6.21	1.98	8.19	8.19	0.11
AS	0.03	1.03	1.06	1.07	0.75
BE	0.03	0.26	0.30	0.30	0.11
SE	0.03	0.05	0.09	0.12	27.81
CHLO	0.03	7.72	7.75	7.78	0.31
O&G	1651.32	4960.35	6611.67	6611.70	0.00
COD	66052.80	21723.72	87776.52	87776.52	-0.00
TL	4.92	0.16	5.08	5.11	0.54
SB	0.03	0.74	0.77	0.77	0.13

* -1 INDICATES DATA NOT AVAILABLE

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EAST BANK PLANT *

POLLUTANT	INHIBITION CONCENTRATIONS (MG/L) INHIB. ACT. SL.	INHIB. NITRIF.	INHIB. AN. DIG.	CALC HEADWKS INHIB THRESHOLD VALUES (LBS/D) INHIB. ACT. SL.	INHIB. NITRIF.	INHIB. AN. DIG.	LIMITING FACTOR	POLLUTANT
BOD	-1.00	-	-	-1.00	-	-	-1.00	BOD
TSS	-1.00	-	-	-1.00	-	-	-1.00	TSS
NH3	480.00	-	-	287906.90	-	-	287906.90	NH3
CD	1.00	-	-	698.60	-	-	698.60	CD
CR	1.00	-	-	599.81	-	-	599.81	CR
CU	1.00	-	-	761.29	-	-	761.29	CU
CN	0.24	-	-	195.22	-	-	195.22	CN
PB	0.10	-	-	138.09	-	-	138.09	PB
HG	0.10	-	-	65.98	-	-	65.98	HG
NI	1.00	-	-	690.47	-	-	690.47	NI
AG	0.25	-	-	185.56	-	-	185.56	AG
ZN	0.45	-	-	366.05	-	-	366.05	ZN
PHE	50.00	-	-	32272.17	-	-	32272.17	PHE
AS	0.10	-	-	67.48	-	-	67.48	AS
BE	-1.00	-	-	-1.00	-	-	-1.00	BE
SE	-1.00	-	-	-1.00	-	-	-1.00	SE
CHLO	-1.00	-	-	-1.00	-	-	-1.00	CHLO
O&G	-1.00	-	-	-1.00	-	-	-1.00	O&G
COD	-1.00	-	-	-1.00	-	-	-1.00	COD
TL	-1.00	-	-	-1.00	-	-	-1.00	TL
SB	-1.00	-	-	-1.00	-	-	-1.00	SB

COMPARISON OF HEADWORKS LOADING LIMITS
(LBS/DAY)

POLLUTANT	PASS-THROUGH	INHIBITION	SLUDGE QUALITY	DESIGN LOAD	FINAL LOADING
BOD	%197936.10	-1.00	-1.00	71256.96	71256.96
TSS	%356284.70	-1.00	-1.00	%118761.60	%118761.60
NH3	68804.75	%287906.90	-1.00	-1.00	68804.75
CD	85.41	698.60	22.55	-1.00	22.55
CR	106.29	599.81	2593.74	-1.00	106.29
CU	664.62	761.29	-1.00	-1.00	664.62
CN	301.44	195.22	-1.00	-1.00	195.22
PB	876.92	138.09	34.13	-1.00	34.13
HG	1.99	65.98	1.55	-1.00	1.55
NI	13540.73	690.47	-1.00	-1.00	690.47
AG	78.92	185.56	28.82	-1.00	28.82
ZN	2321.27	366.05	-1.00	-1.00	366.05
PHE	69.97	32272.17	-1.00	-1.00	69.97
AS	548.08	67.48	64.84	-1.00	64.84
BE	352.10	-1.00	-1.00	-1.00	352.10
SE	3420.00	-1.00	10.37	-1.00	10.37
CHLO	25533.77	-1.00	-1.00	-1.00	25533.77
O&G	-1.00	-1.00	-1.00	-1.00	-1.00
COD	-1.00	-1.00	-1.00	-1.00	-1.00
TL	-1.00	-1.00	-1.00	-1.00	-1.00
SB	-1.00	-1.00	-1.00	-1.00	-1.00

* -1 INDICATES DATA ARE NOT AVAILABLE OR APPLICABLE

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INFORMATION ON
EAST BANK PLANT

TOTAL POUNDS OF POLLUTANT COMPARED TO ACTUAL POUNDS/DAY *
USING A SAFETY FACTOR OF 0 %

POL.	ALLOWABLE TOTAL POUNDS/DAY	EXPANSION AND SAFETY FACTOR	ACTUAL UNCONTROL. POUNDS/DAY	ALLOWABLE INDUSTRIAL POUNDS/DAY	ACTUAL INDUSTRIAL POUNDS/DAY	REQUIRED IND. REMOVAL POUNDS/DAY
BOD	71256.96	0.00	42934.32	28322.65	4829.44	0.00
TSS	118761.60	0.00	59447.52	59314.08	30489.41	0.00
NH3	68804.75	0.00	330.26	68474.49	177.44	0.00
CD	22.55	0.00	0.76	21.79	0.45	0.00
CR	106.29	0.00	22.42	83.87	1.82	0.00
CU	664.62	0.00	20.51	644.11	9.33	0.00
CN	195.22	0.00	78.50	116.72	37.84	0.00
PB	34.13	0.00	4.16	29.97	8.41	0.00
HG	1.55	0.00	0.23	1.32	0.05	0.00
NI	690.47	0.00	0.03	690.44	3.14	0.00
AG	28.82	0.00	3.40	25.42	0.98	0.00
ZN	366.05	0.00	49.28	316.77	51.29	0.00
PHE	69.97	0.00	6.21	63.76	1.98	0.00
AS	64.84	0.00	0.03	64.81	1.03	0.00
BE	352.10	0.00	0.03	352.06	0.26	0.00
SE	10.37	0.00	0.03	10.34	0.05	0.00
CHLO	25533.77	0.00	0.03	25533.74	7.72	0.00
O&G	-1.00	-1.00	1651.32	-1.00	4960.35	-1.00
COD	-1.00	-1.00	66052.80	-1.00	21723.72	-1.00
TL	-1.00	-1.00	4.92	-1.00	0.16	-1.00
SB	-1.00	-1.00	0.03	-1.00	0.74	-1.00

* -1 INDICATES DATA ARE NOT AVAILABLE/APPLICABLE

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UNIFORM CONCENTRATION INDUSTRIAL EFFLUENT LIMITS (MG/L) FOR EAST BANK PLANT
(APPLIES TO NON-DOMESTIC FLOW ONLY)

POLLUTANT	USING TOTAL INDUSTRIAL FLOW W/O SAFETY FACTOR	USING TOTAL INDUSTRIAL FLOW W/ SAFETY FACTOR	USING ONLY IND. CONTRIBUTORY FLOWS* W/O SAFETY FACTOR	USING ONLY IND. CONTRIBUTORY FLOWS* W/ SAFETY FACTOR	CNTRIB FL (MGD)	BASIS	POLLUTANT
BOD	107.47	107.47	107.47	107.47	31.6000	D	BOD
TSS	200.00	200.00	200.00	200.00	31.6000	D	TSS
NH3	259.82	259.82	259.82	259.82	31.6000	W	NH3
CD	0.08	0.08	0.08	0.08	31.6000	S	CD
CR	0.32	0.32	0.32	0.32	31.6000	W	CR
CU	2.44	2.44	2.44	2.44	31.6000	W	CU
CN	0.44	0.44	0.44	0.44	31.6000	I	CN
PB	0.11	0.11	0.11	0.11	31.6000	S	PB
HG	0.00	0.00	0.00	0.00	31.6000	S	HG
NI	2.62	2.62	2.62	2.62	31.6000	I	NI
AG	0.10	0.10	0.10	0.10	31.6000	S	AG
ZN	1.20	1.20	1.20	1.20	31.6000	I	ZN
PHE	0.24	0.24	0.24	0.24	31.6000	W	PHE
AS	0.25	0.25	0.25	0.25	31.6000	S	AS
BE	1.34	1.34	1.34	1.34	31.6000	W	BE
SE	0.04	0.04	0.04	0.04	31.6000	S	SE
CHLO	96.89	96.89	96.89	96.89	31.6000	P	CHLO
O&G	@	@	@	@	31.6000		O&G
COD	@	@	@	@	31.6000		COD
TL	@	@	@	@	31.6000		TL
SB	@	@	@	@	31.6000		SB

* - INDUSTRY CONTRIBUTORY FLOW IS DEFINED AS TOTAL FLOW FROM THOSE INDUSTRIES CONTRIBUTING THE SPECIFIC POLLUTANT.

@ - INSUFFICIENT DATA AVAILABLE FOR DERIVATION OF A LOCAL LIMIT.

SEE USERS MANUAL FOR A MORE DETAILED EXPLANATION OF FOOTNOTES.

BASIS DEFINITION:

W - PASS THROUGH (WATER QUALITY)
P - PASS THROUGH (NPDES PERMIT)
I - INHIBITION
S - SLUDGE QUALITY
D - DESIGN LOADING**

** - BOD/TSS LOCAL LIMITS SET EQUAL TO POTW DESIGN LEVELS WHEN PRELIM MASS BALANCE DERIVES LESS STRINGENT LIMITS.

APPENDIX C
PRELIM RUN #2

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DATA AND INFLUENT LIMITS FOR 2AST BANK PLANT w/o PRIMARY, JEFFERSON PARISH, HARRAHAN, LA

UNIT OPERATIONS FOR 2AST BANK PLANT w/o PRIMARY INCLUDE:

ACTIVATED SLUDGE
FLOTATION THICKENING
OTHER SECONDARY

SLUDGE DISPOSAL METHOD FOR 2AST BANK PLANT w/o PRIMARY IS LANDFILL

PARAMETERS USED FOR CALCULATIONS

AVERAGE TOTAL FLOW(MGD) : 71.2

DOMESTIC FLOW (MGD) : 39.6

INDUSTRIAL FLOW (MGD): 31.6

SLUDGE FLOW TO DIGESTER(MGD) : .759

SLUDGE FLOW TO DISPOSAL (MGD) : .0311

% SOLIDS : 24.44

POLLUTANT	INFLUENT SAMPLING DATA (MG/L)	NPDES EFFLUENT LIM. (MG/L)	WATER QUAL. CRITERIA* (MG/L)	SLUDGE CRITERIA (MG/L)
BOD	80.440	30.000	30.000	-1.000
TSS	151.450	30.000	30.000	-1.000
NH3	0.855	-1.000	6.800	-1.000
CD	0.002	-1.000	0.010	20.000
CR	0.041	-1.000	0.016	100.000
CU	0.050	-1.000	0.019	-1.000
CN	0.196	-1.000	0.022	-1.000
PB	0.021	-1.000	0.032	100.000
HG	0.001	-1.000	0.000	4.000
NI	0.005	-1.000	1.400	-1.000
AG	0.007	-1.000	0.001	100.000
ZN	0.169	-1.000	0.120	-1.000
PHE	0.014	-1.000	0.005	-1.000
AS	0.002	-1.000	0.050	100.000
BE	0.001	-1.000	0.053	-1.000
SE	0.000	-1.000	0.260	20.000
CHLO	0.013	0.430	-1.000	-1.000
O&G	11.134	-1.000	-1.000	-1.000
COD	147.820	-1.000	-1.000	-1.000
TL	0.009	-1.000	-1.000	-1.000
SB	0.001	-1.000	-1.000	-1.000

-1 = NO DATA PROVIDED

* DILUTION RATIO IS: 11.07584 :1

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INFORMATION ON
2AST BANK PLANT w/o PRIMARY

INDUSTRIES AND DISCHARGE POLLUTANT CONCENTRATIONS (MG/L)

	NAME 1
FLOW (MGD)	31.600
POL	
BOD	18.325
TSS	115.690
NH3	0.673
CD	0.002
CR	0.007
CU	0.035
CN	0.144
PB	0.032
HG	0.000
NI	0.012
AG	0.004
ZN	0.195
PHE	0.008
AS	0.004
BE	0.001
SE	0.000
CHLO	0.029
O&G	18.822
COD	82.429
TL	0.001
SB	0.003

INDUSTRIES AND POLLUTANT LOADINGS (POUNDS/DAY)

	NAME 1
POL	
BOD	4829.445
TSS	30489.410
NH3	177.444
CD	0.448
CR	1.818
CU	9.329
CN	37.845
PB	8.407
HG	0.053
NI	3.136
AG	0.975
ZN	51.286
PHE	1.977
AS	1.028
BE	0.264
SE	0.053
CHLO	7.722
O&G	4960.346
COD	21723.720
TL	0.158
SB	0.738

* = DEFAULT DATA USED

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INFORMATION ON
2AST BANK PLANT w/o PRIMARY

POLLUTANT	DOMESTIC (UNCONTROL.) CONTRIB.		REMOVAL RATES (%)			POLLUTANT
	CONC. (MG/L)	LOAD(LBS/D)	PRIMARY	SECONDARY	TERTIARY	
BOD	130.00	42934.32	-	91 *	-	BOD
TSS	180.00	59447.52	-	95	-	TSS
NH3	1.00	330.26	-	35 *	-	NH3
CD	0.00	0.76	-	23	-	CD
CR	0.07	22.42	-	1	-	CR
CU	0.06	20.51	-	81	-	CU
CN	0.24	78.50	-	52	-	CN
PB	0.01	4.16	-	76	-	PB
HG	0.00	0.23	-	67	-	HG
NI	0.00	0.03	-	32 *	-	NI
AG	0.01	3.40	-	90	-	AG
ZN	0.15	49.28	-	66	-	ZN
PHE	0.02	6.21	-	53	-	PHE
AS	0.00	0.03	-	40 *	-	AS
BE	0.00	0.03	-	1 *	-	BE
SE	0.00	0.03	-	50 *	-	SE
CHLO	0.00	0.03	-	99	-	CHLO
O&G	5.00	1651.32	-	1 *	-	O&G
COD	200.00	66052.80	-	1 *	-	COD
TL	0.01	4.92	-	1 *	-	TL
SB	0.00	0.03	-	1 *	-	SB

* - INDICATES DEFAULT DATA USED

MASS BALANCE CHECK *

POLLUTANT	CALCULATED FROM FIELD SAMPLING DATA: UNCONTROLLABLE (DOMESTIC) SOURCES (LBS/DAY)	CONTROLLABLE (INDUSTRIAL) SOURCES (LBS/DAY)	TOTAL SOURCES (COLS.2&3) (LBS/DAY)	POTW INFLUENT LOAD (LBS/DAY)	DIFFERENCE (COLS.4&5) (PERCENT)
BOD	42934.32	4829.44	47763.76	47765.92	0.00
TSS	59447.52	30489.41	89936.92	89932.22	-0.01
NH3	330.26	177.44	507.71	507.71	-0.00
CD	0.76	0.45	1.21	1.19	-1.69
CR	22.42	1.82	24.24	24.23	-0.07
CU	20.51	9.33	29.84	29.81	-0.10
CN	78.50	37.84	116.35	116.33	-0.02
PB	4.16	8.41	12.57	12.59	0.16
HG	0.23	0.05	0.28	0.30	4.38
NI	0.03	3.14	3.17	3.15	-0.70
AG	3.40	0.98	4.38	4.39	0.39
ZN	49.28	51.29	100.56	100.53	-0.03
PHE	6.21	1.98	8.19	8.19	0.11
AS	0.03	1.03	1.06	1.07	0.75
BE	0.03	0.26	0.30	0.30	0.11
SE	0.03	0.05	0.09	0.12	27.81
CHLO	0.03	7.72	7.75	7.78	0.31
O&G	1651.32	4960.35	6611.67	6611.70	0.00
COD	66052.80	21723.72	87776.52	87776.52	-0.00
TL	4.92	0.16	5.08	5.11	0.54
SB	0.03	0.74	0.77	0.77	0.13

* -1 INDICATES DATA NOT AVAILABLE

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2AST BANK PLANT w/o PRIMARY *

POLLUTANT	INHIBITION CONCENTRATIONS (MG/L)			CALC HEADWKS INHIB THRESHOLD VALUES (LBS/D)				POLLUTANT
	INHIB. ACT. SL.	INHIB. NITRIF.	INHIB. AN. DIG.	INHIB. ACT. SL.	INHIB. NITRIF.	INHIB. AN. DIG.	LIMITING FACTOR	
BOD	-1.00	-	-	-1.00	-	-	-1.00	BOD
TSS	-1.00	-	-	-1.00	-	-	-1.00	TSS
NH3	480.00	-	-	285027.90	-	-	285027.90	NH3
CD	1.00	-	-	593.81	-	-	593.81	CD
CR	1.00	-	-	593.81	-	-	593.81	CR
CU	1.00	-	-	593.81	-	-	593.81	CU
CN	0.24	-	-	142.51	-	-	142.51	CN
PB	0.10	-	-	59.38	-	-	59.38	PB
HG	0.10	-	-	59.38	-	-	59.38	HG
NI	1.00	-	-	593.81	-	-	593.81	NI
AG	0.25	-	-	148.45	-	-	148.45	AG
ZN	0.45	-	-	267.21	-	-	267.21	ZN
PHE	50.00	-	-	29690.40	-	-	29690.40	PHE
AS	0.10	-	-	59.38	-	-	59.38	AS
BE	-1.00	-	-	-1.00	-	-	-1.00	BE
SE	-1.00	-	-	-1.00	-	-	-1.00	SE
CHLO	-1.00	-	-	-1.00	-	-	-1.00	CHLO
O&G	-1.00	-	-	-1.00	-	-	-1.00	O&G
COD	-1.00	-	-	-1.00	-	-	-1.00	COD
TL	-1.00	-	-	-1.00	-	-	-1.00	TL
SB	-1.00	-	-	-1.00	-	-	-1.00	SB

COMPARISON OF HEADWORKS LOADING LIMITS
(LBS/DAY)

POLLUTANT	PASS-THROUGH	INHIBITION	SLUDGE QUALITY	DESIGN LOAD	FINAL LOADING
BOD	63622.30	-1.00	-1.00	71256.96	71256.96
TSS	71256.96	-1.00	-1.00	%118761.60	%118761.60
NH3	67762.26	%285027.90	-1.00	-1.00	67762.26
CD	71.49	593.81	64.84	-1.00	64.84
CR	106.29	593.81	2593.74	-1.00	106.29
CU	307.99	593.81	-1.00	-1.00	307.99
CN	192.92	142.51	-1.00	-1.00	142.51
PB	259.83	59.38	136.51	-1.00	59.38
HG	1.53	59.38	1.82	-1.00	1.53
NI	11228.90	593.81	-1.00	-1.00	593.81
AG	26.31	148.45	37.05	-1.00	26.31
ZN	1293.82	267.21	-1.00	-1.00	267.21
PHE	59.79	29690.40	-1.00	-1.00	59.79
AS	456.73	59.38	92.63	-1.00	59.38
BE	352.10	-1.00	-1.00	-1.00	352.10
SE	2898.31	-1.00	12.65	-1.00	12.65
CHLO	982.07	-1.00	-1.00	-1.00	982.07
O&G	-1.00	-1.00	-1.00	-1.00	-1.00
COD	-1.00	-1.00	-1.00	-1.00	-1.00
TL	-1.00	-1.00	-1.00	-1.00	-1.00
SB	-1.00	-1.00	-1.00	-1.00	-1.00

* -1 INDICATES DATA ARE NOT AVAILABLE OR APPLICABLE

10-06-1994
13:48:31

INFORMATION ON
2AST BANK PLANT w/o PRIMARY

TOTAL POUNDS OF POLLUTANT COMPARED TO ACTUAL POUNDS/DAY *
USING A SAFETY FACTOR OF 0 %

POL.	ALLOWABLE TOTAL POUNDS/DAY	EXPANSION AND SAFETY FACTOR	ACTUAL UNCONTROL. POUNDS/DAY	ALLOWABLE INDUSTRIAL POUNDS/DAY	ACTUAL INDUSTRIAL POUNDS/DAY	REQUIRED IND. REMOVAL POUNDS/DAY
BOD	71256.96	0.00	42934.32	28322.65	4829.44	0.00
TSS	118761.60	0.00	59447.52	59314.08	30489.41	0.00
NH3	67762.26	0.00	330.26	67432.00	177.44	0.00
CD	64.84	0.00	0.76	64.08	0.45	0.00
CR	106.29	0.00	22.42	83.87	1.82	0.00
CU	307.99	0.00	20.51	287.48	9.33	0.00
CN	142.51	0.00	78.50	64.01	37.84	0.00
PB	59.38	0.00	4.16	55.22	8.41	0.00
HG	1.53	0.00	0.23	1.30	0.05	0.00
NI	593.81	0.00	0.03	593.77	3.14	0.00
AG	26.31	0.00	3.40	22.91	0.98	0.00
ZN	267.21	0.00	49.28	217.94	51.29	0.00
PHE	59.79	0.00	6.21	53.58	1.98	0.00
AS	59.38	0.00	0.03	59.35	1.03	0.00
BE	352.10	0.00	0.03	352.06	0.26	0.00
SE	12.65	0.00	0.03	12.62	0.05	0.00
CHLO	982.07	0.00	0.03	982.03	7.72	0.00
O&G	-1.00	-1.00	1651.32	-1.00	4960.35	-1.00
COD	-1.00	-1.00	66052.80	-1.00	21723.72	-1.00
TL	-1.00	-1.00	4.92	-1.00	0.16	-1.00
SB	-1.00	-1.00	0.03	-1.00	0.74	-1.00

* -1 INDICATES DATA ARE NOT AVAILABLE/APPLICABLE

10-06-1994
13:48:31

UNIFORM CONCENTRATION INDUSTRIAL EFFLUENT LIMITS (MG/L) FOR 2AST BANK PLANT w/o PRIMARY
(APPLIES TO NON-DOMESTIC FLOW ONLY)

POLLUTANT	USING TOTAL INDUSTRIAL FLOW W/O SAFETY FACTOR	USING TOTAL INDUSTRIAL FLOW W/ SAFETY FACTOR	USING ONLY IND. CONTRIBUTORY FLOWS* W/O SAFETY FACTOR	USING ONLY IND. CONTRIBUTORY FLOWS* W/ SAFETY FACTOR	CNTRIB FL (MGD)	BASIS	POLLUTANT
BOD	107.47	107.47	107.47	107.47	31.6000	D	BOD
TSS	200.00	200.00	200.00	200.00	31.6000	D	TSS
NH3	255.87	255.87	255.87	255.87	31.6000	W	NH3
CD	0.24	0.24	0.24	0.24	31.6000	S	CD
CR	0.32	0.32	0.32	0.32	31.6000	W	CR
CU	1.09	1.09	1.09	1.09	31.6000	W	CU
CN	0.24	0.24	0.24	0.24	31.6000	I	CN
PB	0.21	0.21	0.21	0.21	31.6000	I	PB
HG	0.00	0.00	0.00	0.00	31.6000	W	HG
NI	2.25	2.25	2.25	2.25	31.6000	I	NI
AG	0.09	0.09	0.09	0.09	31.6000	W	AG
ZN	0.83	0.83	0.83	0.83	31.6000	I	ZN
PHE	0.20	0.20	0.20	0.20	31.6000	W	PHE
AS	0.23	0.23	0.23	0.23	31.6000	I	AS
BE	1.34	1.34	1.34	1.34	31.6000	W	BE
SE	0.05	0.05	0.05	0.05	31.6000	S	SE
CHLO	3.73	3.73	3.73	3.73	31.6000	P	CHLO
O&G	@	@	@	@	31.6000		O&G
COD	@	@	@	@	31.6000		COD
TL	@	@	@	@	31.6000		TL
SB	@	@	@	@	31.6000		SB

* - INDUSTRY CONTRIBUTORY FLOW IS DEFINED AS TOTAL FLOW FROM THOSE INDUSTRIES CONTRIBUTING THE SPECIFIC POLLUTANT.

@ - INSUFFICIENT DATA AVAILABLE FOR DERIVATION OF A LOCAL LIMIT.

SEE USERS MANUAL FOR A MORE DETAILED EXPLANATION OF FOOTNOTES.

BASIS DEFINITION:

W - PASS THROUGH (WATER QUALITY)
P - PASS THROUGH (NPDES PERMIT)
I - INHIBITION
S - SLUDGE QUALITY
D - DESIGN LOADING**

** - BOD/TSS LOCAL LIMITS SET EQUAL TO POTW DESIGN LEVELS WHEN PRELIM MASS BALANCE
DERIVES LESS STRINGENT LIMITS.

APPENDIX D
PRELIM RUN #3
SLUDGE DISPOSAL

10-06-1994
13:46:52

DATA AND INFLUENT LIMITS FOR EAST BANK PLANT, JEFFERSON PARISH, HARRAHAN, LA

UNIT OPERATIONS FOR EAST BANK PLANT INCLUDE:

PRIMARY CLARIFICATION
ACTIVATED SLUDGE
FLOTATION THICKENING
OTHER SECONDARY

SLUDGE DISPOSAL METHOD FOR EAST BANK PLANT IS LANDFARM

PARAMETERS USED FOR CALCULATIONS

AVERAGE TOTAL FLOW(MGD) : 71.2

DOMESTIC FLOW (MGD) : 39.6

INDUSTRIAL FLOW (MGD): 31.6

SLUDGE FLOW TO DIGESTER(MGD) : .759

SLUDGE FLOW TO DISPOSAL (MGD) : .0311

% SOLIDS : 24.44

POLLUTANT	INFLUENT SAMPLING DATA (MG/L)	NPDES EFFLUENT LIM. (MG/L)	WATER QUAL. CRITERIA* (MG/L)	SLUDGE CRITERIA (MG/L)
BOD	80.440	30.000	30.000	-1.000
TSS	151.450	30.000	30.000	-1.000
NH3	0.855	-1.000	6.800	-1.000
CD	0.002	-1.000	0.010	39.000
CR	0.041	-1.000	0.016	1000.000
CU	0.050	-1.000	0.019	1000.000
CN	0.196	-1.000	0.022	-1.000
PB	0.021	-1.000	0.032	300.000
HG	0.001	-1.000	0.000	17.000
NI	0.005	-1.000	1.400	420.000
AG	0.007	-1.000	0.001	-1.000
ZN	0.169	-1.000	0.120	1000.000
PHE	0.014	-1.000	0.005	-1.000
AS	0.002	-1.000	0.050	41.000
BE	0.001	-1.000	0.053	-1.000
SE	0.000	-1.000	0.260	36.000
CHLO	0.013	0.430	-1.000	-1.000
O&G	11.134	-1.000	-1.000	-1.000
COD	147.820	-1.000	-1.000	-1.000
TL	0.009	-1.000	-1.000	-1.000
SB	0.001	-1.000	-1.000	-1.000

-1 = NO DATA PROVIDED

* DILUTION RATIO IS: 11.07584 :1

10-06-1994
13:46:52

INFORMATION ON
AST BANK PLANT

INDUSTRIES AND DISCHARGE POLLUTANT CONCENTRATIONS (MG/L)

	NAME 1
FLOW (MGD)	31.600
OL	
OD	18.325
SS	115.690
NH3	0.673
D	0.002
R	0.007
CU	0.035
CN	0.144
B	0.032
G	0.000
NI	0.012
G	0.004
N	0.195
PHE	0.008
S	0.004
E	0.001
SE	0.000
CHLO	0.029
&G	18.822
OD	82.429
TL	0.001
B	0.003

INDUSTRIES AND POLLUTANT LOADINGS (POUNDS/DAY)

	NAME 1
OL	
BOD	4829.445
SS	30489.410
NH3	177.444
CD	0.448
R	1.818
J	9.329
CN	37.845
B	8.407
G	0.053
NI	3.136
AG	0.975
N	51.286
HE	1.977
AS	1.028
BE	0.264
SE	0.053
CHLO	7.722
O&G	4960.346
COD	21723.720
TL	0.158
SB	0.738

* = DEFAULT DATA USED

10-06-1994
13:46:52

:NFORMATION ON
:AST BANK PLANT

POLLUTANT	DOMESTIC (UNCONTROL.) CONTRIB.		REMOVAL RATES (%)			POLLUTANT
	CONC. (MG/L)	LOAD(LBS/D)	PRIMARY	SECONDARY	TERTIARY	
BOD	130.00	42934.32	19 *	91 *	-	BOD
TSS	180.00	59447.52	20	95	-	TSS
NH3	1.00	330.26	1 *	35 *	-	NH3
CD	0.00	0.76	15 *	23	-	CD
CR	0.07	22.42	1	1	-	CR
CU	0.06	20.51	22 *	81	-	CU
CN	0.24	78.50	27 *	52	-	CN
PB	0.01	4.16	57 *	76	-	PB
HG	0.00	0.23	10 *	67	-	HG
NI	0.00	0.03	14 *	32 *	-	NI
AG	0.01	3.40	20 *	90	-	AG
ZN	0.15	49.28	27 *	66	-	ZN
PHE	0.02	6.21	8 *	53	-	PHE
AS	0.00	0.03	12 *	40 *	-	AS
BE	0.00	0.03	1 *	1 *	-	BE
SE	0.00	0.03	9 *	50 *	-	SE
CHLO	0.00	0.03	25	99	-	CHLO
O&G	5.00	1651.32	1 *	1 *	-	O&G
COD	200.00	66052.80	1 *	1 *	-	COD
TL	0.01	4.92	1 *	1 *	-	TL
SB	0.00	0.03	1 *	1 *	-	SB

* - INDICATES DEFAULT DATA USED

MASS BALANCE CHECK *

POLLUTANT	CALCULATED FROM FIELD SAMPLING DATA: UNCONTROLLABLE (DOMESTIC) SOURCES (LBS/DAY)	CALCULATED FROM FIELD SAMPLING DATA: CONTROLLABLE (INDUSTRIAL) SOURCES (LBS/DAY)	CALCULATED FROM FIELD SAMPLING DATA: TOTAL SOURCES (COLS.2&3) (LBS/DAY)	POTW INFLUENT LOAD (LBS/DAY)	DIFFERENCE (COLS.4&5) (PERCENT)
BOD	42934.32	4829.44	47763.76	47765.92	0.00
TSS	59447.52	30489.41	89936.92	89932.22	-0.01
NH3	330.26	177.44	507.71	507.71	-0.00
CD	0.76	0.45	1.21	1.19	-1.69
CR	22.42	1.82	24.24	24.23	-0.07
CU	20.51	9.33	29.84	29.81	-0.10
CN	78.50	37.84	116.35	116.33	-0.02
PB	4.16	8.41	12.57	12.59	0.16
HG	0.23	0.05	0.28	0.30	4.38
NI	0.03	3.14	3.17	3.15	-0.70
AG	3.40	0.98	4.38	4.39	0.39
ZN	49.28	51.29	100.56	100.53	-0.03
PHE	6.21	1.98	8.19	8.19	0.11
AS	0.03	1.03	1.06	1.07	0.75
BE	0.03	0.26	0.30	0.30	0.11
SE	0.03	0.05	0.09	0.12	27.81
CHLO	0.03	7.72	7.75	7.78	0.31
O&G	1651.32	4960.35	6611.67	6611.70	0.00
COD	66052.80	21723.72	87776.52	87776.52	-0.00
TL	4.92	0.16	5.08	5.11	0.54
SB	0.03	0.74	0.77	0.77	0.13

* -1 INDICATES DATA NOT AVAILABLE

EAST BANK PLANT *

POLLUTANT	INHIBITION CONCENTRATIONS (MG/L) INHIB. ACT. SL.	INHIB. NITRIF.	INHIB. AN. DIG.	CALC HEADWKS INHIB THRESHOLD VALUES (LBS/D) INHIB. ACT. SL.	INHIB. NITRIF.	INHIB. AN. DIG.	LIMITING FACTOR	POLLUTANT
BOD	-1.00	-	-	-1.00	-	-	-1.00	BOD
TSS	-1.00	-	-	-1.00	-	-	-1.00	TSS
NH3	480.00	-	-	287906.90	-	-	287906.90	NH3
CD	1.00	-	-	698.60	-	-	698.60	CD
CR	1.00	-	-	599.81	-	-	599.81	CR
CU	1.00	-	-	761.29	-	-	761.29	CU
CN	0.24	-	-	195.22	-	-	195.22	CN
PB	0.10	-	-	138.09	-	-	138.09	PB
HG	0.10	-	-	65.98	-	-	65.98	HG
NI	1.00	-	-	690.47	-	-	690.47	NI
AG	0.25	-	-	185.56	-	-	185.56	AG
ZN	0.45	-	-	366.05	-	-	366.05	ZN
PHE	50.00	-	-	32272.17	-	-	32272.17	PHE
AS	0.10	-	-	67.48	-	-	67.48	AS
BE	-1.00	-	-	-1.00	-	-	-1.00	BE
SE	-1.00	-	-	-1.00	-	-	-1.00	SE
CHLO	-1.00	-	-	-1.00	-	-	-1.00	CHLO
O&G	-1.00	-	-	-1.00	-	-	-1.00	O&G
COD	-1.00	-	-	-1.00	-	-	-1.00	COD
TL	-1.00	-	-	-1.00	-	-	-1.00	TL
SB	-1.00	-	-	-1.00	-	-	-1.00	SB

COMPARISON OF HEADWORKS LOADING LIMITS
(LBS/DAY)

POLLUTANT	PASS-THROUGH	INHIBITION	SLUDGE QUALITY	DESIGN LOAD	FINAL LOADING
BOD	%197936.10	-1.00	-1.00	71256.96	71256.96
TSS	%356284.70	-1.00	-1.00	%118761.60	%118761.60
NH3	68804.75	%287906.90	-1.00	-1.00	68804.75
CD	85.41	698.60	43.98	-1.00	43.98
CR	106.29	599.81	25937.40	-1.00	106.29
CU	664.62	761.29	320.21	-1.00	320.21
CN	301.44	195.22	-1.00	-1.00	195.22
PB	876.92	138.09	102.38	-1.00	102.38
HG	1.99	65.98	6.58	-1.00	1.99
NI	13540.73	690.47	340.43	-1.00	340.43
AG	78.92	185.56	-1.00	-1.00	78.92
ZN	2321.27	366.05	392.99	-1.00	366.05
PHE	69.97	32272.17	-1.00	-1.00	69.97
AS	548.08	67.48	26.59	-1.00	26.59
BE	352.10	-1.00	-1.00	-1.00	352.10
SE	3420.00	-1.00	18.67	-1.00	18.67
CHLO	25533.77	-1.00	-1.00	-1.00	25533.77
O&G	-1.00	-1.00	-1.00	-1.00	-1.00
COD	-1.00	-1.00	-1.00	-1.00	-1.00
TL	-1.00	-1.00	-1.00	-1.00	-1.00
SB	-1.00	-1.00	-1.00	-1.00	-1.00

* -1 INDICATES DATA ARE NOT AVAILABLE OR APPLICABLE

10-06-1994
13:46:52

INFORMATION ON
EAST BANK PLANT

TOTAL POUNDS OF POLLUTANT COMPARED TO ACTUAL POUNDS/DAY *
USING A SAFETY FACTOR OF 0 %

POL.	ALLOWABLE TOTAL POUNDS/DAY	EXPANSION AND SAFETY FACTOR	ACTUAL UNCONTROL. POUNDS/DAY	ALLOWABLE INDUSTRIAL POUNDS/DAY	ACTUAL INDUSTRIAL POUNDS/DAY	REQUIRED IND. REMOVAL POUNDS/DAY
BOD	71256.96	0.00	42934.32	28322.65	4829.44	0.00
TSS	118761.60	0.00	59447.52	59314.08	30489.41	0.00
NH3	68804.75	0.00	330.26	68474.49	177.44	0.00
CD	43.98	0.00	0.76	43.22	0.45	0.00
CR	106.29	0.00	22.42	83.87	1.82	0.00
CU	320.21	0.00	20.51	299.71	9.33	0.00
CN	195.22	0.00	78.50	116.72	37.84	0.00
PB	102.38	0.00	4.16	98.22	8.41	0.00
HG	1.99	0.00	0.23	1.76	0.05	0.00
NI	340.43	0.00	0.03	340.40	3.14	0.00
AG	78.92	0.00	3.40	75.52	0.98	0.00
ZN	366.05	0.00	49.28	316.77	51.29	0.00
PHE	69.97	0.00	6.21	63.76	1.98	0.00
AS	26.59	0.00	0.03	26.55	1.03	0.00
BE	352.10	0.00	0.03	352.06	0.26	0.00
SE	18.67	0.00	0.03	18.64	0.05	0.00
CHLO	25533.77	0.00	0.03	25533.74	7.72	0.00
O&G	-1.00	-1.00	1651.32	-1.00	4960.35	-1.00
COD	-1.00	-1.00	66052.80	-1.00	21723.72	-1.00
TL	-1.00	-1.00	4.92	-1.00	0.16	-1.00
SB	-1.00	-1.00	0.03	-1.00	0.74	-1.00

* -1 INDICATES DATA ARE NOT AVAILABLE/APPLICABLE

UNIFORM CONCENTRATION INDUSTRIAL EFFLUENT LIMITS (MG/L) FOR EAST BANK PLANT
(APPLIES TO NON-DOMESTIC FLOW ONLY)

POLLUTANT	USING TOTAL INDUSTRIAL FLOW		USING ONLY IND. CONTRIBUTORY FLOWS*			BASIS	POLLUTANT
	W/O SAFETY FACTOR	W/ SAFETY FACTOR	W/O SAFETY FACTOR	W/ SAFETY FACTOR	CNTRIB FL (MGD)		
BOD	107.47	107.47	107.47	107.47	31.6000	D	BOD
TSS	200.00	200.00	200.00	200.00	31.6000	D	TSS
NH3	259.82	259.82	259.82	259.82	31.6000	W	NH3
CD	0.16	0.16	0.16	0.16	31.6000	S	CD
CR	0.32	0.32	0.32	0.32	31.6000	W	CR
CU	1.14	1.14	1.14	1.14	31.6000	S	CU
CN	0.44	0.44	0.44	0.44	31.6000	I	CN
PB	0.37	0.37	0.37	0.37	31.6000	S	PB
HG	0.01	0.01	0.01	0.01	31.6000	W	HG
NI	1.29	1.29	1.29	1.29	31.6000	S	NI
AG	0.29	0.29	0.29	0.29	31.6000	W	AG
ZN	1.20	1.20	1.20	1.20	31.6000	I	ZN
PHE	0.24	0.24	0.24	0.24	31.6000	W	PHE
AS	0.10	0.10	0.10	0.10	31.6000	S	AS
BE	1.34	1.34	1.34	1.34	31.6000	W	BE
SE	0.07	0.07	0.07	0.07	31.6000	S	SE
CHLO	96.89	96.89	96.89	96.89	31.6000	P	CHLO
O&G	@	@	@	@	31.6000		O&G
COD	@	@	@	@	31.6000		COD
TL	@	@	@	@	31.6000		TL
SB	@	@	@	@	31.6000		SB

* - INDUSTRY CONTRIBUTORY FLOW IS DEFINED AS TOTAL FLOW FROM THOSE INDUSTRIES CONTRIBUTING THE SPECIFIC POLLUTANT.

@ - INSUFFICIENT DATA AVAILABLE FOR DERIVATION OF A LOCAL LIMIT.

SEE USERS MANUAL FOR A MORE DETAILED EXPLANATION OF FOOTNOTES.

BASIS DEFINITION:

W - PASS THROUGH (WATER QUALITY)
P - PASS THROUGH (NPDES PERMIT)
I - INHIBITION
S - SLUDGE QUALITY
D - DESIGN LOADING**

** - BOD/TSS LOCAL LIMITS SET EQUAL TO POTW DESIGN LEVELS WHEN PRELIM MASS BALANCE DERIVES LESS STRINGENT LIMITS.

APPENDIX E
PROCESS CALCULATIONS

CALCULATION OF EXISTING AND FUTURE EBWWTP SLUDGE PRODUCTION

Present Sludge Waste Rate
From Existing Filter Belt Presses = 2114 tons/month @ 23% solids per 30 MGD flow*

= 69.5 tons/day @ 23% solids per 30 MGD flow

<u>Present Sludge Waste Rate to Existing Belt Filter Presses</u>

1% sludge at specific gravity = 1.00

$$\frac{16.98 \text{ tons}}{\text{day}} \quad \frac{1}{1.0\%} \quad \frac{2000 \text{ lbs}}{\text{tons}} \quad \frac{\text{cf}}{62.4 \text{ lbs}} \quad \frac{7.48 \text{ gallons}}{\text{cf}}$$

= 395671 gallons/day = 396000 gallons/day

Projected Sludge Waste Rate From Belt Filter Presses of the EBWWTP after additional 22,000 gpm stormwater flows.

$$= 16.98 \text{ tons/day} \ \frac{71 \text{ MGD}}{30 \text{ MGD}} \ \frac{113}{150}$$

= 30.2 tons/day

Projected Volume of sludge removed from various unit operations after additional 22,000 gpm stormwater flows.

$$= 39600 \text{ gals/day} \ \frac{71 \text{ MGD}}{30 \text{ MGD}} \ \frac{113}{150}$$

= 70,000 gals/day

CALCULATION OF UPGRADED ACTIVATED SLUDGE AERATION BASIN CAPACITY

ACTIVATED SLUDGE AERATION BASIN

TYPE : Conventional
Sludge Recycle rate : 50 % to 70 %
Dimensions : Width : 70 '
Depth : 15 '
Length : 270 '
No. of Basins : 4
Total Aeration Basin Volume : 8.48 MG
Converting four Primary Clarifier into Aeration Basin
Total Volume available for Activated Sludge Process
= 8.48 + 3.10
= 11.58 MG

Organic Loading :	Average Loading	49900 lbs/day
	Peak Loading	57000 lbs/day

Average Daily Flow = 71 MGD
Assume Peak Daily Flow = 110 MGD

DETENTION TIME

At Peak Daily Flow

V/Q = 2.52 hrs

At Average Daily Flow

V/Q = 4.00 hrs*

* (According to Design of Municipal Wastewater Treatment Plants Volume I {WEF Manual of Practice No. 8}, 1992 p. 627 : For Conventional Plug Flow Basin V/Q = 4 - 8 hrs.

FOOD TO MICROORGANISM RATIO

$$F/M = \frac{\text{Influent BOD5 Concentration}}{\text{Hydraulic detention time x MLVSS}}$$

MLSS is 2200-3000 mg/l

Assume MLSS is 80% MLVSS

MLVSS = 1760-2400 mg/l

O = Hydraulic detention time = 4 hrs = 0.166 days

CASE 1 39.4 MGD wastewater and 31.6 MGD stormwater

** Min F/M = 0.20 day^{-1}

** Max F/M = 0.27 day^{-1}

**Assuming MLVSS =1760 to 2400 mg/l. Influent BOD for wastewater and stormwater are estimated to be 130 mg/l and 18 mg/l , respectively. These values are based on past available data.

MEAN CELL RESIDENCE TIME
Assume 0.5 lbs MLVSS produced per lb BOD5

$$\text{Average BOD5} = 80.5 \text{ mg/l} \times 71 \text{ MGD} \times \frac{8.34 \text{ lbs}}{\text{MG. mg}} = 47667 \text{ lbs/day}$$

Assume that all MLVSS produced must be wasted from system

$$Qc = \frac{\text{lbs MLVSS in aeration basin}}{\text{lbs MLVSS produced per day}}$$

$$= \frac{(11.58 \text{ MG} \times 2400 \text{ mg/l}) \,.\, 8.34}{47667 \text{ lbs/day} \times 0.5}$$

$$= 9.7 \text{ days}$$

Qc = 9.7 days at average BOD loading and 20 % recycling

CASE 2 : 15 MGD wastewater and 56 MGD stormwater

"Min F/M ratio = 0.10 $days^{-1}$

"Max F/M ratio = 0.14 $days^{-1}$

"Assuming MLVSS =1760 to 2400 mg/l. Influent BOD for wastewater and stormwater are estimated to be 130 mg/l and 18 mg/l, respectively. These values are based on past available data.

EXISTING AIR FLOW REQUIREMENT CALCULATIONS

AT AVERAGE FLOW 35 MGD

Mass of BOD_L utilized = Q(So - S)/0.68*

$$\frac{[35 \text{ MGD } (130 - 9) \text{ mg/l} \,.\, 8.34 \text{ lbs}}{0.68 \qquad\qquad \text{MG mg/l}}$$

= 35,400 lbs/day

Carbon Oxygen Requirement

= 35,400 - 1.42 (Mass of organic wasted)
= 61,500 - 1.42 {Yobs x Q (So - S)}

Yobs = Y/(1 + kd $\Theta^d c$)

assume Y = O.5
Kd = .045
$\Theta^d c$ = 8.9 days

Yobs = 0.5/ [1 + (0.045 x 8.9)]
= 0.357

Mass of volatile waste activated sludge
= Yobs x Q (So - S)
= 8594 lbs/day

Carbon Oxygen Requirement
= 35,400 - 1.42 x 8,594
= 23,190 lbs/days

The activated sludge system is intended primarily for the removal of carbonaceous BOD. However it is assumed that nitrification will take place during the warm summer months. therefore oxygen requirements are calculated as a combined carbon-nitrogen oxygen requirements. Influent NH_3 values have never been measured at the EBWWTP, hence it is assumed that at the average flow NH_3 concentration in the wastewater will be 25 mg/l.

NITRO OXYGEN REQUIREMENTS

4.6 lbs O_2 / lbs NH_3 removed
= 4.6 (25.0 mg/l) x 35 MGD x 8.34 lbs/MG mg/l
= 33,568 lbs/day

Total Oxygen at Average Flow = 23,190 + 33,565
= 56,755 lbs/day

FLOW	AOR* (LBS/DAY)	SOR* (LBS/DAY)	SOTE (%)	SCFM REQUIRED
35 MGD	56,755	141,887	0.20	28,357

* Assume an alpha valve of 0.4.

BLOWER CAPACITY

Existing blowers are capable of supplying the volumes of air necessary to meet total oxygen demands (Carbonaceous and Nitrogenous)at average flows of 35 MGD.

CALCULATION OF ACTIVATED SLUDGE AERATION BASIN CAPACITY AFTER UPGRADES

ACTIVATED SLUDGE AERATION BASIN

TYPE : Conventional
Sludge Recycle rate : 50 % to 70 %
Dimensions : Width : 70 '
Depth : 15 '
Length : 270 '
No. of Basins : 4
Total Aeration Basin Volume : 8.48 MG
Converting four Primary Clarifier into Aeration Basin
Total Volume available for Activated Sludge Process
= 8.48 + 3.10
= 11.58 MG

Organic Loading : Average Loading 57745 lbs/day

Average Daily Flow = 71.2 MGD
Assume Peak Daily Flow = 110 MGD

DETENTION TIME

At Average Daily Flow

V/Q = 2.52 hrs

At Average Daily Flow

V/Q = 4.00 hrs*

* (According to Design of Municipal Wastewater Treatment Plants Volume I {WEF Manual of Practice No. 8}, 1992 p. 627 : For Conventional Plug Flow Basin V/Q = 4 - 8 hrs.

FOOD TO MICROORGANISM RATIO

$$F/M = \frac{\text{Influent BOD5 Concentration}}{\text{Hydraulic detention time x MLVSS}}$$

MLSS is 2200-3000 mg/l

Assume MLSS is 80% MLVSS

MLVSS = 1760-2400 mg/l

O = Hydraulic detention time = 4 hrs = 0.166 days

CASE 1 39.6 MGD wastewater and 31.7 MGD stormwater

** Min F/M = 0.28 day^{-1}

** Max F/M = 0.38 day^{-1}

**Assuming MLVSS =1760 to 2400 mg/l. Influent BOD for wastewater and stormwater are estimated to be 130 mg/l and 18 mg/l , respectively. These values are based on past available data.

MEAN CELL RESIDENCE TIME
Assume 0.5 lbs MLVSS produced per lb BOD5

$$\text{Average BOD5} = 80.5 \text{ mg/l} \times 71 \text{ MGD} \times \frac{8.34 \text{ lbs}}{\text{MG. mg}} = 47667 \text{ lbs/day}$$

Assume that all MLVSS produced must be wasted from system

$$Qc = \frac{\text{lbs MLVSS in aeration basin}}{\text{lbs MLVSS produced per day}}$$

$$= \frac{(11.58 \text{ MG} \times 2400 \text{ mg/l}) \,.\, 8.34}{47667 \text{ lbs/day} \times 0.5}$$

$$= 9.7 \text{ days}$$

Qc = 9.7 days at average BOD loading and 20 % recycling

CASE 2 : 15 MGD wastewater and 31.7 MGD stormwater

"Min F/M ratio = 0.12 day^{-1}

"Max F/M ratio = 0.17 day^{-1}

"Assuming MLVSS =1760 to 2400 mg/l. Influent BOD for wastewater and stormwater are estimated to be 130 mg/l and 18 mg/l , respectively. These values are based on past available data.

AIR FLOW
REQUIREMENT CALCULATION AFTER UPGRADES

AT AVERAGE FLOW 71 MGD

Mass of BOD_L utilized = Q(So - S)/0.68*

$$\frac{[39.4 \text{ MGD } (130 - 9) \text{ mg/l} + 31.6 \text{ MGD } (18 - 9) \text{ mg/l}]}{0.68} \cdot \frac{8.34 \text{ lbs}}{\text{MG mg/l}}$$

= 61,500 lbs/day

Carbon Oxygen Requirement

= 61,500 - 1.42 (Mass of organic wasted)
= 61,500 - 1.42 {Yobs x Q (So - S)}

Yobs = $Y/(1 + kd\ \Theta^d c)$

assume Y = O.5
Kd = .045
$\Theta^d c$ = 9.8 days

Yobs = 0.5/ [1 + (0.045 x 9.8)]
= 0.3469

Mass of volatile waste activated sludge
= Yobs x Q (So - S)
= 14507.3 lbs/day
Carbon Oxygen Requirement
= 61,500 - 1.42 x 14507.3
= 40,900 lbs/days

The activated sludge system is intended primarily for the removal of carbonaceous BOD. However it is assumed that nitrification will take place during the warm summer months. therefore oxygen requirements are calculated as a combined carbon-nitrogen oxygen requirements. Influent NH_3 values have never been measured at the EBWWTP, hence it is assumed that at the average flow NH3 concentration in the wastewater will be 30 mg/l. The NH_3 value in the stormwater is measured to be 0.26 mg/l.

NITRO OXYGEN REQUIREMENTS

4.6 lbs O_2 / lbs NH_3 removed
= 4.6 (14.0 mg/l) x 71 MGD x 8.34 lbs/MG mg/l
= 38,133 lbs/day

Total Oxygen at Average Flow = 40900 + 38,133
= 79,033 lbs/day

AT PEAK FLOW 110 MGD

110 MGD BOD5 = 58.2 mg/l

Mass of BOD_L utilized = Q(So - S)/0.68*

$$\frac{[39.4 \text{ MGD } (130 - 10) \text{ mg/l} + 70.4 \text{ MGD } (18 - 10) \text{ mg/l}] \ . \ 8.34 \text{ lbs}}{0.68 \qquad\qquad \text{MG mg/l}}$$

= 64,900 lbs/day

Carbon Oxygen Requirement

= 64,900 - 1.42 (Mass of organic wasted)
= 64,900 - 1.42 {Yobs x Q (So - S)}

Yobs = Y/(1 + kd $\Theta^d c$)

assume Y = O.5
Kd = .045
$\Theta^d c$ = 9.8 days

Yobs = 0.5/ [1 + (0.045 x 9.8)]
= 0.3469

Mass of volatile waste activated sludge
= Yobs x Q (So - S)
= 15310 lbs/day

Carbon Oxygen Requirement
= 64,900 - 1.42 x 15310
= 43,162 lbs/day

NITRO OXYGEN REQUIREMENTS

4.6 lbs O_2 / lbs NH_3 removed
= 4.6 (9.12 mg/l) x 110 MGD x 8.34 lbs/MG mg/l
= 38,486 lbs/day

Total Oxygen at Peak Flow = 43162 + 38,486
= 81648 lbs/day

FLOW	AOR* (LBS/DAY)	SOR* (LBS/DAY)	SOTE (%)	SCFM REQUIRED
71.2 MGD	81,648	204,120	0.20	40,787

$$SCFM = SOR \div SOTE \bullet \frac{ft^3}{.0749 lbs} \div (23.2\%)$$

(air density) (oxygen % of air)

* Assume on alpha valve of 0.4.

BLOWER CAPACITY

Existing blowers are capable of supplying the volumes of air necessary to meet total oxygen demands (Carbonaceous and Nitrogenous) for various design flows.

CALCULATION OF EXISTING ACTIVATED SLUDGE AERATION BASIN CAPACITY

ACTIVATED SLUDGE AERATION BASIN

TYPE : Conventional
Sludge Recycle rate : 50 % to 70 %
Dimensions : Width : 70 '
Depth : 15 '
Length : 270 '
No. of Basins : 4
Total Aeration Basin Volume : 8.48 MG

Organic Loading : Average Loading 49900 lbs/day
Peak Loading 57000 lbs/day

Average Daily Flow = 35 MGD

DETENTION TIME

At Average Daily Flow

V/Q = 5.81 hrs*

* (According to Design of Municipal Wastewater Treatment Plants Volume I {WEF Manual of Practice No. 8}, 1992 p. 627 : For Conventional Plug Flow Basin V/Q = 4 - 8 hrs.

FOOD TO MICROORGANISM RATIO

$$F/M = \frac{\text{Influent } BOD_5 \text{ Concentration}}{\text{Hydraulic detention time x MLVSS}}$$

MLSS is 2200-3000 mg/l

Assume MLSS is 80% MLVSS

MLVSS = 1760-2400 mg/l

Qc = Hydraulic detention time = 5.81 hrs = 0.24 days

f/m RATIO AT AVERAGE FLOW 35 MGD

** F/M = $0.225\ day^{-1}$

**Assuming MLVSS =1760 to 2400 mg/l. Influent BOD for wastewater is estimated to be 130 mg/l

MEAN CELL RESIDENCE TIME

Assume 0.5 lbs MLVSS produced per lb BOD5

$$\text{Average } BOD_5 = 130 \text{ mg/l} \times 35 \text{ MGD} \times \frac{8.34 \text{ lbs}}{\text{MG. mg}} = 37947 \text{ lbs/day}$$

Assume that all MLVSS produced must be wasted from system

$$Qc = \frac{\text{lbs MLVSS in aeration basin}}{\text{lbs MLVSS produced per day}}$$

$$= \frac{(8.48 \text{ MG} \times 2400 \text{ mg/l}) .\ 8.34}{37947 \text{ lbs/day} \times 0.5}$$

$$= 8.9 \text{ days}$$

Qc = 8.9 days at average BOD loading and 20 % recycling