REGIONAL WASTEWATER STUDY

FOR

CITY OF NEDERLAND
CITY OF PORT NECHES
CITY OF GROVES

FINAL REPORT

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EXECUTIVE SUMMARY

Schaumburg & Polk was retained by the Mid-County cities of Nederland, Port Neches, and Groves to conduct a study on their wastewater collection and treatment systems. The objective of this study was to establish whether or not regional wastewater treatment was in fact a viable alternative for the three cities as compared to the actions each city would need to take to solve its problems on an individual basis. In order to accomplish the objective of this study, Schaumburg & Polk has developed regional alternatives for treatment of wastewater for the three cities and has also developed individual alternatives for the three cities which would provide treatment of their wastewater for the next thirty years. After development of the various alternatives for wastewater treatment whether regional or individual, construction cost estimates were made for each alternative and the operation and maintenance cost for a thirty year period was established. As provided for in the scope of work, a matrix analysis comparing the various alternatives in different categories, including cost effectiveness and environmental impacts was developed. This matrix analysis is presented immediately following this Executive Summary. The recommendations of this study are in keeping with the preferred alternatives derived from the matrix analysis.

In areas experiencing fast growth, new development, and cities that are not land-locked, a regional study outlining how surrounding areas may be served with water and wastewater service and how those extensions of service may be financed, or paid for, are many times critical issues. However of the three Mid-County cities, Port Neches and Groves are both land-locked. They are also for the most part developed, and even those areas which are not fully developed have water and wastewater service available to them without the need for major collection system extensions. The City of Nederland has the opportunity for greater growth than the cities of Port Neches and Groves. However, Nederland's growth potential is not extraordinary. Also the collection system for the city of Nederland already serves its service area very comfortably. The portions of this report that deal with collection systems are somewhat abbreviated from what one might find for fast growing communities. Known problems with collection systems were investigated and recommendations for improvement which should be considered by the City of Nederland are presented. Although we do not make recommendations for collection system improvements for the City of Groves, our flow projections infer the need for wastewater treatment plants to handle larger peak hydraulic loads than the collection system is capable of delivering to the treatment plants.

The necessity for improvement of wastewater treatment in the Mid-County area is two-fold. This study was initially undertaken because in the course of renewing the cities' discharge permits (three of which are for discharge into Drainage District 7 drainage ditches), zero discharge permits were anticipated because of presumed high quality aquatic life use of these slow moving streams. Even with a variance to the surface water quality standards, and with the adoption of the new surface water quality standards in the summer of 1995, the allowable discharge limits for these plants are anticipated to be 5 mg/l BOD, 5 mg/l TSS, 3 ammonia mg/l, and a minimum 6 mg/l dissolved oxygen. Treatment to these levels is a great source of concern. The existing discharge limits are 20 mg/l BOD, 20 mg/l TSS. There is currently no ammonia limit for these treatment facilities. So it was a very good decision to look very closely at the options available to these cities when faced with the need for much more stringent treatment in order to maintain a discharge to their current receiving stream.

One of the alternatives which has proven to be viable for all three cities is to change the receiving stream to which they discharge from the DD7 drainage ditches to the Neches River. The Neches River is in relatively close proximity to all three cities and the TNRCC watershed management division modeling teams have indicated that 20/20 permits comparable to those now enjoyed by Mid-County would be possible for discharges to the Neches River. Several alternatives were developed with the idea of discharge to the Neches River rather than to the DD7 drainage ditch system. The regional system was sized to take peak thirty day average flows from all three cities as well as the combined peak wet weather flows from the three cities, treat it with an activated sludge process, and provide adequate solids handling facilities, as well as adequate hydraulic capacities for the handling for the wet weather peak. This alternative also includes the cost of lift stations and force mains to transport flows from the three cities to the regional facility site and from that site to the Neches River. Since the three cities are almost fully developed at this time, it was a challenge to identify an adequate site for construction of a regional facility, and in fact available property precluded us from considering constructed wetlands for a regional facility because of the size of the site required. We calculated the need of 1500 acres in order to treat wastewater from the three cities on a completely passive basis, and a single parcel of property of this size is simply not available within a reasonable distance or in reasonable location for the three cities.

After development of the regional facility concept, alternative approaches for each city were developed. Several different strategies for each city were considered and estimated, then the best alternatives for solving the cities' problems individually were then compared to the regional concept. We discovered that there is definitely an economy of scale available to the three cities through construction of a regional facility. However, because each city currently has substantial investment in individual facilities, and because these facilities can be upgraded and used in the future, we found the regional concept or the regional approach for the three cities is in fact not the most cost effective, or environmentally friendly method of accomplishing the three cities' goals of cost effective wastewater collection and treatment.

In performing a Regional Wastewater Study for the cities of Nederland, Port Neches, and Groves, Schaumburg & Polk, Inc. prepared numerous alternatives for wastewater treatment. Several of those alternatives were designed as regional facilities in which all three or two cities would cooperate in the collection and treatment of wastewater. Alternatives were also developed to address each city's individual needs.

Three City Regional Plant.

The regional facility developed to serve these three cities was located in Port Neches in an area convenient for discharge to the Neches River. This site was particularly difficult to identify since property adjacent to the river is very scarce. However a useable site was identified. The construction costs for the site are typical. There are no unusual construction conditions. And no environmental problems other than those typically dealt with are anticipated to be encountered. However, since each of the three cities now collects its wastewater at widely separated points, transporting each city's wastewater flow to the new treatment plant site was a substantial cost. We developed this alternative in anticipation of a 10/15/3 permit limit being required at some time in the future. We understand that 20/20 limits are available at the present time; however, over the course of the thirty year life of the facility, we anticipate that a 10/15/3 permit may be required. Anticipated construction cost for this facility is $36,278,000. Present value of thirty year operation and maintenance budget is $38,250,000, for a total present value of $74,528,000. These figures can be seen on Table 1. For comparison purposes, alternatives were developed to address each city's individual needs.

Port Neches

The City of Port Neches has expended considerable capital in the recent past to upgrade its plant; in fact, its current needs are not due to any deficiencies in wastewater treatment. The City's particular need is due to a change in discharge parameters brought about by a presumed high quality aquatic life use in its receiving stream. The Port Neches treatment plant does not require any upgrades; it is capable of handling peak flows and average daily flows at a 20/20 level of treatment. However, in order to maintain a 20/20 permit, it is necessary to divert the flow to discharge into the Neches River. The preferred alternative for accomplishing this is a joint lift station with the City of Groves which would pump average daily flows from each of these plants. We plan to negotiate, as part of their amended discharge permit, the ability to discharge peak wet weather flows, over and above ADF, into the existing receiving stream when high flows are occurring in the receiving stream. Capital cost for this alternative is $1,909,000. This alternative has a present value thirty year O & M cost of $1,665,000, for a total present value for this alternative of $3,574,000.

Nederland

The City of Nederland was evaluated using several scenarios. Its existing wastewater plant does in fact need to be upgraded. Nederland has had difficulty in consistently complying with its present discharge permits. We evaluated for the City of Nederland terminating use of the trickling filter portion of its plant, upgrading the remaining facilities to achieve 5/5 permit limits and continuing to discharge in the present location. We also evaluated upgrading the existing plant, discontinuing trickling filter treatment, and enhancing the activated sludge treatment to achieve treatment to 10/15 levels and construction of lift station and force main discharge to the Neches River. We evaluated utilizing the plant as is, taking the treated effluent to a wetland facility for polishing. Two alternatives were considered in this vein. One would discharge to Rhodair Gully (*alternately Johns Gully*); the other would discharge to a proposed Star Enterprise wetland. We also evaluated construction of a completely passive treatment system, whereby a lagoon would be followed by a free water service constructed wetland. The flow would then be discharged into Rhodair (*or Johns*) Gully. We also evaluated a minimal upgrade of the existing treatment facility, continuing to operate the existing trickling filters, and constructing a lift station to discharge average daily flows to the Neches River. We anticipated being able to negotiate discharge of peak wet weather flows into the current receiving stream.

Although construction of a completely passive system was just slightly more cost effective considering present value, the much lower capital cost of upgrading the existing plant and discharging to the Neches River makes this the preferred alternative. Also we anticipate many fewer environmental difficulties with this alternative. Also, the City is much more familiar with this type of treatment process and this particular upgrade can be enhanced to meet 10/15 limits by construction of a digester in the future at an estimated cost of just over $600,000. There is a fair amount of flexibility in this particular alternative. The present value of this alternative is $15,151,623. The capital cost of this alternative is $4,828,000.

Two City Regional Plant.

For the City of Groves we also evaluated a regional facility whereby the City of Groves and the City of Nederland would cooperate and construct a new wastewater treatment facility for their flows only. The capital cost for this facility was $33,284,000. We believe the reason the capital cost is so high is that Groves and Nederland are at opposite ends of the study area and transportation costs for getting their flows to a common site were extremely high. Present value of thirty year O&M for this alternative is $23,980,593, for a total present value cost of $57,264,593.

Groves

In addition to the regional alternative in cooperation with the City of Nederland, we looked at upgrading the City of Groves North and South Plants in various fashions. We considered constructing new treatment plants at each location, and in a new location, each of which would serve the entire City.

The most cost effective alternatives for the City of Groves, developed as a part of this study, include upgrading the City's existing North Plant to a flow of 1.99 million gallons per day with a peak flow of 6 million gallons per day by changing the rock media in the trickling filter to a synthetic media and constructing new primary and secondary clarifiers, new chlorine contact chamber, and the necessary solids handling facilities. Capital cost for these improvements are anticipated to be $4,093,000. The alternative selected for upgrading the South Plant is to upgrade the existing trickling filters by changing to synthetic media, along with construction of new primary and secondary clarifiers to accommodate peak flows of up to 18 million gallons a day, chlorine contact and dechlorination facilities, an effluent lift station, and the solids handling facilities necessary for this plant. Capital cost of these facilities are anticipated to be $8,448,000. Discharge from the South Plant on an average daily flow basis would be 3.33 million gallons per day, and would be discharged into the Sabine/Neches Ship Channel. Flows from the North Plant would be permitted for 1.99 million gallons per day and would be pumped from the joint lift station, operated cooperatively with the City of Port Neches, to the Neches River. We anticipate continuing 20/20 limits for each of these facilities for the foreseeable future. The design for the North Plant improvements will incorporate the ability to construct a solids contact unit for nitrification should surface water quality standards dictate higher levels of treatment in the future.

Summary

Table 1 can be found at the end of the Executive Summary section. It clearly indicates the alternatives selected as the most cost effective alternatives available to the cities of Port Neches, Nederland, and Groves. After developing the various alternatives for this study, Schaumburg & Polk believes that the reason a regional facility is not cost effective for these three cities is the fact that Port Neches does not need to construct wastewater treatment improvements. Port Neches only needs a pump station to divert its treated effluent from a receiving stream which would require treatment to 5/5 levels to the Neches River which will allow discharge at 20/20 permit levels for many years. Without the need for Port Neches to spend large sums of money on upgrading its facilities, it becomes very difficult to justify construction of new regional facilities for all three cities. Also, the City of Nederland having existing facilities which may be upgraded reduces the capital cost required for them. Nederland is in a poor location to act as a regional site for discharge to the Neches River, and the key for economic treatment of wastewater for these three cities is discharge to the Neches River. Utilization of the Nederland site as a regional facility is not possible. One reason is that its location is not convenient for River discharge, and two is because the site is landlocked and adequate area is not available. Groves is the only city which requires extensive upgrades of its facilities. The proposed regional facility is located in or near Groves. So there is very little more that could be done. We did consider construction of a regional facility adjacent to Groves North Plant and the Port Neches plant. However, sufficient land area is simply not available in this area to accommodate such a facility.

The recommendation of this study is that each city undertake action on its own behalf to gain compliance with current and future permits. The one area of cooperation between two cities that does hold promise is between the cities of Port Neches and Groves to cooperate in construction of a common lift station and force main to serve the Groves North treatment facility and the Port Neches treatment facility These facilities are next door to one another and it is economically feasible for this lift station and force main facilities to be constructed as a joint use facility.

Schaumburg & Polk has very much enjoyed working to develop the regional wastewater studies for the cities of Nederland, Port Neches, and Groves. The Steering Committee and the staffs of each city have been of immeasurable assistance in preparation of this document. We trust that because of the effort expended by the cities in preparation of this study by their involvement this study will be a benefit to them for many years to come.

This study is organized such that information for each city and the regional alternatives are presented each in a separate section of the report. There is one section of the report each for the City of Nederland, City of Groves, City of Port Neches, and one section deals with the issues of the regional facility. Certain issues common to all cities, including environmental issues, are covered in sections following the regional plant. The matrix analysis is located after this Executive Summary, and the supporting data and information are contained in the appendixes of the report.

Alternative	Technical Feasibility	Environmental Challenges	Capital Cost	Present Value '30yr O & M	Total PV Cost	Recommended Alternates
3 City Regional Facility	X	O	$36,278,000	$38,250,167	$74,528,167	
2 City Regional Facility (Groves, Nederland)	X	O	$33,284,000	$23,980,593	$57,264,593	
N-1 Upgrade Existing AS 5/5 Present Discharge	X	X	$10,002,000	$16,562,588	$26,564,588	
N-2 Upgrade Existing AS 10/15 Discharge to Neches River	X	O	$11,617,000	$18,353,585	$29,970,585	
N-3 Utilize Existing & Wetland Rhodair Gully	X	O	$11,786,000	$7,122,061	$18,132,061	
N-4 Utilize Existing & Wetland Star Enterprise	X	-	$10,300,000	$7,122,000	$17,422,000	
N-5 New Wetland Rhodair Gully	O	-	$14,182,000	$2,140,362	$14,910,362	
N-6 Upgrade Existing TF & AS 20/20 Neches River	X	O	$4,828,000	$10,323,623	$15,151,623	X
PN-1 All Flows	X	O	$2,461,000	$1,688,325	$4,149,325	
PN-2 ADF Only	X	O	$1,426,000	$899,545	$2,325,545	
PN/G-1 All Flows	X	O	$4,088,500	$2,491,916	$6,580,416	
PN/G-2 ADF Only	X	O	$1,909,000	$1,665,000	$3,574,000	X
PN/G-3 N & S ADF Only	X	O	$2,030,000	$1,800,000	$3,830,000	
G-1 North Plant TF 20/20	X	O	$4,093,000	$3,245,527	$7,338,527	X
G-2 North Plant AS 10/15	X	O	$5,273,000	$5,108,638	$10,381,638	
G-3 South Plant AS 10/15	O	-	$8,747,000	$8,449,610	$17,196,610	
G-4 South Plant TF 20/20	O	-	$8,448,000	$4,119,263	$12,567,263	X
G-5 Entire City @ North Location AS 10/15	X	O	$15,146,000	$12,302,243	$27,448,243	
G-6 Entire City @ South Location AS 10/15	O	-	$13,337,000	$10,629,797	$23,966,797	
G-7 Entire City @ 32nd Street TF 20/20	X	-	$14,337,000	$7,316,655	$21,653,655	
G-8 Lift Station for North Plant ADF Flows to Neches River	X	O	$1,200,000	$600,000	$1,800,000	

X = Positive
O = Neutral
- = Negative

SECTION 1-1 - NEDERLAND EXISTING WASTEWATER TREATMENT FACILITIES

A. CITY OF NEDERLAND WWTF

Plant Location: The plant site is located immediately east of the intersection of Hardy Avenue and Avenue D in Nederland, Jefferson County, Texas. The plant site is along the east side of Hardy Avenue and the southeast side of Main Ditch C (Jefferson County Drainage District No. 7); and approximately 2000 ft. NE of U. S. 69-96, 3700 ft. NW of Farm Road 365, and 1300 ft. SE of Nederland Avenue.

Receiving Stream: The discharge point is into Main Ditch C adjacent to the plant site, approximately 80 ft. NE of Hardy Avenue. From the plant site through a 36" pipe to Main Ditch C (Jefferson County Drainage District No. 7), then to Main Ditch B (Jefferson County DD 7), then to Main Outfall Canal (Jefferson County DD 7), then through Alligator Pump Station (Jefferson County DD 7) to Taylor Bayou (east distributary branch), then to the Intercoastal Waterway in Segment 0702 of Neches-Trinity Coastal Basin.

Discharge Permits: State - 10483-02
NPDES - TX0026476

Permit Limits: Maximum Monthly ADF = 3.8 mgd
Two-hour Peak = 8350 gpm (12.024 mgd)
20 mg/l BOD_5, 20 mg/l TSS
pH-6-9
D.O. = 2 mgl
Chlorination/Dechlorination
24 hr. and 48 hr. acute biomonitoring

No discharge beginning April 1, 1996; but City has variance allowing permit amendment based on new stream standards.

Existing Treatment Units and Sizes:

The Nederland plant contains three parallel treatment tracks between preliminary treatment and chlorination. Two of these tracks consist of identical contact stabilization plants, while the third track is a trickling filter process. Sludge is digested aerobically and dewatered with a centrifuge. Dried sludge is landfilled. A description of the existing treatment units and their respective capacities are included in APPENDIX A.

SECTION 1-2 - NEDERLAND WASTEWATER TREATMENT REQUIREMENTS

A. CITY OF NEDERLAND

	Baseline (1994)	5 years	30 years
Population[1]	17,650	18,674	19,127
Design ADF[2]	5.00 mgd	5.13 mgd	5.22 mgd
Design Peak Flow[2]	25.00 mgd	25.67 mgd	26.10 mgd

[1] *Population projection calculations are included in APPENDIX B.*
[2] *Present and projected flow calculations are included in APPENDIX B.*

As indicated below the existing treatment capacity of the City of Nederland WWTF is inadequate for both Present Design ADF and Present Design Peak Flow.

	Permit Limits	WWTF Capacity[3]	Present Need
ADF	3.8 mgd	1.53 mgd	5.00 mgd
Peak Flow	12.024 mgd	10.71 mgd	25.00 mgd

[3] *WWTF Capacity as per APPENDIX A May be somewhat higher when sludge thickener is considered.*

It is noted that the present ADF Capacity is limited by the capacity of the aerobic digesters; however, the ADF Capacity of the three (3) final clarifiers is 5.35 mgd and the ADF Capacity of the aeration units plus the trickling filters is 5.22 mgd.

The existing WWTF is designed to produce a secondary effluent of 20 mg/l BOD_5 and 20 mg/l TSS; however, it is expected that the WWTF will be required to meet effluent limits of 5 mg/l BOD_5, 5 mg/l TSS, 2 mg/l NH_3 and 6 mg/l D.O. in the future for continued discharge into the existing receiving stream.

Therefore, the existing WWTF is inadequate to meet future permit requirements and flow conditions.

SECTION 1-3 - NEDERLAND COLLECTION SYSTEM REQUIREMENTS

A. CITY OF NEDERLAND

The City of Nederland operates and maintains a gravity a collection system with some force main pumping. The collection system serves the City of Nederland, which encompasses approximately 2720 acres, in addition to some small outlying communities such as Parkway Village Mobile Home Park, located to the west of the City. The collection system is approximately 30 to 40 years old. It currently serves the entire City of Nederland with no areas within the City being without service. The collection is so configured that any additions to the City can be readily served by area trunk lines.

The analysis of the City of Nederland's collection system consisted of computer modeling of all trunk lines 21" and larger. Using specific manhole elevation data obtained from the City, a model of the existing trunk lines was created. Estimated peak flows were then introduced into each system and the resulting hydraulic conditions were analyzed.

With this model, it was possible to create surcharged conditions where manholes are known to overflow. This made it possible to analyze the performance of the overall system under what is known to be existing conditions. It also allowed for the determination of the capacity of each system.

1. 21" & 24" TRUNK LINE ALONG 27th STREET

Analysis: This system serves portions of west Nederland. It empties into a 30" sanitary sewer trunk line near the Waste Water Treatment Plant (WWTP). According to the computer model created for this system, the capacity is approximately 6.04 mgd. However, flows of this magnitude are only experienced under heavy wet weather conditions. There were no problem areas indicating overflowing manholes requiring improvements.

Improvements: NONE

2. 21" & 24" TRUNK LINE ALONG 36th STREET

Analysis: This system serves most of south Nederland. It empties into a 30" sanitary sewer trunk line near the WWTP. The model that was generated for this system indicates that the system has a capacity of approximately 3.30 mgd. The decrease in capacity of this system as opposed to the first system can be attributed to low natural ground elevations (shallow manholes) along the trunk line.

This trunk line runs from the WWTP to 36th St. at Nederland Ave. It then runs west along 36th St. and crosses Helena Ave. The area along 36th St. from Nederland Ave.

to Helena Ave. is low in elevation and natural ground is below the Hydraulic Grade Line. According to City officials, manholes in this area have overflowed and have, in fact, been bolted shut. The system surcharges and relieves itself farther upstream from Helena Ave. where natural ground rises in elevation. Therefore, the problem lies in the low natural ground elevations along the system.

Improvements:

No improvements are recommended for this system beyond sealing and securing all manhole lids on this trunk line along 36th St. from Nederland Ave. to west of Helena Ave., if they are not currently secured. In addition, a thorough Inflow and Infiltration Reduction Program should be implemented to reduce wet weather flows to the sanitary sewer collection system. This could include flow monitoring of selected manholes and a comprehensive smoke testing program covering all collection lines contributing to this trunk line.

3. 30" TRUNK LINE ALONG FM 365

Analysis: This system serves much of the northernmost portions of the City of Nederland. It has a capacity of approximately 8.65 mgd. However, in the portion of the system bounded by FM 365, SH 347, Ave. H, and a Drainage District #7 drainage ditch, the system is experiencing surcharged conditions with overflowing manholes, according to City officials. This was further confirmed by the hydraulic model created for this system.

This area is exceptionally low, causing the Hydraulic Grade Line to rise above natural ground. During wet weather flow the manholes in this neighborhood overflow. However, farther upstream on the trunk line north of SH 347, no overflowing manholes have been detected because natural ground in this area is high. Because the trunk line adequately serves the area for which it was intended, with the exception of the area mentioned above, no replacement or improvements to the trunk line should be made. However, the aforementioned area should be served by an alternate means of sewage collection and taken off the trunk line to prevent the surcharged conditions in the problem area.

Improvements:

The neighborhood mentioned above should be served by another means other than the existing trunk line. A pump station and force main is proposed for serving this area. In addition, a small series of gravity sanitary sewer collection lines and manholes is proposed. In addition, all manhole lids on the trunk line from 27th St. to north of SH 347 should be sealed and secured to the manholes.

SECTION 1-4 - NEDERLAND PROPOSED ALTERNATIVES

A. GENERAL

Present, 15 year, and 30 year flow projections do not vary more than approximately 5%; therefore, the proposed alternatives have been analyzed based on the projected requirements for 30 years in the future.

B. CITY OF NEDERLAND

1. Wastewater Treatment Needs and Alternatives

Several wastewater treatment alternatives were analyzed for the City of Nederland including upgrading the existing treatment facility, diverting the discharge to the Neches River, and construction of a wetland system both for polishing of existing effluent and for full treatment. A summary of each alternative is provided below and a detailed analysis of each alternative is included in APPENDIX D.

a. Alternate N1. This alternate proposes to upgrade the existing WWTF for continued discharge into the existing receiving stream at effluent limits of 5 mg/l BOD_5, 5 mg/l TSS, 2 mg/l NH_3, and 6 mg/l DO.

Opinion of Probable Construction Cost = $10,002,000

Annual O & M Costs = $880,363

Advantages: WWTF will remain at existing site.

Disadvantages: To meet the proposed effluent limits will require an extremely intensive operating program, and any upset within the system could likely result in non-compliance with permitted effluent limits; requires significant amount of land near residential area.

b. Alternate N2. This alternate proposes to upgrade the existing WWTF and divert the discharge to the Neches River at effluent limits of 20 mg/l BOD_5, 20 mg/l TSS, and 4 mg/l DO. *(Alternately, divert all flows up to design ADF to Neches River and discharge excess flows into drainage ditch during high ditch flows from wet weather).* This facility would be capable of meeting 10/15/3 permit limits.

Opinion of Probable Construction Cost = $11,617,000 *($8,883,000)*

Annual O & M Costs = $1,044,796 *($983,023)*

Advantages: WWTF will remain at existing site.

Disadvantages: May require pumping all effluent (ADF and Peak Flow) approximately 4.5 miles to the Neches River. *Possibly will only require that ADF be pumped to the Neches River and allow Peak Flows in excess of ADF to be discharged into current receiving stream.*

c. Alternate N3. This alternate proposes to continue to operate the existing WWTF, construct a transfer lift station/force main to a proposed surface flow constructed wetland to polish the effluent from the existing WWTF, and then discharge into *Rhodair Gully* (or Johns Gully pending site availability) at effluent limits of 10 mg/l BOD_5, 15 mg/l TSS, 3 mg/l NH_3, and 6 mg/l DO.

Opinion of Probable Construction Cost = $11,786,000 *($9,348,000)*

Annual O & M Costs = $610,727

Advantages: Existing WWTF will not require extensive improvements.

Disadvantages: Complexities of existing WWTF will not be eliminated. Will require operation of two separate treatment facilities. Will require all flows to be pumped twice, first into the existing WWTF and then to the wetland facility. TNRCC may not approve operation of existing WWTF as proposed. Proposed constructed wetland will require approximately 200 acres of land. Current TNRCC Design Criteria requires that the design of any wetland proposed for nitrification below 5 mg/l shall incorporate a separate nitrification process. A variance to this requirement will have to be obtained for approval of this Alternate. *Availability of Site 'A' is questionable, and Site 'B' is located further away from the existing WWTF.*

d. Alternate N4. This alternate, similar to N3, proposes to continue to operate the existing WWTF, construct a transfer lift station/force main to a proposed surface flow constructed wetland shared with STAR Enterprise Port Arthur Plant to polish the effluent from the existing WWTF, and then discharge to STAR Enterprise for industrial reuse at effluent limits of 10 mg/l BOD_5, 15 mg/l TSS, 3 mg/l NH_3, and 6 mg/l DO.

Opinion of Probable Construction Cost[1] = $10,300,000 (Nederland Share)

Annual O & M Costs[1] = $610,727 (Nederland Share)

1 COSTS ARE NEDERLAND TRANSFER PUMPING $ & EXISTING WWTF UPGRADES $ + FLOW PROPORTIONAL (5.22/15.22) $ FOR CONSTRUCTED WETLAND. O&M COSTS DO NOT INCLUDE EFFLUENT PUMPING.

Advantages: Existing WWTF will not require extensive improvements.

Disadvantages: Complexities of existing WWTF will not be eliminated. Will require operation of two separate treatment facilities, one of which will require joint operation. Will require all flows to be pumped twice, first into the existing WWTF and then to the wetland facility. TNRCC may not approve operation of existing WWTF as proposed. Proposed constructed wetland will require approximately 600 acres of land. Proposed site at Hwy. 69 and Hwy. 73 may be insufficient for wetlands, requiring additional property acquisition or dividing of wetlands north and south of Hwy. 73. Proposed site appears to be wetlands which will require mitigation. Mitigation costs (i.e., conversion of at least 3 times existing natural wetland area) can be cost prohibitive. EPA may not allow construction on natural wetlands. Current TNRCC Design Criteria requires that the design of any wetland proposed for nitrification below 5 mg/l shall incorporate a separate nitrification process. A variance to this requirement will have to be obtained for approval of this Alternate. Possible treatment problems may occur due to mixing of industrial and municipal wastewater.

e. Alternate N5. This alternate proposes to abandon the existing WWTF, convert the existing influent lift station to a transfer lift station, construct a force main to pump the raw wastewater to a proposed facultative lagoon/surface flow constructed wetland for full treatment of all flows and then discharge into *Rhodair Gully* (or Johns Gully pending site availability) at effluent limits of 10 mg/l BOD_5, 15 mg/l TSS, 3 mg/l NH_3, and 6 mg/l DO.

Opinion of Probable Construction Cost = $14,182,000 *($11,410,000)*

Annual O & M Costs = $189,923

Advantages: Will eliminate future operation of existing mechanical WWTF. Annual O & M Costs are very low in comparison to other alternative treatment.

Disadvantages: Will require approximately 300 acres of land. Current TNRCC Design Criteria requires that the design of any wetland proposed for nitrification below 5 mg/l shall incorporate a separate nitrification process. A variance to this requirement will have to be obtained for approval of this Alternate. *Availability of Site 'A' is questionable, and Site 'B' is located further away from the existing WWTF.*

f. Alternate N6. This alternate proposes to upgrade the existing WWTF, and construct a lift station and force main to the Neches River anticipating a discharge permit of 20 mg/l BOD_5 and 20 mg/l TSS. Peak flows would be treated, but discharged to the present receiving stream, while flows up to 4.76 MGD (5.22 MGD future) would be pumped to the Neches River.

Opinion of Probable Construction Cost = $4,828,000

Annual O & M Cost = $750,000.00

Advantages: Achieves discharge to a receiving stream that will allow construction of a 20/20 discharge permit. Lowest capital cost alternative.

Disadvantages: Continues operation of dual process treatment plant.

2. Collections System Needs and Alternatives

a. 21" & 24" TRUNK LINE ALONG 27TH STREET

No improvements are recommended for this trunk line.

b. 21" & 24" TRUNK LINE ALONG 36TH STREET

No major system improvements are recommended for this trunk line. However, an extensive Inflow and Infiltration reduction program should be implemented as outlined in the preceding section.

c. <u>30" TRUNK LINE ALONG FM 365</u>

An alternate means of collection for the portion of the collection system bounded by FM 365, SH 347, Ave. H, and a Drainage District #7 drainage ditch is needed. A pump station is proposed to transfer the flows to the Wastewater Treatment Plant. Flows for this pump station were estimated by determining acreage of the above mentioned area and dividing it by the overall acreage of the City of Nederland's collection system and multiplying this ratio by the projected 2-hour peak flow. Therefore, the proposed pump station will have firm capacity of one (1) mgd (695 gpm). Three 350 gpm pumps are proposed. In addition, approximately 9100 linear feet of 10" force main will be required along with some improvements to the existing collection system in the specified area in order to divert all flows to the proposed pump station. Because the proposed route of the force main will be along FM 365, there will be several bores including one with casing at 27th St.

OPINION OF PROBABLE COSTS:

1.	Proposed Pump Station	$ 74,590.00
2.	400 L.F. 6" San. Swr.	$ 4,000.00
3.	3700 L.F. 8" San. Swr.	$ 44,400.00
4.	9 San. Swr. MH's	$ 10,800.00
5.	7200 L.F. 10" Force Main	$ 180,000.00
6.	100 L.F. Bore & Case at 27th St.	$ 13,500.00
7.	1800 L.F. Bore w/no Case for Various Drives	$ 81,000.00
8.	Bolt & Seal 25 San. Swr. MH's	$ 7,500.00
	Sub-Total	$ 415,790.00
	15% Contingency	$ 62,369.00
	Total Costs	$ 478,159.00

SECTION 1-5 - NEDERLAND ENFORCEMENT ACTION STATUS

The City has the following enforcement actions active or pending:

TNRCC: On December 13, 1994 the TNRCC staff issued formal notice to the City of a proposed Enforcement Order with administrative penalties, recommending that the City reach an agreement with the TNRCC for an Agreed Enforcement Order. The notice cited the City for unauthorized discharges of wastewater through a manhole overflow line; for various violations of plant effluent quality including suspended solids and chlorine residual; and for inadequate solids management including a waste stream *(from the water treatment plant)* with a high solids content.

Requirements in the proposed order included infiltration/inflow mitigation measures; interim measures to mitigate the effects of the unauthorized discharge pending elimination; remediation of the receiving stream and affected property; a preventive maintenance plan; an engineering assessment of the treatment plant; a solids management plan; a system for responding to citizen complaints of unauthorized discharges; and notification to all sewer customers regarding the order.

The City has requested and attended a hearing in regard to the proposed order, and negotiation of the final Agreed Enforcement Order is pending. Meanwhile, the City has had a solids management plan prepared as required by the order. The requirement for the engineering assessment of the plant can be satisfied with Appendix A1 of this report. The requirement for a preventive maintenance plan is partially addressed in a report recently submitted to the EPA as discussed below.

EPA: The City is under an Administrative Order, Docket No. VI-95-1212 *(January 31, 1995)*, citing various violations of BOD_5, TSS, and chlorine residual requirements in the plant effluent as well as the recurrent manhole overflow noted above. Requirements include an operation and maintenance plan for the treatment plant; a plan addressing the I/I problem and the manhole overflow line; and a summary of the recommendations in this report regarding treatment plant improvements or new plant construction.

The City recently submitted a response package to the EPA addressing the items above. It should be noted that any final response to the last item must include a schedule for treatment plant construction, and that according to typical agency practice the City can expect the EPA to incorporate the schedule into a new order superseding the current order.

SECTION 1-6 - NEDERLAND: OTHER INFORMATION

A. WATER SUPPLY

The City draws water from the local LNVA canal system, then treats it in a City plant. The City's contract with the LNVA does not specify an upper limit of usage. The City anticipates continuing its existing water supply practice for the entire 30 year study period. See the separately bound Water Conservation Plan for further information.

B. SLUDGE MANAGEMENT

Under present circumstances the City would continue to send all sludge to a Class I landfill for codisposal regardless of the amounts of sludge generated under the various alternatives.

Although state and federal policies on sludge disposal nominally encourage beneficial use of municipal sludge, the corresponding standards for sludge quality in practice make beneficial use unfeasible for the Southeast Texas area. To the knowledge of the Engineer, only one site in Southeast Texas has been registered, north of Orange, and the site owner has terminated his contract with at least one community in recent years. The nearest known registered site is located in Tyler County over 50 miles away from Nederland and reportedly accepts only liquid sludge, for which it charges a fee. It appears that practices such as land application are proving feasible only for large metropolitan areas such as Houston, where use of the sludge is more attractive economically.

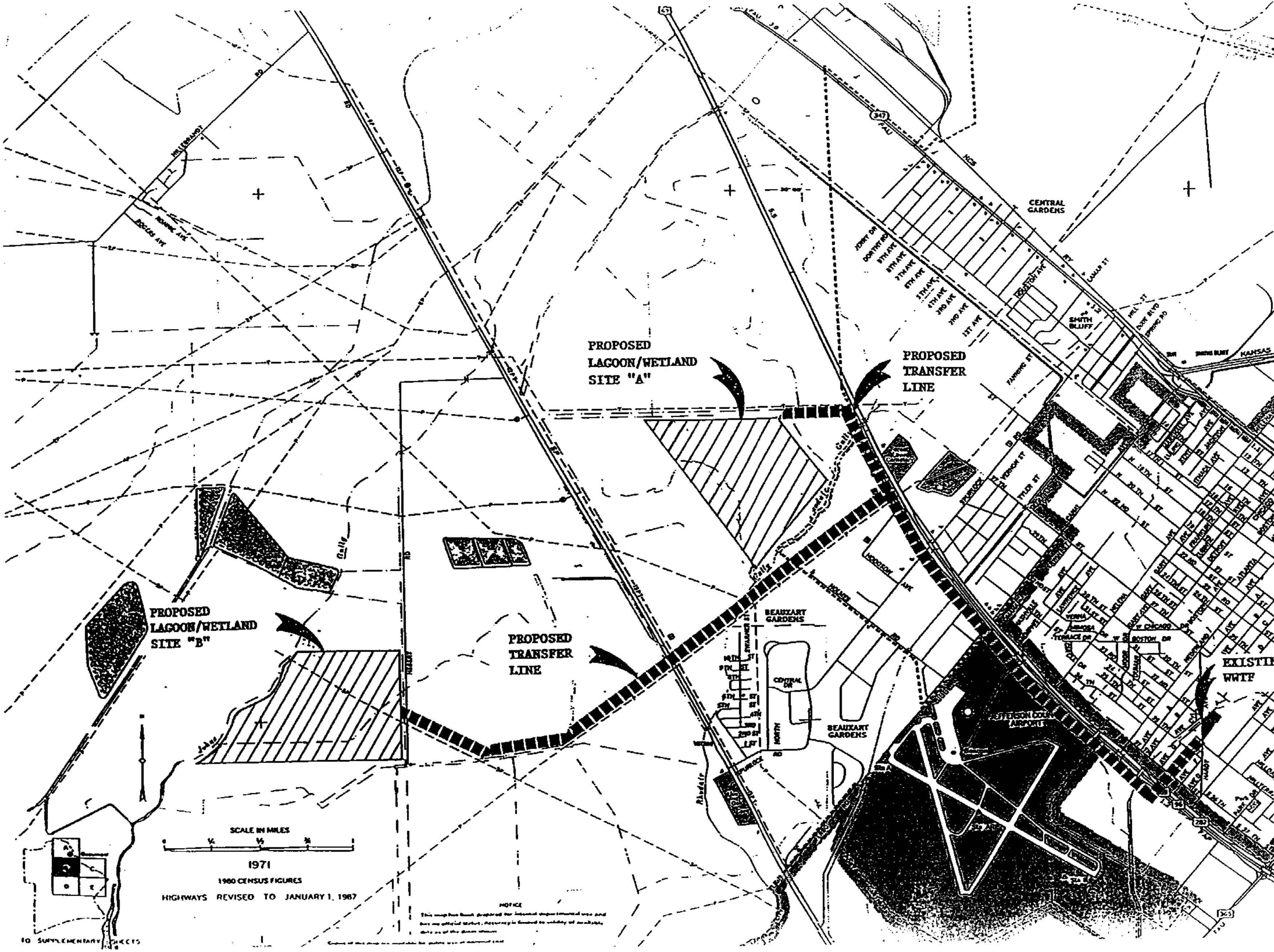
The City presently sends its sludge in dewatered form to a commercial landfill south of Beaumont on LaBelle Road, operated by Browning-Ferris Industries. According to the Engineer's conversation with the landfill manager (7/5/95), the presently permitted landfill facility is expected to have capacity through the year 2030, or several years beyond the study period.

C. PROJECT SCHEDULE

It is impractical to develop a realistic schedule for project implementation at this time, since the City must continue to assess its changing situation before implementing its project. One of the prime reasons for needing the project is the upgrading several years ago of the stream standards for the drainage ditch system into which the plant discharges.

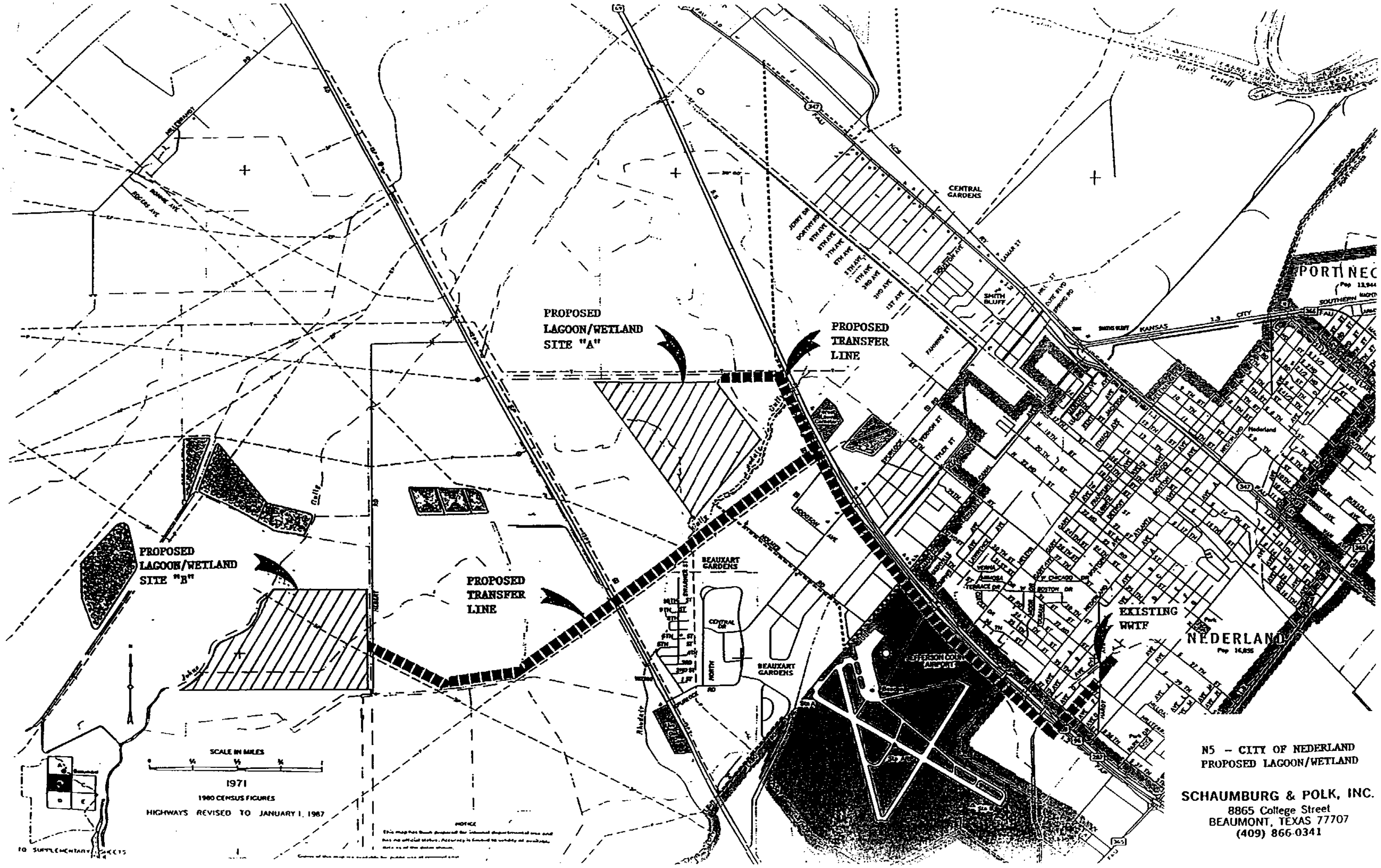
The present TNRCC permit for Nederland provides for no discharge into the present receiving stream beginning April 1, 1996. However, the permit contains a variance reflecting the probability that the TNRCC will adopt slightly relaxed stream standards for the receiving streams. Since the TNRCC actually adopted the revised standards on June 14, 1995, the City can now apply for a permit amendment reflecting the revised standards. Such an amendment would grant additional time for compliance with the new standards *(or for alternate measures* *such as diversion to another stream)* and in all probability also relax the requirements for continued discharge into the stream.

In light of the recent revisions to TNRCC stream standards, it appears that the next step for Nederland may be to request new stream modelling from the TNRCC staff to verify what new effluent standards would apply to the appropriate design flows for continued discharge into the existing stream. At the same time, the City could also request a determination regarding the discharge of excess storm flows into the existing stream in periods of high stream flows. After receiving responses from the TNRCC staff *(assuming the responses to be consistent with the assumptions used in the report)*, the City could then begin implementing the project recommended in the report. This implementation would begin with the appropriate permit amendment application and/or with the necessary SRF engineering studies for TWDB financing. The SRF study, should the City pursue SRF financing, would in itself contain an implementation schedule.



N5 - CITY OF NEDERLAND
PROPOSED LAGOON/WETLAND

SCHAUMBURG & POLK, INC
8865 College Street
BEAUMONT, TEXAS 77707
(409) 866-0341



N5 - CITY OF NEDERLAND
PROPOSED LAGOON/WETLAND

SCHAUMBURG & POLK, INC.
8865 College Street
BEAUMONT, TEXAS 77707
(409) 866-0341

SECTION 2-1 - PORT NECHES EXISTING WASTEWATER TREATMENT FACILITIES

A. CITY OF PORT NECHES WWTF

Plant Location: The plant site is located in the extreme southwest portion of the City adjacent to the City of Groves, 1 mile northwest of the intersection of State Highway 347 and State Highway 73 in the 6100 block of Georgia street in Jefferson County, Texas.

Receiving Stream: The discharge point is into a concrete lined Jefferson County Drainage District No. 7 (DD7) drainage canal; thence to DD7 Canal A; thence to Alligator Bayou; thence to Taylor Bayou; thence to DD7 Main Outfall Canal; thence to the Intracoastal Waterway in Segment 0702 of the Neches-Trinity Coastal Basin.

Discharge Permits: State - 10477-004
NPDES - TX0022926

Permit Limits: Maximum Monthly ADF = 4.98 mgd
Two-hour Peak = 6250 gpm (9.0 mgd) for main units
Storm Water Clarifiers = 17.0 mgd two hr. peak
BOD_5 = 20 mg/l
TSS = 20 mg/l
pH = 6 to 9
D.O. = 5 mg/l
Chlorination/Dechlorination
Chronic and Acute Biomonitoring
Copper = 65 ug/l (NPDES Permit only)

No discharge beginning May 1, 1997; but City has variance allowing permit amendment based on new stream standards.

Existing Treatment Units and Sizes:

The Port Neches plant utilizes the fixed film treatment process for treatment of the wastewater flows. The major wastewater treatment units consist of the headworks including a comminutor, manually cleaned bar screen, and flow measuring device; aerated grit basin including a grit classifier; primary clarifier; trickling filters including one (1) primary and two (2) secondary; two (2) final clarifiers; two (2) stormwater clarifiers; and chlorination facilities. Under normal operation, the two stormwater clarifiers follow the two final clarifiers. During storm flow, several automatic gates and valves divert normal plant flow around the stormwater clarifiers. The stormwater flows are directed around the main treatment units to the stormwater clarifiers for solids settling prior to disinfection and discharge. Sludge treatment units consist of the grit classifier; primary and secondary digesters; drying beds; and belt press. Dried sludge is disposed of at a landfill. A description of the existing treatment units and their respective capacities are included in APPENDIX A.

SECTION 2-2 - PORT NECHES WASTEWATER TREATMENT REQUIREMENTS

A. CITY OF PORT NECHES

	Baseline (1994)	15 years	30 years
Population[1]	13,479	14,517	15,040
Design ADF[2]	3.64 mgd	3.82 mgd	3.88 mgd
Design Peak Flow[2]	20.02 mgd	21.01 mgd	21.34 mgd

[1] *Population projection calculations are included in APPENDIX B.*
[2] *Present and projected flow calculations are included in APPENDIX B.*

As indicated below the existing treatment capacity of the City of Port Neches WWTF is adequate for both Present Design ADF and Present Design Peak Flow.

	Permit Limits	WWTF Capacity[3]	Present Need
ADF	4.98 mgd	4.98 mgd	3.64 mgd
Peak Flow	26.0 mgd	25.13 mgd[4]	20.02 mgd

[3] *WWTF Capacity as per APPENDIX A.*
[4] *9.05 mgd for main units, 16.08 mgd for stormwater clarifiers.*

It should be noted that the Peak Flow Capacity is currently limited by capacity of the final clarifier plus the two storm water clarifiers. Although the flows through the stormwater clarifiers are presently chlorinated in the clarifiers, this may not be allowed in the future. The final clarifier and stormwater clarifiers were apparently previously approved for a total Peak Flow Capacity of 26.0 mgd.

The existing WWTF is designed to produce a secondary effluent of 20 mg/l BOD_5 and 20 mg/l TSS; however, it is expected that the WWTF will be required to meet effluent limits of 5 mg/l BOD_5, 5 mg/l TSS, 2 mg/l NH_3 and 6 mg/l D.O. in the future for continued discharge into the existing receiving stream.

Therefore, the existing WWTF is inadequate to meet future permit requirements.

SECTION 2-3 - PORT NECHES COLLECTION SYSTEM REQUIREMENTS

A. <u>CITY OF PORT NECHES</u>

The collection system for the City of Port Neches consists primarily of gravity sanitary sewer lines with pumping stations at various locations. In general, the collection system adequately serves the City of Port Neches. There are not any areas for which service is not available. The collection system itself is approximately 30 to 40 years old. The system is configured so as to allow new areas to be added without major upgrades to the collection system. Additions to the collection system which require a pump station can be readily accommodated by the layout of the existing collection system.

SECTION 2-4 - PORT NECHES PROPOSED ALTERNATIVES

A. GENERAL

Present, 15 year, and 30 year flow projections do not vary more than approximately 5%; therefore, the proposed alternatives have been analyzed based on the projected requirements for 30 years in the future.

B. CITY OF PORT NECHES

1. Wastewater Treatment Needs and Alternatives

The existing WWTF is capable of treating design ADF and Peak Flows to typical secondary treatment limits (20 mg/l BOD_5, 20 mg/l TSS); however, proposed future effluent limits for the existing receiving stream will be considerably lower than these existing limits. Therefore, the City of Port Neches would have to either upgrade the existing WWTF to meet the proposed future limits or divert its effluent to a receiving stream with the existing secondary limits.

Therefore, construction of an individual lift station and force main for diverting the discharge to the Neches River was analyzed. A summary of this alternative is provided below and a detailed analysis is included in APPENDIX D.

Because the City of Groves' North WWTF is located directly adjacent to the Port Neches WWTF and discharges into the same receiving stream, that WWTF must also upgrade to meet the proposed future limits or divert its effluent to a receiving stream with the existing secondary limits.

Therefore, construction of a common lift station and force main(s) for diverting the discharges to the Neches River was also analyzed. A summary of this alternative is provided below and a detailed analysis is included in APPENDIX D.

a. Alternates PN-1 and PN-2. These alternates proposed to construct an effluent lift station and force main for the City of Port Neches WWTF to divert the discharge to the Neches River at effluent limits of 20 mg/l BOD_5, 20 mg/l TSS, and 4 mg/l DO. Alternate PN-1 is for all flows to be diverted to the river. *Alternate PN-2 is for all flows up to the design ADF to be diverted to the river with excess flows discharged into the existing receiving stream during wet weather flows in that stream.*

Opinion of Probable Construction Cost = $2,461,000

($1,426,000)

Annual O & M Cost = $121,203
($65,351)

Advantages: Will not require upgrading the existing Port Neches WWTF.

Disadvantages: May require pumping all effluent (ADF and Peak Flow) approximately 3-3.5 miles to the Neches River. *Possibly will only require that ADF be pumped to the Neches River and allow Peak Flows in excess of ADF to be discharged into current receiving stream.*

b. Alternates PN/G1 and PN/G2. These alternates propose to construct a common effluent lift station and force mains for the City of Port Neches WWTF and the City of Groves North WWTF to divert the respective discharges to the Neches River at effluent limits of 20 mg/l BOD_5, 20 mg/l TSS, and 4 mg/l DO. Alternate PN/G-1 is for all flows to be diverted to the river. *Alternative PN/G-2 is for all flows up to the design ADF to be diverted to the river, with excess flows discharged into the existing receiving storm during wet weather flows in that stream.*

Opinion of Probable Construction Cost = $4,088,500
($1,909,000)

Annual O & M Cost = $181,035
($120,937)

Advantages: Will not require upgrading the existing Port Neches WWTF.

Disadvantages: May require pumping all effluent (ADF and Peak Flow) approximately 3-3.5 miles to the Neches River. *Possibly will only require that ADF be pumped to the Neches River and allow Peak Flows in excess of ADF to be discharged into current receiving stream.*

c. Alternate PN/G3. This alternative is basically the same as PN/G-2; however, Alternate PN/G-3 is to construct an effluent lift station and force main(s) to serve both the City of Port Neches WWTF and a

Regional Groves WWTF located at the North Plant Site.

Opinion of Probable Construction Cost = $2,030,000

Annual O & M Cost = $130,768

Advantages: Will not require upgrading the existing Port Neches WWTF.

Disadvantages: May require pumping part of effluent (ADF only) approximately 3-3.5 miles to the Neches River.

SECTION 2-5 - PORT NECHES: OTHER INFORMATION

A. ENFORCEMENT ACTION STATUS

The City is not presently under any TNRCC or EPA enforcement action.

B. WATER SUPPLY

The City draws water from the local LNVA canal system, then treats it in a City plant. The City's contract with the LNVA does not specify an upper limit of usage. The City anticipates continuing its existing water supply practice for the entire 30 year study period. See the separately bound Water Conservation Plan for further information.

C. SLUDGE MANAGEMENT

Under present circumstances the City would continue to send all sludge to a Class I landfill for codisposal regardless of the amounts of sludge generated under the various alternatives.

Although state and federal policies on sludge disposal nominally encourage beneficial use of municipal sludge, the corresponding standards for sludge quality in practice make beneficial use unfeasible for the Southeast Texas area. To the knowledge of the Engineer, only one site in Southeast Texas has been registered, north of Orange, and the site owner has terminated his contract with at least one community in recent years. The nearest known registered site is located in Tyler County over 50 miles away from Port Neches and reportedly accepts only liquid sludge, for which it charges a fee. It appears that practices such as land application are proving feasible only for large metropolitan areas such as Houston, where use of the sludge is more attractive economically.

The City presently sends its sludge in dewatered form to a commercial landfill south of Beaumont on LaBelle Road, operated by Browning-Ferris Industries. According to the Engineer's conversation with the landfill manager (7/5/95), the presently permitted landfill facility is expected to have capacity through the year 2030, or several years beyond the study period.

D. PROJECT SCHEDULE

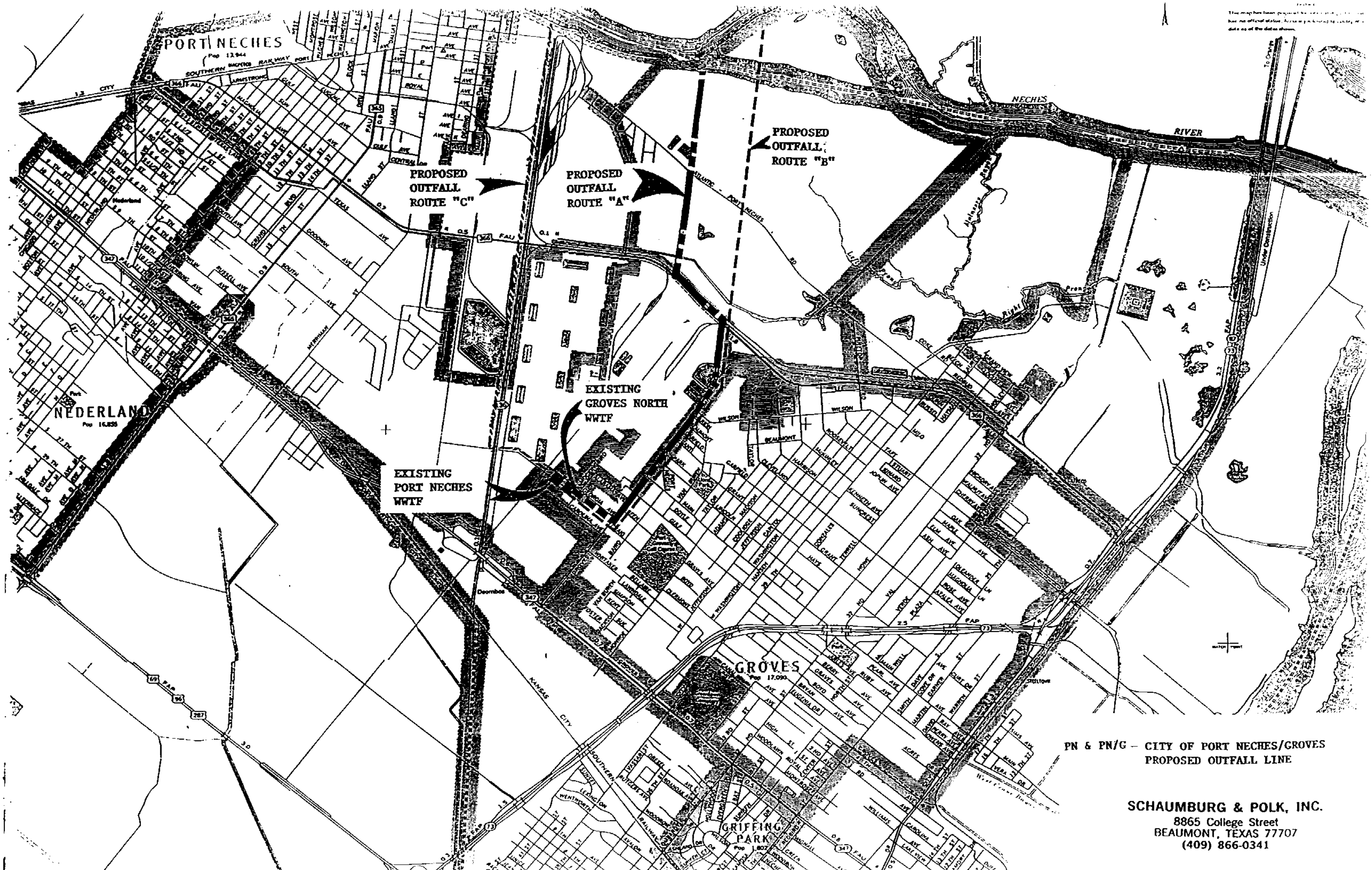
It is impractical to develop a realistic schedule for project implementation at this time, since the City must continue to assess its changing situation before implementing its project. One of the prime reasons for needing the project is the upgrading several years ago of the stream standards for the drainage ditch system into which the plant discharges.

The present TNRCC permit for Port Neches provides for no discharge into the present receiving streams beginning May 1, 1997. However, the permit contains a variance reflecting the probability that the TNRCC will adopt slightly relaxed stream standards for the receiving streams. Since the TNRCC actually adopted the revised standards on June 14, 1995, the city can now apply for a permit amendment reflecting the revised standards. Such an amendment would grant additional time for compliance with the new standards *(or for alternate measures such as diversion to another stream)* and in all probability also relax the requirements for continued discharge into the stream.

In light of the recent revisions to TNRCC stream standards, it appears that the next step for Port Neches may be to request new stream modelling from the TNRCC staff to verify what new effluent standards would apply to the appropriate design flows for continued discharge into the existing stream. At the same time, the city could also request a determination regarding the discharge of excess storm flows into the existing stream in periods of high stream flows. After receiving responses from the TNRCC staff *(assuming the responses to be consistent with the assumptions used in the report)*, the City could then begin implementing the project recommended in the report. This implementation would begin with the appropriate permit amendment application. The amended TNRCC permit would in itself contain an implementation schedule.

Before submitting a permit application, the City should first confirm with the City of Groves whether Groves still plans to construct a plant at its North Plant location and route the flows from that plant to the river. Once that determination is made, Port Neches can begin the permit amendment process *(reflecting whether the outfall to the river would carry flows from one or both cities)* and other project implementation.


PN & PN/G – CITY OF PORT NECHES/GROVES
PROPOSED OUTFALL LINE
SCHAUMBURG & POLK, INC.
8865 College Street
BEAUMONT, TEXAS 77707
(409) 866-0341
NECHES RIVER
PROPOSED OUTFALL ROUTE "B"
PROPOSED OUTFALL ROUTE "A"
PROPOSED OUTFALL ROUTE "C"
EXISTING GROVES NORTH WWTF
EXISTING PORT NECHES WWTF
GROVES
GRIFFING PARK
NEDERLAND
KANSAS CITY SOUTHERN RAILWAY



PN & PN/G – CITY OF PORT NECHES/GROVES
PROPOSED OUTFALL LINE

SCHAUMBURG & POLK, INC.
8865 College Street
BEAUMONT, TEXAS 77707
(409) 866-0341

SECTION 3-1 - GROVES EXISTING WASTEWATER TREATMENT FACILITIES

A. CITY OF GROVES NORTH WWTF

Plant Location: The plant site is located 1 mile northwest of the intersection of State Highway 347 and State Highway 73 in the 6100 block of Georgia Street north of Hogaboom Road in Jefferson County, Texas.

Receiving Stream: The discharge point is into a concrete lined Jefferson County Drainage District No. 7 (DD7) drainage canal; thence to DD7 Canal A; thence to Alligator Bayou; thence to Taylor Bayou; thence to DD7 Main Outfall Canal; thence to the Intracoastal Waterway in Segment 0702 of the Neches-Trinity Coastal Basin.

Discharge Permits: State - 10094-02
NPDES - TX0024651

Permit Limits: Maximum Monthly ADF = 0.83 mgd
Two-hour Peak = 2000 gpm (2.88 mgd)
BOD_5 = 20 mg/l
TSS = 20 mg/l
pH = 6 to 9
D.O. = 5 mg/l
Chlorination

Draft renewal permit calls for no discharge beginning October 1, 1998; City has requested variance allowing permit amendment based on new stream standards.

Existing Treatment Units and Sizes:

The Groves North treatment facility consist of a comminutor, bar screen, influent lift station, primary clarifier, trickling filter, final clarifier, chlorine contact, sludge digester, and sludge drying beds. Sludge is land filled. A description of the existing treatment units and their respective capacities are included in APPENDIX A.

B. CITY OF GROVES SOUTH WWTF

Plant Location: The south WWTF is located on Taft Avenue approximately 1 mile southeast of the intersection of Taft Avenue and State Highway 73 in Jefferson County, Texas.

Receiving Stream: The discharge point is into the Sabine-Neches Canal in Segment No. 0703 of the Neches-Trinity Coastal Basin.

Discharge Permits: State - 10094-01
NPDES - TX0024643

Permit Limits: Maximum Monthly ADF = 2.29 mgd
Two-hour Peak = 4771 gpm (6.87 mgd)
BOD_5 = 20 mg/l
TSS = 20 mg/l
pH = 6 to 9
D.O. = 5 mg/l (per EPA)
Chlorination/Dechlorination
Copper 0.061 mg/l*
Chronic and acute biomonitoring

* Copper limit being deleted per draft amended permit.

Existing Treatment Units and Sizes:

The Groves South treatment facility consist of bar screens, preaeration units, primary clarifier, trickling filters, final clarifier, chlorination, dechlorination, anaerobic sludge digesters, and sludge drying beds. Sludge is landfilled. A description of the existing treatment units and their respective capacities are included in APPENDIX A.

SECTION 3-2 - GROVES WASTEWATER TREATMENT REQUIREMENTS

A. <u>CITY OF GROVES</u>

1. **North Wastewater Treatment Facility:**

	Baseline (1994)	15 years	30 years
Population[1]	5,888	6,029	6,164
Design ADF[2]	1.95 mgd	1.96 mgd	1.99 mgd
Design Peak Flow[2]	5.85 mgd	5.88 mgd	5.97 mgd

[1] *Population projection calculations are included in APPENDIX B.*
[2] *Present and projected flow calculations are included in APPENDIX B.*

As indicated below the existing treatment capacity of the City of Groves North WWTF is inadequate for both Present Design ADF and Present Design Peak Flow.

	Permit Limits	WWTF Capacity[3]	Present Need
ADF	0.83 mgd	0.31 mgd	1.95 mgd
Peak Flow	2.88 mgd	0.62 mgd	5.85 mgd

[3] *WWTF Capacity as per Appendix A.*

It is noted that the present ADF and Peak Flow Capacity is limited by the capacity of the final clarifier based on minimum effective detention time. Additionally, the side water depth of the final clarifier does not meet TNRCC requirements.

The existing WWTF is designed to produce a secondary effluent of 20 mg/l BOD_5 and 20 mg/l TSS; however, it is expected that the WWTF will be required to meet effluent limits of 5 mg/l BOD_5, 5 mg/l TSS, 2 mg/l NH_3 and 6 mg/l D.O. in the future for continued discharge into the existing receiving stream.

Therefore, the existing WWTF is inadequate to meet future permit requirements and flow conditions.

A. CITY OF GROVES (continued)

2. **South Wastewater Treatment Plant:**

	Baseline (1994)	15 years	30 years
Population[1]	11,679	11,962	12,230
Design ADF[2]	3.26 mgd	3.28 mgd	3.33 mgd
Design Peak Flow[2]	18.70 mgd	18.70 mgd	18.70 mgd

[1] *Population projection calculations are included in APPENDIX B.*
[2] *Present and projected flow calculations are included in APPENDIX B.*

As indicated below the existing treatment capacity of the City of Groves South WWTF is inadequate for both Present Design ADF and Present Design Peak Flow.

	Permit Limits	WWTF Capacity[3]	Present Need
ADF	2.29 mgd	1.15 mgd	3.26 mgd
Peak Flow	6.87 mgd	2.31 mgd	18.70 mgd

[3] *WWTF Capacity as per Appendix A.*

It is noted that the present ADF and Peak Flow Capacity is limited by the capacity of the final clarifier based on minimum effective detention time. Additionally, the side water depth of the final clarifier does not meet TNRCC requirements.

SECTION 3-3 - GROVES COLLECTION SYSTEM REQUIREMENTS

A. CITY OF GROVES

The City of Groves operates and maintains a gravity collection system with some force main pumping. The collection system serves the entire City of Groves as there are no areas within the City for which service is not available. The system is approximately 40 years old. Additions to the City are not deemed a problem as far as capacity to serve is concerned. Most of the areas which have the potential for being developed are in the general vicinity of large collection lines from which service may be extended.

The City has a dual collection system in that it has two wastewater treatment plants. The north plant is served by a gravity collection system consisting of one trunk line 21" and larger and several smaller collection lines. The south plant, on the other hand, is served by one large pump station and force main. The pump station is served by two separate trunk lines which are 21" and larger along with several smaller collection lines. The lift station serving the south plant has an average daily pumping capacity of approximately 5500 gpm. This value was derived from the discharge records for the south plant.

SECTION 3-4 - GROVES PROPOSED ALTERNATIVES

A. GENERAL

Present, 15 year, and 30 year flow projections do not vary more than approximately 5%; therefore, the proposed alternatives have been analyzed based on the projected requirements for 30 years in the future.

B. CITY OF GROVES

1. Wastewater Treatment Needs and Alternatives

a. **NORTH WWTF:**

The existing WWTF is not capable of treating design ADF and Peak Flows, and proposed future effluent limits for the existing receiving stream will be considerably lower than these existing limits. Therefore, the City of Groves will need to construct a new North WWTF to meet the proposed future limits, or construct a new North WWTF to meet existing secondary limits and divert its effluent to a receiving stream with the existing secondary limits.

Because the City of Port Neches' WWTF is located directly adjacent to the Groves North WWTF and discharges into the same receiving stream, that WWTF must also upgrade to meet the proposed future limits or divert its effluent to a receiving stream with the existing secondary limits.

Therefore, construction of a common lift station and force main(s) for diverting the discharges to the Neches River and construction of a new WWTF to meet secondary limits was analyzed. A summary of these alternatives is provided below. A detailed analysis of each alternative is included in APPENDIX E, except that the common lift station/force main is covered in APPENDIX D.

a. Alternate G1. This alternate proposes to construct improvements to the North WWTF to fully treat all flows utilizing the trickling filter process. Discharge of all *(or part)* of flows will be to the Neches River at effluent limits of 20 mg/l BOD_5, 20 mg/l TSS, and 4 mg/l DO.

Opinion of Probable Construction Cost = $4,093,000*

Annual O&M Costs = $237,000*
* Plus cost for outfall to river or City's share of PN/G-1 or PN/G-2.

Advantages: Makes use of existing process units to economically upgrade the plant.

Disadvantages: Will require continued operation of 2 treatment facilities. may require future upgrade to meet 10/15/3 limits. *Possibly will only require that ADF be pumped to the Neches River and allow Peak Flows in excess of ADF to be discharged into current receiving stream.*

b. Alternate G2. This alternate proposes to construct a new activated sludge WWTF for full treatment of all flows and discharge to the proposed effluent lift station (Alternate PN/G2) for discharge to the Neches River at effluent limits of 20 mg/l BOD_5, 20 mg/l TSS, and 4 mg/l DO.

Opinion of Probable Construction Cost = $5,273,000 *(+ City's share of PN/G2)*

Annual O & M Costs = $371,138 *(+ City's share of PN/G2)*

Advantages: Will not require construction of new WWTF to meet advanced effluent limits.

Disadvantages: Will require future operation of two separate treatment facilities as opposed to Alternate G5, G6, or G7; may require pumping all effluent (ADF and Peak Flow) approximately 3-3.5 miles to the Neches River. *Possibly will only require that ADF be pumped to the Neches River and allow Peak Flows in excess of ADF to be discharged into current receiving stream.*

b. **SOUTH WWTF:**

The existing WWTF is not capable of treating future design and Peak Flows. Therefore, construction of a new WWTF was analyzed. A summary of this alternative is provided below and a detailed analysis is included in APPENDIX E.

a. Alternate G3. This alternate proposes to construct a new activated sludge WWTF for full treatment of all flows and discharge to the existing receiving stream at effluent limits of 20 mg/l BOD_5, 20 mg/l TSS, and 5 mg/l DO.

Opinion of Probable Construction Cost = $8,747,000

Annual O & M Costs = $675,914

Advantages: Will provide additional capacity for treatment of excessive I/I within the south collection system. Will bring South WWTF into full compliance with current TNRCC Design Criteria.

Disadvantages: Will require future operation of two separate treatment facilities as opposed to Alternates G5, G6, and G7.

Alternate G4. This alternate proposes to construct improvements to the South WWTF to fully treat all flows utilizing the trickling filter process. Discharge will be to the Sabine-Neches Ship Channel at effluent limits of 20 mg/l BOD_5 20 mg/1 TSS, and 4 mg/l DO.

Opinion of Probable Construction Cost = $8,448,000

Annual O&M Costs = $297,000

Advantages: Makes use of existing process units to economically upgrade the plant.

Disadvantages: Will require continued operation of 2 treatment facilities. Environment at this location is aggressive and creates higher than "normal" maintenance requirements.

c. **GROVES REGIONAL WWTF:**

As an alternative to the construction of two new WWTF's within the City of Groves, a single regional WWTF to serve the entire City of Groves was analyzed. A summary of this alternative is provided below and a detailed analysis is included in APPENDIX E.

a. Alternate G5. This alternate proposes to construct a new regional activated sludge WWTF at the North Plant site for full treatment of all flows within the City of Groves and divert (pump) discharge to the Neches River at effluent limits of 20 mg/l BOD_5, 20 mg/l TSS, and 4 mg/l DO. For pumping cost for diversion of discharge to the Neches River, see PN/G3.

Opinion of Probable Construction Cost = $12,184,000 *(11,238,000)**

Annual O & M Costs = $773,223*

* Plus City's share of PN/G3.

Advantages: Will eliminate construction and operation of one entire WWTF, and provide capacity for treatment of wet weather flow in the north and south collection system.

Disadvantages: Will require all flows from the south collection system to be pumped to the north WWTF site.

Alternate G6. This alternate proposes to construct a new regional activated sludge WWTF at the South Plant site for full treatment of all flows within the City of Groves. Discharge will be to the Sabine Neches Ship Channel at effluent limits of 20 mg/l BOD_5, 20 mg/l TSS, and 5 mg/l DO.

Opinion of Probable Construction Cost = $13,337,000

Annual O&M Costs = $387,652

Advantages: Will eliminate construction and operation of one entire WWTF, and provide capacity for treatment of wet weather flow in both the North and South collection systems.

Disadvantages: Will require all flows from the north collection system to be pumped to the south WWTF site.

Alternate G7. This alternate proposes to construct a new regional trickling filter WWTF near the intersection of 32nd Street and Hwy. 366 for full treatment of all flows within the City of Groves. Discharge will be to the Sabine Neches Ship Channel at effluent limits of 20 mg/l BOD_5, 20 mg/l TSS, and 5 mg/l DO.

Opinion of Probable Construction Cost= $14,337,000

Annual O&M Costs = $531,000

Advantages: Will eliminate construction and operation of one entire WWTF, and provide capacity for treatment of wet weather flow in both the North and South collection system.

Disadvantages: Will require all flows from both north & south collection systems to be pumped to the new WWTF site.

2. Collections System Needs and Alternatives

a. 21" & 24" TRUNK LINE SERVING THE NORTH PLANT

No improvements are recommended for this trunk line. However, steps to eliminate I/I and associated by-passing are recommended.

b. <u>21" & 24" TRUNK LINE ALONG TAFT AVENUE</u>

No improvements are recommended for this trunk line. However, steps to eliminate I/I and associated manhole surcharging are recommended.

c. <u>Alternate G-8.</u> This alternate proposes to construct a new lift station to divert discharge (up to design ADF) from the Groves North WWTF to the Neches River. This alternate will pump 2.00 MGD ADF to the river.

Opinion of Probable Construction Cost = $1,200,000

Annual O&M Costs = $50,000

Advantages: Will allow Groves to operate this facility independently. Schedule for construction, etc. is not dependent on other entities.

Disadvantages: Is not the most cost efficient alterative. Duplicates a similar effort by Port Neches.

d. <u>PUMP STATION & LIFT STATION CONSTRUCTION</u>

A new pump station along side the existing lift station at Taft Ave. and 25th St. is needed. For continued transportation to the South Plant (or to a new City regional plant at the South Plant site), the proposed pump station will have a firm capacity of 7,500 gpm and will consist of three (3) 3750 gpm pumps. Included is 7500 linear feet of 20" force main. The pump station will have a 14' x 14' x 15' SWD (18' box depth) wet well. Additional land may be required adjacent to the existing site.

OPINION OF PROBABLE COSTS:

1.	Three (3) 3750 gpm Pumps Installed	$ 150,000.00
2.	Piping & Valves	$ 30,000.00
3.	Electrical & Instrumentation	$ 40,000.00
4.	Misc (Fence, Hatches, Etc.)	$ 7,000.00
5.	Concrete Structure	$ 49,950.00
6.	7500 L.F. 20" Force Main	$ 525,000.00
	Sub-Total	$ 801,950.00
	15% Contingency	$ 120,293.00
	Total Cost	$ 922,243.00

(Included in Alternates G3, G4, and G7)

SECTION 3-5 - GROVES ENFORCEMENT ACTION STATUS

The City is under the following enforcement actions:

TNRCC: The City is under the 75/90 rule for its North Plant because of flows which periodically approach or exceed plant capacity. This rule requires the City to work toward plant expansion and/or other means of correcting the flow problems such as I/I correction.

EPA: The City is under an Administrative Order, Docket No. VI-95-122 *(March 24, 1995)*, which imposes a corrective action schedule for the South Plant. This schedule was submitted in response to a previous order stemming from BOD_5, suspended solids, and chlorine residual violations. The schedule is as follows:

- Select an option for plant improvements, relocation, etc. by August 1995.
- Complete financing arrangements for the project by November 1995.
- Begin construction by October 1996.
- Complete construction by October 1998.
- Attain compliance by January 1999.

SECTION 3-6 - GROVES: OTHER INFORMATION

A. WATER SUPPLY

The City draws water from the local LNVA canal system, then treats it in a City plant. The City's contract with the LNVA does not specify an upper limit of usage. The City anticipates continuing its existing water supply practice for the entire 30 year study period. See the separately bound Water Conservation Plan for further information.

B. SLUDGE MANAGEMENT

Under present circumstances the City would continue to send all sludge to a Class I landfill for codisposal regardless of the amounts of sludge generated under the various alternatives.

Although state and federal policies on sludge disposal nominally encourage beneficial use of municipal sludge, the corresponding standards for sludge quality in practice make beneficial use unfeasible for the Southeast Texas area. To the knowledge of the Engineer, only one site in Southeast Texas has been registered, north of Orange, and the site owner has terminated his contract with at least one community in recent years. The nearest known registered site is located in Tyler County over 50 miles away from Groves and reportedly accepts only liquid sludge, for which it charges a fee. It appears that practices such as land application are proving feasible only for large metropolitan areas such as Houston, where use of the sludge is more attractive economically.

The City presently sends its sludge in dewatered form to a commercial landfill south of Beaumont on LaBelle Road, operated by Browning-Ferris Industries. According to the Engineer's conversation with the landfill manager (7/5/95), the presently permitted landfill facility is expected to have capacity through the year 2030, or several years beyond the study period.

C. PROJECT SCHEDULE

It is impractical to develop a realistic schedule for project implementation at this time, since the City must continue to assess its changing situation before implementing its project. One of the prime reasons for needing a project *(for the North Plant)* is the upgrading several years ago of the stream standards for the drainage ditch system into which the North Plant discharges.

The recently issued draft of the renewed TNRCC permit for the Groves North Plant provides for no discharge into the present receiving streams beginning October 1, 1998. However, while awaiting permit issuance, the City has requested a variance *(similar to the existing variances in the Nederland and Port Neches permits)* reflecting the probability that the TNRCC will adopt slightly relaxed stream standards for the receiving streams. Since the

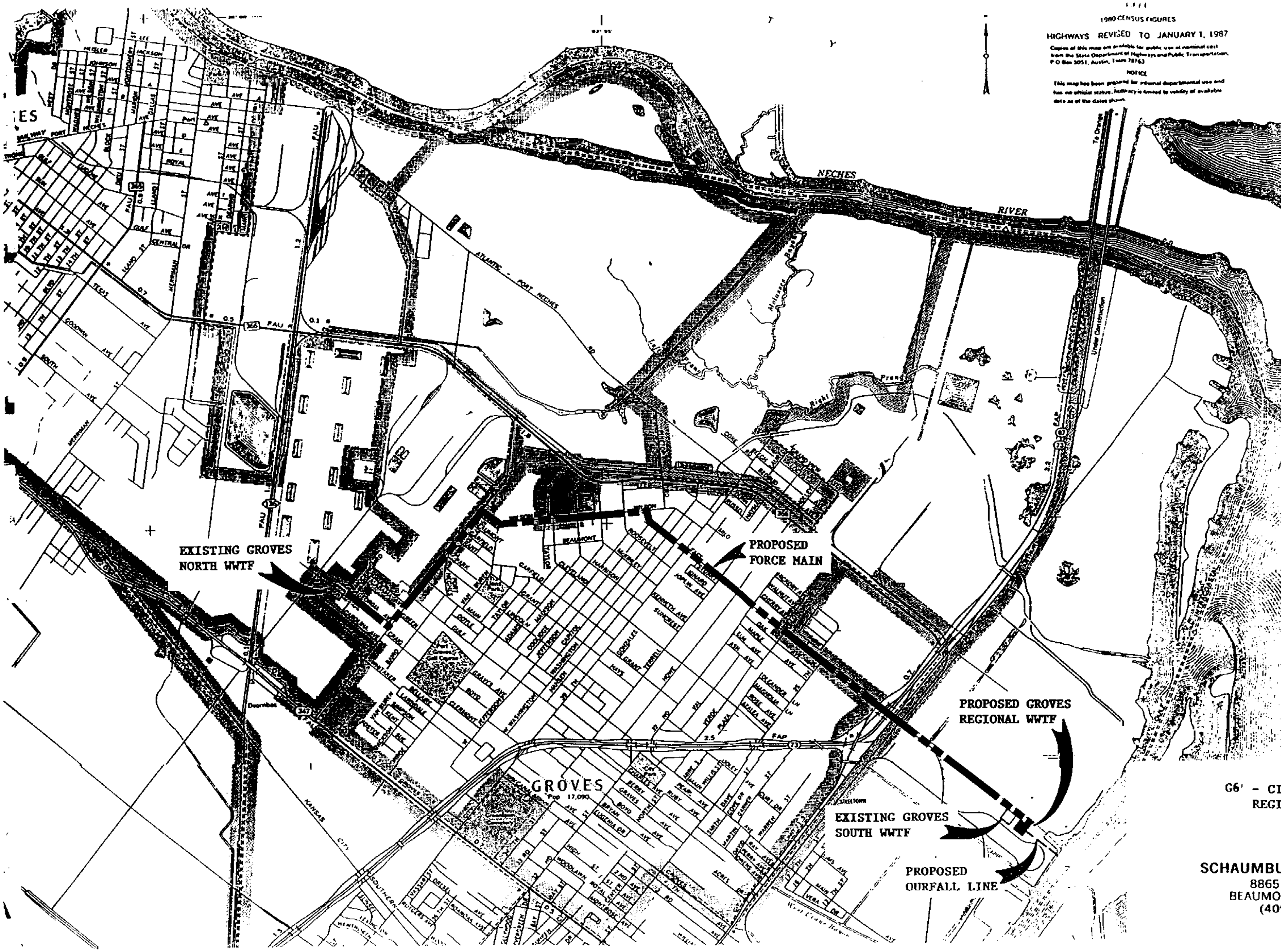
TNRCC actually adopted the revised standards on June 14, 1995, the City can *(following permit issuance)* reflecting the revised standards. Such an amendment would grant additional time for compliance with the new standards *(or for alternate measures such as diversion to another stream)* and in all probability also relax the requirements for continued discharge into the stream.

The Groves South Plant, which discharges into the Sabine-Neches Canal, has recently received a draft of an amended permit for the purpose of removing a copper limit. Unlike the Groves North permit, this permit does not provide for a no-discharge condition or an upgrading of effluent standards. It should be noted, however, that the permit is set to expire at the end of July 1998 according to the basin plan schedule. The next renewal could possibly impose stricter standards, but the TNRCC has not given any indication that would be the case.

However, the Groves South Plant has experienced various problems including overloading from infiltration/inflow. The City is under an EPA administrative order imposing a schedule for bringing the plant into compliance. The regional wastewater study indicates that if the plant is retained to serve its existing service area, it will require a major expansion. A major permit amendment would be required for the expansion, and the increased flows could possibly result in upgraded effluent standards in the amended permit or in future renewals.

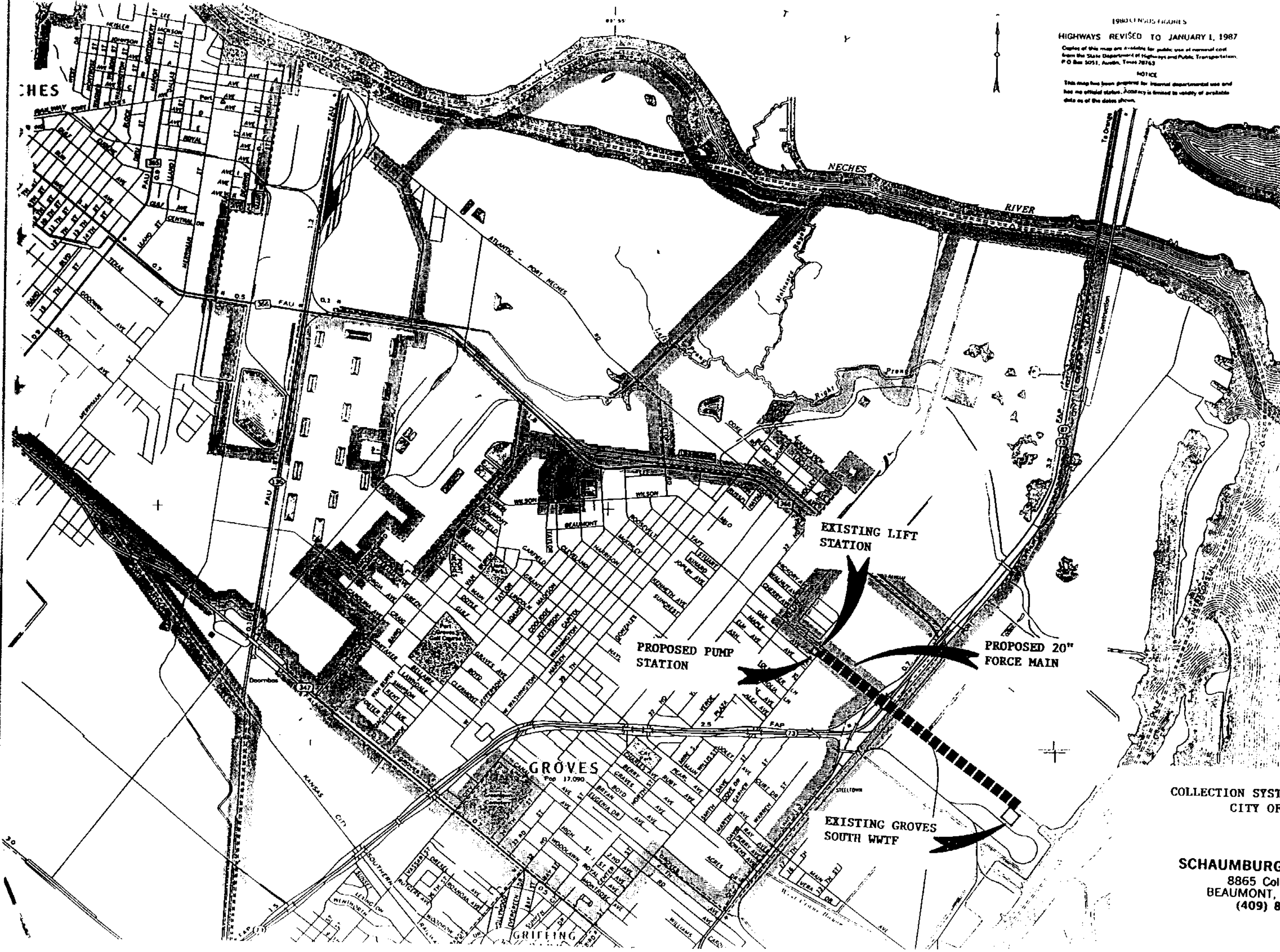
In light of the recent revisions to TNRCC stream standards, it appears that the next step for Groves *(with regard to the North Plant)* may be to request new stream modelling from the TNRCC staff to verify what new effluent standards would apply to the appropriate design flows for continued discharge into the existing stream. At the same time, the City could also request a determination regarding the discharge of excess storm flows into the existing stream in periods of high stream flows. After receiving responses from the TNRCC staff *(assuming the responses to be consistent with the assumptions used in the report)*, the City could then begin implementing the project recommended in the report. This implementation would begin with the appropriate permit amendment application and/or with the necessary SRF engineering studies for TWDB financing. The SRF study would itself contain an implementation schedule.

For the North Plant, the City should also coordinate with Port Neches regarding a possible joint outfall to the river. However, Groves must also consider the urgency of the needed improvements to its South Plant. After considering these matters, the City can confirm or revise its previously selected alternatives and begin the permitting and SRF engineering processes. As in the case of the North Plant, the resulting SRF report would contain an implementation schedule. However, any schedule must meet the minimum requirements of the schedule in the EPA Administrative Order.



G6' - CITY OF GROVES
REGIONAL WWTF

SCHAUMBURG & POLK, INC.
8865 College Street
BEAUMONT, TEXAS 77707
(409) 866-0341



COLLECTION SYSTEM IMPROVEMENTS
CITY OF GROVES

SCHAUMBURG & POLK, INC.
8865 College Street
BEAUMONT, TEXAS 77707
(409) 866-0341

SECTION 4-1 - REGIONAL TREATMENT FACILITY

REGIONAL FACILITY

A Regional WWTF was analyzed for treatment of all wastewater flows from the City of Nederland, Port Neches and Groves (both North and South systems). An alternate Regional WWTF was also analyzed with Port Neches excluded. A summary of each alternative is provided below, along with a detailed analysis of the three city plant.

1. Alternate D8. This alternate proposes to construct a regional WWTF for treatment of all wastewater from the Cities of Nederland, Port Neches and Groves (both North and South systems), and discharge into the Neches River at effluent limits of 20 mg/l BOD_5, 20 mg/l TSS, and 4 mg/l DO.

   Opinion of Probable Construction Cost = $36,278,000

   Annual O & M Costs = $2,620,329

   Advantages: Would consolidate the treatment of all three Cities to one site.

   Disadvantages: Would required abandonment of all existing WWTF's, including the City of Port Neches WWTF which has sufficient capacity to treat the project 30 year design flows for the City of Port Neches. Is not cost effective.

2. Alternate D-9. This alternate proposes to construct a regional WWTF for treatment of all wastewater from the Cities of Nederland and Groves (both North and South systems), and discharge into the Neches River at effluent limits of 20 mg/l BOD_5, 20 mg/l TSS, and 4 mg/l DO.

   Opinion of Probable Construction Cost = $33,294,000

   Annual O & M Costs = $1,642,791

   Advantages: Would consolidate the treatment of two Cities to one site.

   Disadvantages: Would required abandonment of all existing WWTF's, except the City of Port Neches WWTF which has sufficient capacity to treat the project 30 year design flows for the City of Port Neches. Is not cost effective.

3. Alternate D10. A Regional Laboratory was analyzed versus individual laboratories for each city. A detailed analysis of this alternative is included in this section.

The three-city regional system was sized to take peak thirty day average flows from all three cities as well as the combined peak wet weather flows from the three cities, treat it with an activated sludge process, provide adequate solids handling facilities, and provide adequate hydraulic capacities for the handling for the wet weather peak flows. This alternative also includes the cost of lift stations and force mains to transport flows from the three cities to the regional facility site and from the site to the Neches River.

The two-city system was similar to the three-city system, allowing for Port Neches to retain its existing plant which is adequate for 20/20 treatment. The plant was downsized to treat only flows from Nederland and Port Neches. The transportation facility from the Groves North Plant site to the new plant, which was previously sized to include flows from the adjacent Port Neches Plant, was also downsized.

A lagoon/wetland facility for the intercity plant was considered initially, but would have required approximately 1500 acres for all three. A single parcel of property of this size (or even large enough for two cities) is simply not available within a reasonable distance or in reasonable location for the three cities.

Since the three cities are almost fully developed at this time, it was a challenge to identify an adequate site for construction of even a conventional regional facility. The land adjacent to the Port Neches and Groves North Plants was not available in sufficient quantity (see also discussion in Appendix C). The existing Nederland Plant is landlocked. Property near the river and not inside a marshy area is very scarce. A useable site in an undeveloped portion of Port Neches, outside the Neches River marshes, was finally identified. Transfer facilities (lift stations with force mains) from the various existing plant sites were laid out.

SECTION 4-2 - REGIONAL WWTF: (Alternate D8) **Abandon Existing WWTF's, Construct New Regional Activated Sludge Treatment Facility, Divert Discharge to the Neches River**

Abandon existing WWTF's, convert existing influent lift station(s) to transfer lift station(s) to pump the raw wastewater flow to a regional activated sludge treatment facility and then discharge into the Neches River at the following effluent limits.

	Nederland	Port Neches	Groves North	Groves South	Total
ADF	5.22 mgd	3.88 mgd	1.99 mgd	3.33 mgd	**14.42 mgd**
2-Hour Peak	26.10 mgd	21.34 mgd	5.97 mgd	18.70 mgd	**72.11 mgd**

BOD_5	=	20 mg/l
TSS	=	20 mg/l
NH_3	=	no limit
D.O.	=	4 mg/l

A. Transfer Lift Stations

1. Nederland. Convert the existing influent lift station to transfer the raw wastewater flows to the proposed regional wastewater treatment facility.

Existing: Four pumps, submersible type, installed in dry pit.

Required: *Firm pumping capacity (largest pump out of service) must be adequate to pump peak flow to destination; three or more pumps (or duplex pumps with automatic variable speed control) required.*

Analysis: Firm capacity = 18,125 gpm

Four (4) pumps with a firm pumping capacity of 18,125 gpm with largest pump out of service.

Transfer force main(s) sized for a minimum of 2 fps at low flow and a maximum 10 fps at peak flow.

Install two (2) force mains, one for ADF and one as a Peak Flow force main.

Proposed ADF force main = 18" diameter
Proposed Peak Flow force main = 30" diameter

Improvements: **Convert existing influent lift station to a transfer lift station**

with four (4) pumps (firm capacity of 18,125 gpm) and a dual 18"/30" diameter transfer force main to the proposed regional wastewater treatment facility.

2. Port Neches/Groves North. Construct a transfer lift station to pump the raw wastewater flows to the proposed regional wastewater treatment facility.

Existing: Port Neches - NONE. Raw wastewater flows are pumped to the existing WWTF from three off-site lift stations.

Groves North - To be abandoned.

Required: Firm pumping capacity (largest pump out of service) must be adequate to pump peak flow to destination; three or more pumps (or duplex pumps with automatic variable speed control) required.

Analysis: As the existing Port Neches WWTF and the Groves North WWTF are located adjacent to each other, the proposed transfer lift station should be sized to handle the raw wastewater flows from both of these systems.

Firm capacity = 31.10 mgd (21,597 gpm) [Refer to Alternate PNG-3]

Five (5) pumps total; firm capacity of 21,597 gpm with largest pump out of service (i.e. 4 pumps pumping + 1 spare).

Transfer force main(s) sized for a minimum of 2 fps at low flow and a maximum 10 fps at peak flow.

Install two (2) force mains, one for ADF and one as a Peak Flow force main.

Proposed ADF force main = 24" diameter
Proposed Peak Flow force main = 30" diameter

Improvements: Construct a transfer lift station with five (5) pumps (firm capacity of 21,597 gpm) and dual 24"/30" diameter force mains to the regional wastewater treatment facility.

3. Groves South. All influent flow to the South WWTF are pumped from the Taft Avenue lift station. This lift station will have to be upgraded to provide a firm capacity of 12,986 gpm (18.70 mgd). Upgrading of this lift station is addressed

under the proposed collection system improvements for the City of Groves.

A transfer force main will have to be constructed from the Taft Avenue lift station to the proposed regional treatment facility.

Required: *Firm pumping capacity (largest pump out of service) must be adequate to pump peak flow to destination; three or more pumps (or duplex pumps with automatic variable speed control) required.*

Analysis: Firm capacity = 12,986 gpm

Transfer force main sized for a minimum of 2 fps at low flow and a maximum 10 fps at peak flow gpm.

Proposed force main = 30" (5.9 fps @ 12,986 gpm)

Improvements: **Convert existing Taft Avenue lift station to a transfer lift station and construct a 30" diameter transfer force main to the proposed regional wastewater treatment facility.**

B. Preliminary Treatment

1. Screening

Required: *Some form of screening; bar openings minimum ½" for mechanical screens; velocities @ design flow minimum 2 ft./sec through channel, < 3 ft./sec. through screen.*

Analysis:

ADF	= 14.42 mgd	= 22.31 cfs
2-Hour Peak	= 72.11 mgd	= 111.57 cfs

Channel Width = 14 ft. max

Screen Size = 14 ft. wide x 0.5 inch bars x 0.75 inch openings
Assumed Screen Efficiency = 60 %

Improvements: **Construct a three channel (7 ft. wide/channel) influent structure with two mechanical bar screens and one fixed bar screen.**

2. Grit Chamber (Aerated)

Required: *Grit removal recommended; if removal units are provided, must have method of removing grit from unit, and any unit with single* *chamber must have bypass.*

Analysis: Detention time = 20 minutes @ ADF, 5 minutes @ Peak
Air requirements = 20-25 cfm / 1,000 ft^3
Draft tube = 25 cfm / 1,000 ft^3

Volume required:
@ ADF = (14,420,000 gpd)(20 min.)/(7.48 gal/ft^3)(1 day/1440 min.)
= 26,775 ft^3

@ Peak = (72,110,000 gpd)(5 min.)/(7.48 gal/ft^3)(1 day/1440 min.)
= 33,474 ft^3

Use square basin = 47 ft. x 47 ft. x 15 ft. SWD w/ 5:12 bottom slope

Air required:
(47 ft. x 47 ft. x 15 ft.)(25 cfm/1,000 ft^3) = 828 cfm

Draft tube required:
Area = (828 cfm)(1 ft^2/25cfm) = 33.1 ft^2
Use 6.5 ft. diameter tube

Improvements: **Construct 47 ft. x 47 ft. x 15 ft. SWD aerated grit chamber with 828 cfm aeration within a 6.5 ft. draft tube.**

3. Influent Lift Station

Required: *Firm pumping capacity (largest pump out of service) must be adequate to pump peak flow to destination; three or more pumps (or duplex pumps with automatic variable speed control) required.*

Analysis: ADF = 14.42 mgd (10,014 gpm)
Firm capacity = 72.11 mgd (50,076 gpm)

Seven (7) pumps total; 2 pumps for ADF + 5 pumps for firm capacity of 50,076 gpm with largest pump out of service (i.e. 7 pumps pumping + 1 spare).

Improvements: **Construct an influent lift station with seven (7) pumps (firm capacity of 50,076 gpm) at the regional wastewater treatment facility.**

4. Primary Clarifiers

Required: *Primary clarifier maximum surface loading at Peak flow of 1800 gal./day/ft^2, and at Design flow of 1000 gal./day/ft^2. Side water depth must be at least 7 ft. Allow 35% reduction in BOD_5 through primary clarification.*

Analysis: Required area:
@ Peak flow = 72,110,000 gpd / 1800 gal./day/ft^2 = 40,061 ft^2
@ Design flow = 14,420,000 gpd / 1000 gal./day/ft^2 = 14,420 ft^2

Four (4) - 114 ft. diameter clarifiers w/14 ft. stilling well = 40,828 ft^2
Provide for 14 ft. side water depth

Improvements: **Construct four (4) primary clarifiers, 114 ft. diameter each with 14 ft. side water depth. Provide flow splitting/collection structures/piping, and sludge collection/pumping.**

C. Activated Sludge Process. Construct a single stage nitrification, activated sludge unit.

Required: *Total volume shall be 1000 ft^3 per 35 lb. BOD_5/day. Diffused aeration shall be designed for 3200 SCF per lb. BOD_5. The diffuser system must be capable of providing 150% of design requirements.*

Analysis: Use 30 lb. BOD_5/day per 1000 ft^3 aeration volume

lbs. BOD_5/day = [(14.42 mgd)(8.345)(200 mg/l)](65%)
= 15,644 lbs. BOD_5/day

Required Volume = (15,644 lb BOD_5/day)(1000 ft^3)/30 lb BOD_5/day
= 521,467 ft^3

Proposed basins ratio = 2 basins, 22 ft. deep SWD, w/ 2length:1width

= 2 x 22 ft. deep x 77 ft. wide x 154 ft. long
= 521,752 ft^3

Air Requirements = (15,644 lbs. BOD_5/day)(3200 SCF/lbs. BOD_5)
= 50,060,800 SCF/day
= 34,764 cfm

150% Design Req. = (34,764 cfm)(1.5)

$= 52,146$ cfs

Improvements: **Construct a dual activated sludge deep tank aeration basin (each 22 ft. deep x 77 ft. wide x 154 ft. long) and provide 52,146 cfm aeration capacity.**

D. Final Clarifiers.

Required: *Maximum surface loading at Peak flow of 1200 gal./day/ft², and at Design flow of 600 gal./day/ft². Side water depth must be at least 10 ft. for surface areas of 1250 ft² or more. Effective detention times (based on liquid volume above a 3 ft. sludge blanket) must be 1.5 hr. @ Peak flow and 3.0 hr. @ Design flow.*

Analysis: Required area based on surface area:
@ Peak flow = 72,110,000 gpd / 1200 gal./day/ft² = 60,092 ft²
@ Design flow = 14,420,000 gpd / 600 gal./day/ft² = 24,033 ft²

Required area based on detention time:
Detention time is based on side water depth of 14 ft. less 3 ft. sludge blanket

$$\text{@ Peak Flow} = \frac{(72{,}110{,}000\ \text{gpd})(1.5\ \text{hrs.})}{(24\ \text{hrs/day})(7.48\ \text{gal/ft}^3)(11\ \text{ft.})} = 54{,}775\ \text{ft}^2$$

$$\text{@ Design Flow} = \frac{(14{,}420{,}000\ \text{gpd})(3.0\ \text{hrs.})}{(24\ \text{hrs/day})(7.48\ \text{gal/ft}^3)(11\ \text{ft.})} = 21{,}907\ \text{ft}^2$$

Required area based on Peak flow surface area requirements govern.

Four (4) - 140 ft. diameter clarifiers w/18 ft. stilling well = 60,557 ft² effective surface area. 14 ft. side water depth.

Improvements: **Construct four (4) 140 ft. diameter final clarifiers with 14 ft. side water depths. Provide flow splitting/collection structures/piping, and sludge collection/pumping.**

E. Effluent Works.

1. Chlorine Contact Chamber.

Required: *Detention time of 20 minutes @ peak flow.*

Analysis: (72,110,000 gpd/1440 min/day)(20 min)/(7.48 gal/ft³) = 133,894 ft³

Basin Dimensions:	2 Basin(s)		
	Width	= 38 ft./basin	
	Length	= 150 ft.	
	Depth	= 12.0 ft. SWD	

Improvements: **Construct a dual chlorine contact chamber (each basin - 38 ft. wide x 150 ft. long by 12 ft. SWD).**

2. Chlorine Feed Equipment.

Required: *Feed equipment must be able to provide more than the highest dosage to be required at any time. Dosage must be adequate to maintain a chlorine residual of at least 1 mg/l after 20 minutes detention, prior to dechlorination.*

Improvements: **Provide chlorine feed equipment as necessary to provide for chlorination of 72.11 mgd.**

3. Dechlorination.

Required: *The effluent, after chlorination and 20 minutes detention time, must be dechlorinated to less than 0.1 mg/l. For most dechlorination agents, 1 minute detention is generally considered adequate.*

Analysis: (72,110,000 gpd/1440 min/day)(1 min)/(7.48 gal/ft^3) = 6695 ft^3

Basin Dimensions:	2 Basin(s)	
	Length	= 38 ft./basin
	Width	= 9 ft.
	Depth	= 10.0 ft. SWD

Improvements: **Construct a dual dechlorination chamber (each basin - 38 ft. long x 9 ft. wide x 10 ft. SWD) and provide chemical feed equipment as necessary to provide for dechlorination of 72.11 mgd.**

4. Flow Measurement.

Required: *Continuous effluent measurement required, with capacity for maximum expected peak flow.*

Improvements: **Construct a parshall flume capable of measuring flows up to 75 mgd.**

5. Postaeration.

Improvements: **Construct a passive, cascade type aeration structure on the discharge capable of producing a 4 mg/l dissolved oxygen effluent.**

6. Outfall

Analysis:

ADF	= 14.42 mgd (22.31 cfs)	
Peak flow	= 72.11 mgd (111.57 cfs)	
Gravity flow	= > 2 fps @ ADF, < 10 fps @ Peak	

Required diameter: @ ADF = 48"

48" diameter @ Peak flow (111.57 cfs) = 8.9 fps

Improvements: **Construct a 48" diameter gravity outfall line to the Neches River.**

F. Sludge Processing.

1. Sludge Thickener.

Required: *Digesters should be provided with sludge thickening.*

Improvements: **Sludge thickening will be provided by decanting inside the digestors.**

2. Aerobic Digesters.

Requirement: *Minimum solids retention time of 15 days (may be calculated as 20 ft^3 for each lb. influent BOD_5 per day). Diffused air requirement is 30 cfm per 1000 ft^3 of volume.*

Analysis: Digester volume required = 20 ft^3 / lb. influent BOD_5 per day

lbs. BOD_5/day = (14.42 mgd)(8.345)(200 mg/l)
= 24,067 lbs. BOD_5/day

Required Digester Volume = (20)(24,067) = 481,340 ft^3

Proposed Digesters (2)	= 22 ft. SWD
	= 481,340 ft^3 / (22 ft. x 2)
	= 10,940 ft^2
	= 105 ft. x 105 ft.

Required aeration = (30 cfm/1000 ft^3)(481,340 ft^3) = 14,440 cfm

Improvements: **Construct two (2) deep tank type aerobic digesters (105 ft. x 105 ft. x 22 ft. SWD) and provide 14,440 cfm aeration equipment capacity.**

3. Sludge Dewatering Facility.

Required: *Sludge shall be dewatered sufficiently to meet the requirements of the ultimate form of disposal.*

Improvements: **Provide sludge dewatering facilities.**

G. Blowers.

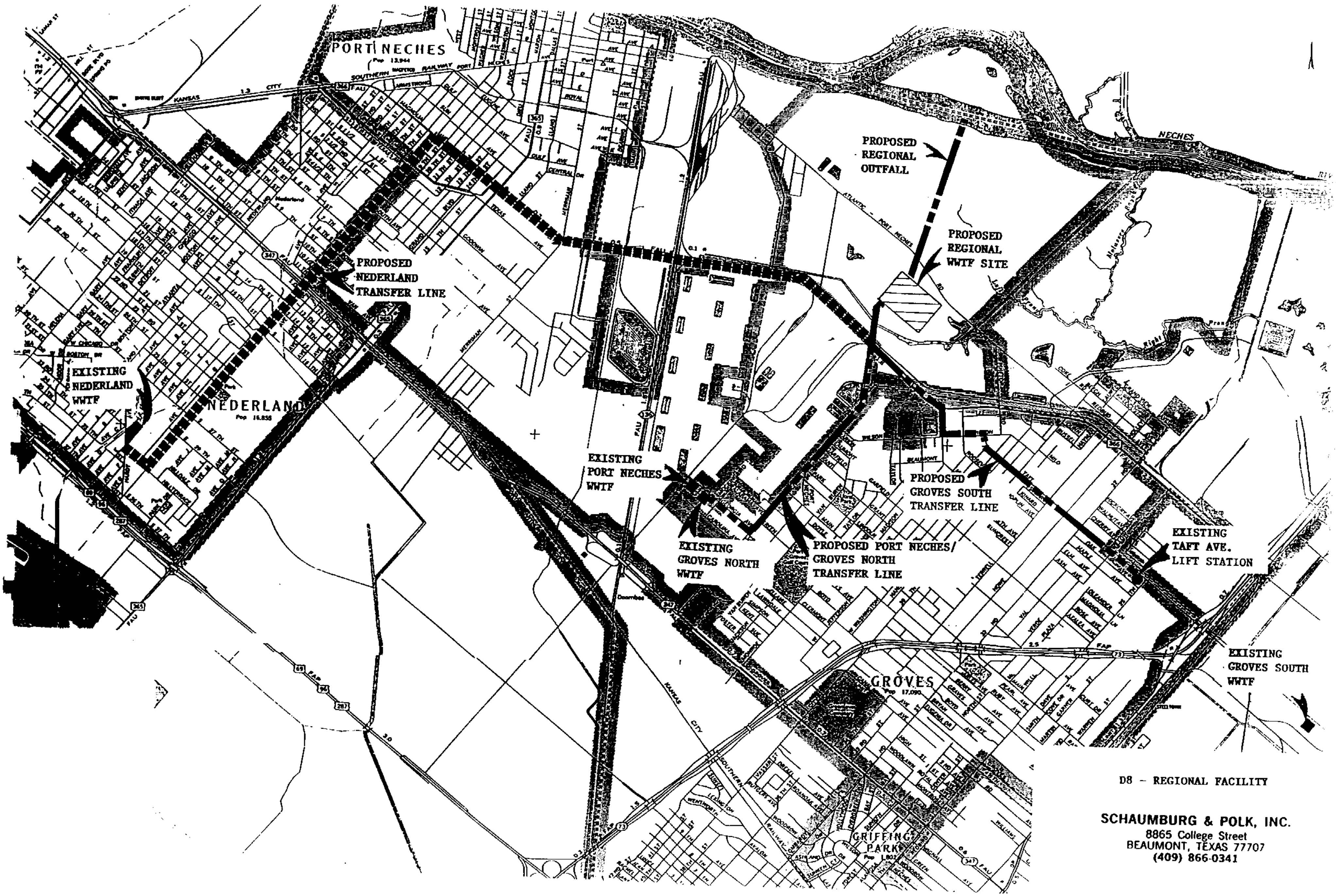
Required: *Blowers must be able to meet maximum aeration requirements with largest unit out of service.*

Analysis:		
	Aerated Grit Chamber	= 828 cfm
	Activated Sludge Aeration	= 34,764 cfm
	Aerobic Digestion	= 14,440 cfm
		= 50,032 cfm

Improvements: **Provide blowers as required for activated sludge aeration and aerobic digestion.**

H. Opinion of Probable Cost

1.	Transfer lift stations/force mains	
	a. Nederland	$ 5,389,000
	b. Port Neches/Groves North	$ 2,616,000
	c. Groves South	$ 1,579,000
2.	Influent headworks, screens	$ 205,000
3.	Grit chamber	$ 329,000
4.	Influent Lift Station	$ 1,155,000
5.	Primary clarifiers	$ 3,011,000
6.	Activated sludge basin(s)	$ 2,242,000
7.	Final Clarifiers	$ 3,962,000
8.	Chlorination/dechlorination chamber/feed equipment	$ 650,000
9.	Effluent flow measurement/post-aeration	$ 86,000
10.	Outfall	$ 745,000
11.	Aerobic digestors	$ 1,589,000
12.	Sludge dewatering facilities	$ 738,000
13.	Aeration blower equipment	$ 844,000
14.	Yard piping improvements	$ 1,882,000
15.	Site work	$ 1,584,000
16.	Electrical and instrumentation	$ 1,760,000
17.	Laboratory/Office	$ 350,000
18.	Site Acquisition	$ 1,034,000
	Subtotal	$ 31,546,000
	Contingency (15%)	$ 4,732,000
	TOTAL OPINION OF PROBABLE CONSTRUCTION COST	**$ 36,278,000**



D8 - REGIONAL FACILITY

SCHAUMBURG & POLK, INC.
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BEAUMONT, TEXAS 77707
(409) 866-0341

SECTION 4 - Regional Laboratory:

EVALUATION OF REGIONAL
WATER AND WASTEWATER
LABORATORY TESTING FACILITIES

SUMMARY OF FINDINGS

The advantages and disadvantages of parameters evaluated in this section reveals that the best laboratory facility arrangement is a combination of two of the options considered. A regional laboratory would be most advantageous for the wastewater systems only in conjunction with a regional treatment plant and if constructed as a part of the new facility. (Option I-A). If individual wastewater plants are maintained then the best concept would be to also maintain individual labs at each facility. (Option II-C).

Similarly, the best water system laboratory arrangement would be to maintain the individual labs with each water treatment facility. (Option II-C). The construction of a regional lab in conjunction with a regional wastewater treatment plant would not provide all of the needs of the individual water plants and could not completely replace the individual labs at each water plant facility.

GENERAL

This section addresses the options available for both water and wastewater qualitative testing facilities. Although this study considers only the regionalization of wastewater treatment facilities, the required testing of potable water has been included in this section for several reasons:

- Some laboratory facilities are common to both water & wastewater testing & combining the facilities may have attractive advantages.

- More stringent treatment limits have been placed on water systems over the past 10 to 20 years resulting in additional and more sophisticated testing requirements. This periodic addition of new testing requirements with its inherent piece-meal addition to existing lab facilities may no longer be the best option available.

- Additional treatment limits will probably be placed on potable water systems during the next five years and, subject to many factors, may continue for 10 to 20 years. A common lab facility should be better suited to adjust to or expand if and when the need arises.

ALTERNATIVE SELECTION

The alternatives considered consist of combinations of the following major options:

I - Construct new regional treatment facility
II - Maintain existing individual treatment facilities
and
A - Construct new regional laboratory facility
B - Expand an existing laboratory facility to be a regional laboratory
C - Maintain existing individual laboratory facilities

Options I & II pertain to the basic alternative treatment systems while options A, B, and C pertain to alternative laboratory facilities. Although six (6) combined alternatives are possible, only five (5) are discussed in detail. Option I-C (Regional WWTP & maintain individual labs) is omitted because it would not be reasonable. If a regional treatment facility is constructed, then a single, suitable wastewater laboratory facility should be provided to service the new wastewater treatment plant (Option I-A). None of the three (3) existing wastewater laboratory facilities would be adequate in serving a new regional facility without modification.

A description of the five (5) investigated options are as follows:

OPTION LABEL *	DESCRIPTION
I-A	New regional wastewater treatment plant including new laboratory facility designed to provide testing for new wastewater treatment facility and all water treatment facilities.
I-B	New regional wastewater treatment plant with one of the existing wastewater laboratories expanded and upgraded to provide testing for new wastewater treatment facility and all water treatment facilities.
II-A	Maintain individual treatment facilities and construct new laboratory facility designed to provide testing for all wastewater and water treatment facilities.
II-B	Maintain existing individual treatment facilities with one of existing wastewater laboratories expanded and upgraded to provide testing for all wastewater and water treatment facilities.
II-C	No change to existing operations. Maintain existing individual treatment facilities and individual laboratories at wastewater and water treatment facilities.

** Label used for Option Comparisons in Table A*

EVALUATION PARAMETERS

A comparison of advantages and disadvantages of each alternative was made with regard to all the parameters, financial or otherwise, that were identified as having the potential to create a significant impact. A listing of these parameters and a brief description of each are as follows:

1.	Facility and Equipment:	Considers the financial impact of constructing and properly furnishing the facility.
2.	Staffing:	Considers the financial impact for total staffing requirements with regards to both quantity and quality of personnel.
3.	Control of Operations:	Considers the impact on each entity's control over day-to-day operations.
4.	Cost of Operations:	Considers the total cost of operation and the impact of determining an equitable proration of operational cost subject to different and variable testing requirements of each entity.
5.	Contract Testing:	Considers the financial impact on current and future testing performed by commercial labs.
6.	Operational Testing:	Considers the impact on the daily or routine testing performed at the treatment facility to assist in a treatment system's operations.
7.	Report Testing:	Considers the impact on those tests that are only for the purpose of reporting and are typically performed at lesser frequency.
8.	Permit Compliance Liability:	Considers the impact on each City's liability in regards to meeting regulatory agency requirements.
9.	Required Tests:	Considers the impact on specific aspects of testing requirements that may vary from system to system. Sub-categories consider are as follows: (See Tables B, C, & D) a. Treatment processes of each system b. Frequency of testing c. Permit limits d. Tests common to all entities or systems e. Tests unique to an entity or system f. Future tests requirements

As this list indicates there are many factors involved that are other than strictly financial. If cost was the only, or even the major consideration, then evaluation of the alternatives would be a matter of estimating the construction and operating cost of each option and selecting the lowest. However, each parameter is of significance and the degree of importance may vary from system to system. Therefore, the evaluation included herein rates each parameter for a specific option as either being an advantage, disadvantage, unknown or no significant difference.

EVALUATION OF ALTERNATIVES

Table A is a tabulation of all options and parameters being considered. Separate columns for water and wastewater are provided for each option since the impact for a given parameter could differ between the two systems. If a parameter has an "advantageous" impact for a given option the intersecting space is marked with an "O". A "disadvantage" is indicated by and "X", a "-" is used to indicate "no significant impact foreseen" and a "?" is used to indicate that it is "indeterminable at this time".

It should be noted that for many of the parameters where it is shown as an advantage for the wastewater system testing the opposite is the case for water system testing. The basic reason for this is that the majority of the tests run for the water system are "operational" tests. The results of these frequently run tests are used to gauge the water plant's operation and to determine what if any adjustments in the treatment processes are needed. This capability must be maintained for a given plant, thus some type of lab facilities must remain at each water treatment plant. To provide similar, duplicate facilities at a regional laboratory would not be cost effective.

DISCUSSION

Options I-A: Regional WWTP With A New Regional Lab and I-B: Regional WWTP With An Existing Lab Expanded To A Regional Lab. Overall Options I-A & I-B are very similar in advantage and disadvantages with only their degree of impact differing. Generally either option would be very advantageous for the wastewater systems but a large disadvantage for water systems. For the wastewater facilities the costs of staffing, operations, contract testing and in-house testing would all be lower than that required for the total of their individual labs. New facilities and equipment would be an added capital cost but it would not be logical to have a regional treatment plant without a common lab facility. This cost could be minimized by expanding an existing lab facility (Option I-B) but this would have the disadvantage of being at a location probably remote to the treatment facility. Providing for representative day-to-day control of operations for the three Citys and an equitable means of separating operation costs between the three systems would be a disadvantage because it is not required with separate systems. The liability for permit compliance should be lessened and total test requirements should be reduced as a result of having a single discharge with a single permit versus having four (4) discharges with four (4) separate permits.

A regional laboratory facility for the individual water treatment plants may have an advantage only in one of the parameters - future testing that may be required as a result of EPA's pending disinfectant/disinfectant by-product rule. If this regulation requires similar testing of each system then the regional lab would be the preferred method. However, due to dissimilarities in treatment facilities, techniques and processes, some type of separate requirements for each treatment system will probably result. Also, considering the possible highly technical aspects and complexity of any future test requirements, these tests may be best contracted out to commercial labs in the area. Until actual testing requirements for each system are defined the best alternative cannot be determined.

All other parameters would result in a definite disadvantage for the water systems because of the need for "on-site" operational testing at each water plant and the resulting duplication of capabilities. In addition, although the three water plants are subject to meeting the same basic water system limitations, there is sufficient difference in the individual systems and processes to create deviations in frequencies, allowable residuals and number of tests. With a regional lab, sampling and testing would have to be performed for each system individually and advantages such as economy of scale would not be realized. A further disadvantage would be that each City individually would be fully responsible for their system's compliance with water system regulations yet would not have complete control over the operations of the testing facility.

<u>Options II -A: Individual WWTP With A New Regional Lab</u> and <u>II-B: Individual WWTP With An Existing Lab Expanded To A Regional Lab</u>

Options II-A and II-B have all of the same advantages and disadvantages in regards to water systems as does Options I-A & I-B but with more disadvantages to the wastewater systems. Much for the same reasons discussed concerning individual water treatment plants with a regional lab; individual wastewater treatment plants will still need to maintain some lab facilities at each plant site for operational tests. Duplication of facilities, equipment, staffing, operation and in-house testing would result in additional cost. As with the water treatment system, there are similarities in the treatment processes of the three systems, but the dissimilar aspects result in different test requirements in terms of type and frequency. Some advantage in costs would be realized for the common BOD & TSS testing of each facility but the savings would not be significant. In addition, the same permit compliance liability problem would exist as with the individual water systems described previously.

Option II-C: Maintain Individual WWTP And Individual Laboratory Facilities

Option II-C is the most advantageous option for the water systems. With the need for daily on-site testing for plant operations, individual labs for each treatment plant provides the best conditions for the parameters evaluated.

For the wastewater systems this option has greater advantages than Options II-A or II-B but fewer advantages than Option I-A or I-B.

The fewer advantages result from the loss of the economy of scale factor for a single lab facility serving a regional plant plus having multiple permitted discharge points rather than only a single discharge point and single permit. The greater advantages in comparison to Options II-A & B are less a matter of economics and more a function of improved general operation control. The need to address only individual testing requirements and the improved conditions regarding permit compliance liability.

CONCLUSIONS AND RECOMMENDATIONS

The findings and recommendations of this evaluation are as follows:

1. Although economics is a major factor, other less tangible parameters are significant and must also be considered.

2. Any regional treatment facility should be served by an on-site laboratory facility.

3. Any individual treatment facility, water or wastewater, should be served by an individual on-site laboratory facility.

TABLE A
COMPARISON OF LABORATORY FACILITY OPTIONS

EVALUATION PARAMETER	I-A Regional WWTP With New Regional Lab		I-B Regional WWTP With Existing Lab as Regional		II-A Individual WWTP With New Regional Lab		II-B Individual WWTP With Existing Lab as Regional		II-C Individual WWTP With Individual Labs	
	Water	Wastewater	Water	Wastewater	Water	Wastewater	Water	Wastewater	Water	Wastewater
1. Facility & Equipment	X	O	X	O	X	X	X	X	O	–
2. Staffing	X	O	X	O	X	O	X	O	O	X
3. Control of Operations *	X	X	X	X	X	X	X	X	O	O
4. Cost of Operations *										
- Total	X	O	X	O	X	X	X	X	O	X
- Proration	X	X	X	X	X	X	X	X	O	O
5. Contract Testing	O	O	O	O	O	O	O	O	X	X
6. Operational Testing	X	O	X	O	X	X	X	X	O	O
7. Report Testing	X	O	X	O	X	X	X	X	O	O
8. Permit Compliance Liab.*	X	O	X	O	X	X	X	X	O	X
9. Required Tests										
a. Treatment Processes	X	O	X	O	X	X	X	X	O	O
b. Frequency	X	O	X	O	X	X	X	X	O	X
c. Permit Limits	X	O	X	O	X	X	X	X	O	X
d. Common	X	O	X	O	X	O	X	O	O	X
e. Unique	X	O	X	O	X	X	X	X	O	O
f. Future	?	O	?	O	?	X	?	X	?	X

* *These items are in reference to each City individually. Other comparisons are in reference to total operations.*

TABLE B
LIST OF TESTING PERFORMED
CITY OF GROVES

TEST PARAMETER	TYPE OF TEST		TYPE OF LAB		
	Operations	Report	City	Commercial	State
WASTEWATER:					
BOD	✓	✓	✓		
TSS	✓	✓	✓		
pH	✓	✓	✓		
Cl Residual	✓	✓	✓		
DO	✓	✓	✓		
Fecal	✓			✓	
TCLP		✓		✓	
Bio Monitoring					
Copper		✓		✓	
Sett. Solids					
WATER:					
Turbidity	✓	✓	✓		
pH	✓	✓	✓		
Cl Residual					
Cl Dioxide	✓	✓	✓		
Fluoride	✓	✓	✓		
TTHM	✓	✓			✓
Alkalinity	✓	✓	✓		
Hardness					
Temperature	✓	✓	✓		
Bacteriological	✓	✓		✓	
Salt					
Orthophosphat					
Lead					
Copper					
TSS					
DO					
Nitrate/Nitrite		✓			✓

TABLE C
LIST OF TESTING PERFORMED
CITY OF NEDERLAND

TEST PARAMETER	TYPE OF TEST		TYPE OF LAB		
	Operations	Report	City	Commercial	State
WASTEWATER:					
BOD	✓	✓		✓	
TSS	✓	✓		✓	
pH	✓	✓	✓	✓	
Cl Residual	✓	✓	✓		
DO			✓		
Fecal	✓			✓	
TCLP		✓		✓	
Bio Monitoring					
Copper					
Sett. Solids	✓		✓		
WATER:					
Turbidity	✓	✓	✓		
pH	✓	✓	✓		
Cl Residual	✓	✓	✓		
Cl Dioxide					
Fluoride	✓	✓	✓		
TTHM	✓	✓			✓
Alkalinity	✓	✓	✓		
Hardness					
Temperature	✓	✓	✓		
Bacteriological	✓	✓		✓	
Salt					
Orthophosphate					
Lead					
Copper					
TSS					
DO					
Nitrate/Nitrite					

TABLE D
LIST OF TESTING PERFORMED
CITY OF PORT NECHES

TEST PARAMETER	TYPE OF TEST		TYPE OF LAB		
	Operations	Report	City	Commercial	State
WASTEWATER:					
BOD	✓	✓			
TSS	✓	✓			
pH	✓	✓			
Cl Residual	✓	✓			
DO	✓	✓			
Fecal	✓			✓	
TCLP	✓			✓	
Bio Monitoring		✓			✓
Copper	✓			✓	
Sett. Solids					
WATER:					
Turbidity	✓	✓	✓		
pH	✓	✓	✓		
Cl Residual	✓	✓	✓		
Cl Dioxide			✓		
Fluoride	✓	✓	✓		
TTHM	✓	✓			✓
Alkalinity	✓	✓	✓		
Hardness	✓		✓		
Temperature	✓	✓	✓		
Bacteriological	✓	✓		✓	
Salt	✓		✓		
Orthophosphate	✓	✓	✓		
Lead	✓	✓		✓	
Copper	✓	✓		✓	
TSS	✓	✓	✓		
DO					

SECTION 5-ENVIRONMENTAL ASSESSMENT

Task II. Develop Environmental Assessment for the Planning Area

A. DESCRIPTION OF STUDY AREA

The planning area for the regional wastewater study consists of three adjacent Jefferson County cities along with their wastewater service areas: Nederland, Port Neches, and Groves. Also construed as part of the study area are all potential project elements which may lie outside the actual service areas. All such elements lie in Jefferson County within a few miles of the three cities.

The three cities are located between Beaumont and Port Arthur. The 1994 populations of the cities are estimated at 16,549; 13,479; and 16,967 respectively.

B. CURRENTLY EXISTING ENVIRONMENT WITHOUT THE PROPOSED PROJECT

(See Appendix E for details.)

1. Geological Elements

   a. Topography. The study area lies in the Gulf Prairies and Marshes. The planning area is generally flat with elevations no more than 20 feet. The planning area is bounded on its northeast side by the Neches River and adjacent marshlands. The planning area itself consists largely of solid residential areas, along with open areas, industrial plants/waste sites, and the Jefferson County Airport. The planning area lies inside the drainage basins of the Neches River and Taylor Bayou.

   b. Soil Types. Most of the study area lies in the Beaumont-Morey association. Other associations in the area include the Morey-Crowley-Hockley association, the Salt water marsh-Tidal marsh association, and the Harris-Made land association.

   Most soils in the study area are clay, acid soils with poor internal drainage. The soils where some project elements may be located (spoil areas or marsh) are saline.

   The Groves South Plant is in the Harris-Made land association. All other existing wastewater treatment plants are in the Beaumont-Morey association. Various potential project elements fall in the Morey-Crowley-Hockley, Beaumont-Morey, and Harris-Made land associations. Outfall lines from most potential plants also pass through the Salt water marsh-Tidal marsh association for at least a short distance. See the Environmental Information following this text for details.

   The soils are relatively impermeable, except for surface layers in the Salt water marsh. Because of the flat topography, erosion is not a major problem. Prime agricultural land is not a consideration in the study area.

c. Geologic Structures. The soils in the Gulf Coast Region are underlain by sedimentary material for several thousand feet below the surface, consisting of Pleistocene, Holocene, and Modern formations. The Pleistocene deposits underlie almost all of Jefferson County.

The geological formations crop out in belts parallel to the coast and dip toward the Gulf at angles much steeper than the land surface, with the older (lower) formations dipping more steeply. The most important aquifer in the Jefferson County area is the Chicot Aquifer.

2. Hydrological Elements

a. Receiving Streams. The existing Nederland, Port Neches, and Groves North plants all discharge into various branches of the drainage network owned and maintained by Jefferson County Drainage District 7. The ditches are located inside an area protected from hurricane surges by a Corps of Engineers levee. All ditches receiving effluent from those plants lead to a DD7 pump station which pumps the flows into the east distributary branch of Taylor Bayou. That stream flows into the Intracoastal Waterway (Segment 0702 of Neches-Trinity Coastal Basin).

The ditch system also receives various industrial discharges, including approximately 10-20 mgd from the Star Enterprise refinery at Port Arthur.

For each of the domestic plants above, the concrete lined ditches at the discharge points are classified as intermittent, with the downstream ditches being perennial.

Stream flow in the ditches, although adequate to prevent upstream flooding, is reportedly cyclic at some locations because of intermittent pump operation. The resulting backwater problem has caused problems for several dischargers, including the City of Port Neches.

The permits for the plants above previously allowed secondary effluent limits (20 mg/l for BOD_5 and suspended solids, no ammonia limit). However, the TNRCC performed stream studies of the ditch system several years ago. Consequently, the permits issued for Nederland and Port Neches within the last two years called for no discharge beginning within three years after permit issuance. The most favorable future limits provided by variances would require tertiary treatment (5 mg/l for BOD_5 and suspended solids, 2 mg/l ammonia). The Groves North Plant, which is up for renewal in 1995, faces similar limits in its draft permit, but the city has asked for a variance.

The series of ditches was presumed to be suitable for high quality aquatic life until even more recent studies resulted in a downgrading to intermediate quality aquatic life. The Intracoastal Waterway is designated for contact recreation and high quality aquatic life.

· The existing Groves South Plant discharges into the Sabine-Neches Canal running between Port Arthur and Pleasure Island. The canal serves as a ship/barge channel.

This saline segment is designated for contact recreation and high quality aquatic life. Secondary effluent standards are applicable and are expected to remain so for the foreseeable future.

· Several of the alternatives include diversion of wastewater flows to the Neches River. The Neches River is a relatively insensitive stream and therefore can receive secondary effluent without major problems.

The Neches River drains 10,100 square miles of East Texas. All potential river outfalls will fall in Segment 601 of the Neches, which is the lowermost segment of the stream. Segment 601 is tidally affected, with varying degrees of salinity. During certain times of the year, the river flow drops to an amount equal to the various surface water diversions in the area. Existing major improvements to Segment 601 include dredging of a ship channel to central Beaumont. A permanent salt water barrier just north of Interstate 10 is proposed as a replacement for the existing seasonal barrier.

The Neches River receives many domestic and industrial discharges throughout its basin. Nonpoint source pollution throughout the basin includes runoff from forest and agricultural land, urban runoff, occasional pipeline breaks, and septic tank leachate.

Desirable uses for Segment 601 are listed by the TNRCC as contact recreation and intermediate quality aquatic habitat. Other uses which are made include industrial cooling water and navigation.

Segment 601 is presently subject to secondary effluent standards, which are expected to remain in effect for the foreseeable future.

Some of the alternatives involve discharge of effluent from Nederland into Rhodair Gully, a local stream flowing into Taylor Bayou (Segment 701, Neches-Trinity Coastal Basin) west of Port Arthur. Other alternatives substitute Johns Gully, a local stream flowing into Hillebrandt Bayou (Segment 704), a tributary of Taylor Bayou. Rhodair Gully receives flows from several local domestic and industrial dischargers., while Johns Gully reportedly receives storm water from a tank farm.

The entire Taylor Bayou watershed, including Rhodair and Johns Gullies, is composed of sluggish coastal streams. Taylor Bayou (above tidal) is designated for contact recreation and intermediate quality aquatic life. It is anticipated that Rhodair Gully or Johns Gully will have advanced secondary effluent standards (10 mg/l BOD_5, 15 mg/l suspended solids, 3 mg/l ammonia) applicable to the City of Nederland.

b. Aquifers. Several aquifers underlie the Gulf Coast area and supply it with fresh water. The principal aquifer in the Jefferson County area is the Chicot Aquifer. Although this aquifer supplies large quantities of water in Hardin County (particularly for three large wells for the City of Beaumont), only small to moderate fresh water supplies can be obtained in Jefferson County. All of the cities and all or most industries in the study area take their water supply from the LNVA canal system.

c. LNVA Canal System. The LNVA operates a canal system throughout much of Jefferson County to supply irrigation, domestic, and industrial water. The intakes are located on the Neches River and on Pine Island Bayou, north of Beaumont. There is little or no need for irrigation water in the area from Beaumont to Port Arthur, but the canal system in this area supplies large amounts of domestic and industrial water. Major customers include the three cities in the planning area; the City of Port Arthur; and a number of industries including duPont, several chemical plants, and the Fina, Star Enterprise, and Chevron refineries.

d. Interbasin Transfer of Water. All water in the local LNVA canal system comes from the Neches River basin. That portion of the water supplying Nederland, Port Neches, and Groves is presently being returned to the Neches-Trinity Coastal Basin. This diverted water amounts to an annual average of approximately 4.3 mgd after excluding the amounts of effluent attributable to local infiltration/inflow.

The quantity of interbasin transfer will experience a net reduction as a result of the project, since diversions of flows to the Neches Rivers will outweigh the slight increases from population growth. Flows from two to four of the plants will be diverted to the river, depending on the alternatives selected.

No interbasin agreement is necessary for continuing, discontinuing, or modifying the interbasin transfer, since the LNVA has jurisdiction over both the Neches-Trinity Coastal Basin and the lower portion of the Neches.

3. Floodplains and Wetlands

The developed portions of Nederland, Port Neches, and Groves are above flooding or are protected by a flood levee, with all rainfall and effluent flows pumped across the levee at appropriate points. The levee protects all existing treatment plants for the three cities against the 100 year flood.

Floodplain areas within the planning area include a portion of the Nederland ETJ on the west side of U. S. 69 (along Rhodair Gully). Potential outside project elements in floodplains include portions of two potential Nederland plant sites, as well as potential outfalls into the Neches River.

Although the actual planning area contains few wetlands in developable areas, the adjacent area along the Neches River is covered with vast salt and brackish marshes. Also, the potential Nederland/Star Enterprise site contains marshes.

4. Climatic Elements

a. General. The climate of the study area is best described as being semitropical, with a mixture of tropical and temperate zone conditions. The mean annual relative humidity is approximately 83 per cent, while the average annual temperature at Port Arthur is about 69° F. The average rainfall for the study area is 60 inches. The prevailing wind direction is south-southeasterly, averaging 11 mph. Except during infrequent tropical disturbances and severe thunderstorms, the wind seldom exceeds 45 mph. Winter temperatures are exceptionally mild. The approximate dates of the first and last killing frosts are December 2 and March 2.

Summers are warm and humid, with a growing season averaging 250 days. The month of July has a mean temperature of 84°F.

Rainfall is abundant during the summer months. Thunderstorms are most frequent during July and August.

b. Air Quality. The Jefferson-Orange County area has been classified by the EPA as a nonattainment area for failure to meet the EPA ozone standards. Consequently, the area may have to begin vehicle emission testing in 1995 and faces possible additional future sanctions for continued noncompliance.

5. Biological Elements

a. Plant Communities. The study area falls within the Gulf Prairies and Marshes. The land appears to have been mainly open land before local residents planted trees around their homes. In the case of Groves, large areas were planted in pecan orchards and later developed into residential lots.

The undeveloped areas are generally open. The open marshland area between the planning area and the Neches River is largely covered with salt tolerant vegetation. The nearest large forested areas are located several miles away.

· The Sabine-Neches Estuary extends from the Gulf of Mexico to the salt water barrier on the Neches River. This area is recognized as a sensitive and unique ecosystem. The principal ecological areas are downstream from Beaumont, especially near Highway 87. Plant life along the estuary includes marsh grasses, tallow and willow trees, sedge, bulrush, and marshay millet.

b. Animal Communities. Animal life in open areas of Jefferson County includes ducks, quail, doves, geese, prairie chickens, raccoons, mink, squirrels, nutria, muskrats, and deer. Aquatic animal life in inland areas includes turtles, moccasins, frogs, and alligators.

· Aquatic life in the Sabine-Neches Estuary includes gar, mullet, crabs, blue catfish, saltwater catfish, shrimp, croakers, common water snakes, and Rangia cuneata (a brackish clam). Land animals include nutria, muskrats, raccoons, opossums, rats, mice, beavers, skunks, and moccasins. The estuary contains over 200 species of birds, over half of which are aquatic species. Birdlife includes cranes, rails, snipes, herons, egrets, ducks, coots, gulls, terns, and waders.

c. Habitats of Endangered Species. During August 1994, the Engineer contacted various agencies, including state and federal wildlife agencies. The U. S. Fish and Wildlife Service indicated no federally listed or proposed threatened or endangered species near the study area. The Texas Parks and Wildlife Department sent lists of endangered or threatened species possibly occurring in Jefferson County. A closer review can be performed after project alternatives are narrowed.

The bald eagle, listed in previous environmental reports for the area, tends to winter along major rivers and reservoirs, possibly including the Neches River.

6. Cultural Resources. Several agencies were contacted in August 1994 regarding cultural or historic resources. None of these agencies has responded, but the agencies can be contacted again once the scope of the project has been better defined.

Cultural remains (from Indian villages) can be expected mainly along major watercourses (in this case, the Neches River) according to a previous TWDB report for a Beaumont project. However, many cultural remains along the river may have been disturbed over the years in the course of repeated channel dredging and other activities.

The Spindletop Oil Field, several miles to the northwest of the study area, is included in a National Historic Landmark along with the Lucas Gusher. Points of interest include several museums in Nederland and Port Neches. Various recreational opportunities can be found in the three cities and within driving distance.

7. Economic Conditions. Nederland, Port Neches, and Groves make up the Midcounty area within the Golden Triangle (Beaumont, Port Arthur, and Orange). The side extending from Beaumont to Port Arthur, including Midcounty, is a highly industrialized area extending the length of eastern Jefferson County in a broad strip. Industries in the area include petroleum refining, chemical and plastics industries, paper mills, shipyards, and a steel mill.

In recent years, a portion of Jefferson County south of Beaumont has become the home of various state, federal, and county correctional facilities with an ultimate capacity of 12,000 inmates.

Agriculture in the Midcounty area is almost nonexistent.

For Jefferson County, the per capita income for 1989 was $16,375. Average weekly wage rate was $446.53 in 1990, with retail sales over $1.8 billion and tax value over $10 billion.

The petroleum industry was born in Jefferson County at the beginning of the century. Over the years the area became highly dependent on the oil industry and various related industries. The local economic growth reached its peak in the early 1980's during a period of high demand for oil and refined products. However, after a worldwide reduction in demand for fuel, local refineries cancelled expansion plans and laid off thousands of workers. Local shipyards declined, and oil prices subsequently fell.

Local employment has subsequently improved, despite several additional plant closings and cutbacks. Factors contributing to improved conditions include diversification efforts, the growth of service industries, tax abatements, plant construction for environmental purposes, and the selection of Jefferson County for state and federal prison facilities.

Transportation facilities serving Midcounty include the Jefferson County Airport, various highways and railroads, ship channels, and the Intracoastal Waterway.

Education is provided by the Nederland, Port Neches-Groves, and Port Arthur school districts, nearby parochial schools, and Lamar University. General hospitals include one each in Nederland and Groves, as well as two in nearby Port Arthur and three in Beaumont.

The 1994 city populations are estimated at 16,549 for Nederland, 13,479 for Port Neches, and 16,967 for Groves. Projected populations in 2009 are 24,240, 14,517, and 17,538. For 2024, the projected city populations are 24,816, 15,040, and 17,794. Sewered populations are close to city populations for Port Neches and Groves. Population served by the Nederland sewer system is estimated at 17,650 for 1994, 18,674 for 2009, and 19,127 for 2024.

8. Land Use. All three cities have zoning, with actual current land use as follows:

- Approximately 90% of the existing City of Nederland is residential, with the remainder commercial, public, and a relatively small amount of vacant land. The outside service area (existing and potential) includes the Jefferson County Airport, large amounts of vacant developable land, several industrial sites, and small amounts of existing residential and commercial development.

- The portion of the City of Port Neches within the planning area is approximately 50% residential, 10% commercial and public, 10% vacant developable, and the remainder undevelopable industrial waste sites. There are substantial amounts of industrial sites included in the planning area, surrounded by the City.

- Approximately 90% of the City of Groves is residential, with the remainder commercial, public, and a minor amount of vacant land. The Groves service area includes a residential area within Port Arthur.

- Groves and Port Neches have only limited space for future growth. Nederland has substantial capacity, having annexed a corridor around several square miles of its ETJ.

9. Other Programs

a. Economic Development. A number of privately and publicly sponsored programs were developed in Southeast Texas in the late 1980's for the purpose of attracting new industries to the area. Some of the programs for attracting new industry included a low interest loan programs; City revolving loan funds (in Beaumont and Port Arthur) for small businesses; several job training programs; tax abatements; and agencies providing various information to new or expanding businesses.

The county and several local governments submitted a proposal within the last two years for a state prison location on a site between Beaumont and Port Arthur. One prison unit is already in service, with others nearing completion or scheduled within the next few years. Similar proposals were submitted to the federal government, and a 4000 bed federal prison is under construction west of the state facilities.

Other recent programs for economic development include establishment of foreign trade zones, enterprise zones, and economic redevelopment zones.

Job creation from these programs could induce the Southeast Texas area to grow beyond the peak population which was reached in the 1980's, affecting the sizing of the necessary wastewater system improvements for the three cities in the planning area. The TWDB has prepared a draft of revised (increased) population projections for Jefferson County and for the three cities.

The size of the communities does not control the basic need for the improvements. The work is necessary for such reasons as new stringent stream standards, excessive infiltration/inflow, and deteriorating condition of several treatment plant units.

b. Drainage. Drainage for the three cities in the study, as well as the Port Arthur area, is enhanced by the efforts of Jefferson County Drainage District No. 7. The District operates a network of improved drainage ditches, many of which are concrete lined. Surrounding the urbanized area on three sides is a storm levee constructed by the Corps of Engineers to protect against the effects of hurricane tidal surges. The drainage system takes the local storm water to various points just inside the levee and then pumps it to the opposite side of the levee.

The intermittent operation of the existing pump station serving the planning area has resulted in cyclic high levels in the lateral ditch which receives effluent from the Port Neches and Groves North Plants. The high stream levels create hydraulic problems in the Port Neches plant, thus reducing effective flow capacity. The District has been seeking funding to upgrade its pumping facilities to eliminate this problem.

c. Miscellaneous Programs. A master plan for future westward highway loop extensions has been prepared. The future highways would link the Midcounty area with Interstate 10 to Houston. Imminent widening of State Highway 73 west of Port Arthur will also improve access to the area.

Other programs which contribute to the quality of life in the Midcounty cities include low rent housing programs; mosquito control by a county agency; and the higher education provided at Beaumont and Port Arthur by Lamar University.

C. PRIMARY IMPACTS OF VARIOUS ALTERNATIVES

1. Short Term Impacts

a. Alterations to Land Forms, Streams, Drainage Patterns

(1) Collection System, Transfer Lines, and Outfalls. Any linework *(except boring, tunnelling, and some overhead crossings)* will temporarily alter the ground surface and any streams crossed. Local drainage patterns will often be disturbed, including temporary impediments to small ditches and streams. However, contractors will normally be required to restore existing conditions.

Stream and canal crossings will be designed to have little or no permanent effect on stream flow. Pipe supports will be located outside the streams or located/designed to minimize erosion and flow impediment.

Permanent impact should be minor for any linework alternatives.

(2) Treatment Plant Construction. Any new treatment units or modification to existing structures may require small amounts of sitework. Plant access roads will be required for new plant sites and in some cases for treatment plant improvements. Other permanent alterations in land forms *(other than in cases of lagoons)* should be minor. Trenching operation for yard piping will cause only temporary alterations. Any drainage pattern alterations *(except for lagoons)* will be minor.

Any lagoon construction will involve considerable amounts of levee work and probably several feet of excavation over the lagoon area. Drainage patterns may be altered considerably within the site. Also, unless the in situ clay meets impermeability requirements, undercut and replacement with a clay or synthetic liner will also be needed. A clay liner may require large amounts of borrow excavation from offsite.

Alterations would be substantial for lagoon alternatives, minor for conventional plant work.

(3) Wetland Construction. Wetland construction (under consideration for only Nederland) would involve levee work over an effective area of approximately 20 to 20 acres (60 for a joint facility with Star Enterprise). Some local drainageways within cells may be required. No investigation has been made into impacts such relocated drainage across the site, access roads, clay borrow sources, etc.

Wetland construction would have much broader impacts than conventional or lagoon construction. A wetland following a lagoon would be larger and have more impact than a wetland following a conventional plant. A wetland operated jointly with Star Enterprise should be even larger with more impacts because of the added volume of partially treated influent from Star.

b. Siltation and Sedimentation. Siltation and sedimentation could occur temporarily and locally in the drainage patterns of the project areas pending revegetation.

Control measures for treatment plant construction will be covered to a large extent by the required Pollution Prevention Plan and may include silt curtains, hay bales, salvaging/replacing topsoil, reseeding, and scheduling operations for favorable weather. All potential plant sites lie in flat areas, thus minimizing the risk of erosion.

Measures for the collection system and other linework will be similar. Additionally, ditch crossings will be sodded and/or covered with riprap as necessary. Headwalls will be placed around outfall lines if necessary.

In the event of wetland and/or lagoon construction, control measures would be similar to those for plant work and could include terraces around the work area if necessary.

Siltation/sedimentation, despite control measures, is potentially much higher for lagoons and wetlands than for conventional plants.

c. Effects of Construction on Area Watercourses. The linework, as well as yard piping in plants, will require large amounts of trenching throughout the construction period. Some temporary and minor siltation of watercourses is expected. Any stream crossing requiring pipe supports in an unlined stream will involve some siltation.

· Some boring and/or tunnelling is anticipated for the linework, but it should not affect watercourses unless soil from the bore pits washes into ditches or streams.

Mitigative measures, in addition to those discussed in subsection b above, may include scheduling for dry weather and low stream flow; possible isolation of the crossing area by sandbags; and location of equipment outside the stream.

· Dredging will be required for all outfalls to the Neches River. Such dredging will be conducted according to requirements of the Corps of Engineers and/or any other agencies with jurisdiction. It is anticipated at this time that the effluent will pass through a pipe buried underneath the river bed and enter the river through multiple riser pipes. Discharge points will be located within the stream to meet agency requirements. If the river cross section is stepped at the outfall locations as a result of ship channel construction, the discharge point will probably be located on an intermediate (shelf) level.

If the agencies do not allow dredging within a flowing river, the outfall zone would probably be isolated by a cofferdam during construction. Construction equipment may be placed on boards or mats. The river bed (especially any gravel or rubble bars) would be restored to preconstruction conditions, with excess excavation removed. The area within the cofferdam would then be refilled slowly with water before removing the cofferdam.

- No investigation has been made as to the need for dredging for outfalls to the Sabine-Neches Canal. However, it is anticipated that any such outfalls can be similar to the existing Groves South outfall, in which the effluent discharges into an existing open ditch just inside the hurricane levee which parallels the canal. Otherwise, the effluent could be pumped to the opposite side of the levee.

In summary, effects on area watercourses would be minor except for outfalls to the Neches River. Mitigative measures would be employed for such outfalls so as to minimize temporary impacts and make permanent impacts negligible.

d. Injury to Cover Vegetation. Vegetation must be removed from construction areas, but the areas will be restored where not covered by permanent improvements such as structures, roadways, lagoons, wetland cells, etc. Care will be taken to minimize destruction to adjacent tree roots.

   Vegetation from any lagoons will be disposed of. It is anticipated that wetland cell bottoms will be mowed before planting in wetland species. Wetland plants will be salvaged only if they are of the right species for use in the cells.

   Any rare or endangered species found in a construction area will be considered for preservation by transplanting or design modifications.

   Permanent injury to cover vegetation would be least for conventional treatment plants and linework, and would increase for lagoon and wetland construction according to the acreage involved.

e. Herbicides, Defoliants, Cutting, Burning. Clearing will not involve herbicides or defoliants. Large amounts of cutting are not expected because of the open nature of the area *(except possibly in the Neches River marshes)*. Burning, if applicable, will be conducted according to TNRCC regulations for areas within and outside cities.

   Cutting and/or burning would be minor for conventional treatment plants and for linework in developed areas, increasing for linework in undeveloped areas.

f. Disposal of Soil and Vegetative Spoil. Any excess linework excavation which cannot be spread along the route must be removed, but can probably be placed on nearby vacant land or construction sites *(excluding lines within the river or adjacent marshes)*. Excess soil from plant construction (including lagoons or wetlands) can probably be placed within the site. Excess excavation from the river bed or marshes must be removed to a location outside wetland areas.

Vegetative spoil, if not placed within unused portions of plant sites, can be disposed of in a commercial landfill south of Beaumont.

Excess soil which must be disposed of offsite will increase with the amount of linework. Several miles of linework will be involved in any alternatives for Port Neches and Groves. Nederland will have approximately 2½ miles of linework for collection system improvements. Additionally, Nederland will have several miles of transfer or outfall line under any alternative except retaining the existing discharge point.

Vegetative spoil disposal, although associated with any linework, would be at a minimum in developed portions of cities, increasing in undeveloped areas, and at a maximum in marsh areas. For plant work, it is least for work within existing plant sites, greater for most new conventional plant sites, and maximized for lagoons and wetlands.

g. Land Acquisition.

(1) Amount to be Acquired.

None of the alternatives involves relocation of people.

Various treatment plant alternatives would require the following amounts of land as a rough estimate:

Nederland wetland -- 200 acres.
Nederland/Star Enterprise wetland -- 600 acres.
Nederland lagoon/wetland -- 300 acres
Nederland existing plant expansion -- 5 to 10 acres
Groves North -- 4.5 acres plus over 2 acres buffer easement
Groves South -- 5 to 8 acres
Groves Regional -- 15 to 20 acres plus buffer easements
Three City Regional -- 60 acres ±
Two City Regional -- 40 acres ±

For Nederland, collection system easements may be minimal because of the presence of City streets. The transfer line to the wetland or lagoon/wetland to the west would require roughly 0.4 to 3 miles of easements for Site A and 3 to 5.5 miles for Site B, depending on whether a line of that size could be placed in highway ROW. The transfer line to the joint facility with Star Enterprise would require up to four miles of easements unless it could be placed in highway ROW. The outfall to the Neches River would require roughly two miles of easements.

The proposed outfall force main from the Port Neches and/or Groves North Plants to the Neches River will require from 0.4 to 3.5 miles of easements, depending on which route is selected and whether the force main can be located within street and highway ROW.

The various Groves alternatives require 1½ to 6 miles of force main (exclusive of the force main already required from Port Neches to the River). For all options, other than the Groves regional plant on the east side, there is a chance of locating all of this force mains in street and highway ROW. However, the outfall from that plant would require at least 0.8 miles of easement unless it is routed to the Sabine-Neches Canal by a more lengthy route.

The regional plant would require approximately 1.2 miles of easements, assuming that all transfer lines can be routed along street and highway easements to a point near the plant.

(2) Method of Acquisition. The plant sites and linework easements will be acquired according to the Uniform Relocation and Assistance Act of 1970. Eminent domain will be exercised only if necessary. Existing improvements will remain undisturbed as much as practical.

(3) Effects on Adjacent Land Values. Little effect on adjacent land values along linework routes is expected. The same is true for land adjacent to plant sites (including any lagoons or wetlands) with the possible exception of any buffer easements outside the sites. *(Such easements, prohibiting residential construction, must provide a buffer totalling 150 feet outside treatment units, or 500 feet for anaerobic lagoons.)*

Land values in areas now subject to overflows could be improved slightly.

h. Abandonment of Facilities. Several of the Nederland alternatives include abandonment of the existing treatment plant except for the lift station. The two alternatives for converting the plant to all activated sludge would involve abandonment of trickling filters and possibly associated units. For those alternatives retaining the plant with a supplementary wetland offsite, all units may be retained, but selected pumps and piping would be abandoned in favor of larger facilities.

An upgrading of the trickling filter and activated sludge processes would likewise involve little or no abandonment of existing units. The outfall into the adjacent ditch would be abandoned in every case except for the most extensive plant upgrading, unless the TNRCC should allow its use for peak wet weather flows.

For Port Neches, no treatment units will be abandoned except in the alternative for a three-city regional plant. The outfall into the adjacent ditch will be abandoned in any case, unless the TNRCC should allow its use for peak wet weather flows.

Most alternatives for Groves involve abandonment of one or both treatment plants, except for the lift station for the North Plant. In some cases, existing units would be replaced by new units on an expanded plant site. The alternatives for upgrading the trickling filter process for the two plants involve removal of existing rock media and replacement by synthetic media, as well as possibly abandoning existing anaerobic digester equipment.

The outfall into the ditch by the North Plant will be abandoned in any case, unless used for wet weather flows. The influent and outfall force mains for the South Plant will be abandoned only in case of a regional plant away from the South Plant.

In any case of a three-city regional plant, all existing plants would be abandoned except for lift stations, and all existing outfalls plus the Groves South Plant influent force main would also be abandoned. For a two-city regional plant, the Port Neches plant would be retained but its outfall possibly abandoned.

i. Bypassing of Sewage. All existing collection system overflows will be eliminated, either through the project(s) or through concurrent efforts by the cities. None of the existing plants has bypass provisions, nor will bypassing be included in any of the various treatment alternatives.

Work sequences for any plant or collection system upgrading will be arranged to preclude construction related bypassing. No existing unit will be taken out service unless a backup unit or a replacement is ready for use. No existing plant will be abandoned until a replacement plant and related transportation segments are permanently operable and have gone through startup.

j. Construction in Waterways. The Corps of Engineers has been contacted regarding the possible need for river outfall permits. The reply to date is that the river outfall(s) will require Section 10 and Section 404 permits. However, the Corps indicated the possibility of coverage under a nationwide permit in lieu of an individual permit. The possible need for a permit for plant construction will depend on the plant site location(s).

The Corps indicated also that no permit would be required for an outfall into Rhodair Gully north of Farm Road 365. Presumably a similar determination would apply to John's Gully. The Corps was not contacted with regard to a new outfall to the Sabine-Neches Canal, since this alternative surfaced late in the study. However, the alternative associated with such an outfall was rejected for economic reasons. For an increased flow capacity for the Groves South Plant, no improvements to the existing structure through the levee into the canal are anticipated.

k. Dust Control. Dust problems are unlikely for any project elements. If necessary, construction areas can be watered in dry weather.

l. Noise. Normal construction noise will be a short term nuisance in the immediate vicinity. Noise will occur in residential and commercial areas, along highways, and also in remote areas. OSHA requirements, including mufflers, should protect residents and wildlife.

m. Blasting. No blasting should be required.

n. Safety Provisions. Construction within plant sites and along some linework routes will not interfere with vehicular or pedestrian traffic. If heavy construction traffic causes problems on roads leading to the sites, or in cases of linework along travelled roads, standard safety precautions will be taken such as barricades, warning signs, etc. Parking of construction vehicles will be kept away from heavy traffic or sensitive areas as much as possible.

   Open trenches will be closed as soon as possible or barricaded to prevent accidental entry. If necessary, pedestrian walkways will be provided.

   The relatively inaccessible locations of most plant sites will tend to keep the public away. Other measures such as warning signs, fences, and locked gates will be used as needed.

o. Night Work. Night work will occur only in special cases such as agency-imposed deadlines; need to restore a unit to service quickly; or sewer rehabilitation requiring minimum flow conditions. Effects of the resulting noise will be minimized by noise control measures or remote locations as appropriate.

p. Effects on Existing Utilities. Owners of all utilities crossing linework routes or plant sites will be notified well in advance of construction. Pipeline owners will be contacted to determine pipeline depths, avert damage, and arrange for any necessary adjustments.

2. Long Term Impacts

   a. Land Affected, Beneficial Uses. Amounts of land required for various treatment plant alternatives as well as lengths of linework are discussed in subsection C. 1. g (1) above.

Away from construction sites, land uses may be affected by slight improvement in developability as a result of adequate wastewater capacity. This future development is not expected to affect wetlands or prime agricultural land, or floodplains other than through infilling.

For Nederland, all alternative sites away from the existing plant are vacant and open. For the wetland which would be operated jointly with Star Enterprise, the area south of Highway 73 (which possibly could serve as part of the facility) is a portion of the land associated with the adjacent Star Enterprise refinery. This area is operated by Star Enterprise as a wildlife attraction.

The land required for expansion of the Nederland plant site includes some adjacent City property as well as small pastures which appear to be associated with nearby residences. Some of the City property is used for offices and equipment maintenance, while some City property is vacant. Other land which may be affected by buffer zone requirements includes an organizational meeting place and possibly similar pasture land on the opposite side of the drainage ditch.

The land for potential expansion of the Groves North Plant is vacant land owned by the Huntsman Corporation, owner of several local petrochemical plants. The land near the Groves South plant is vacant or industrial property. The potential site for the Groves regional plant on the east side of town is vacant and across the highway from the Fina refinery property. The intercity regional plant site is vacant.

For Nederland, the collection system work, most of the transfer line route to the Rhodair Gully site, approximately 40% of the transfer line route to the Johns Gully site, almost all of the transfer line route to the joint wetland with Star Enterprise, and approximately half of the outfall to the river follow existing street and highway routes. The remainder of the routes to Rhodair and Johns Gullys follows vacant, mostly open land parallel to existing utility and/or pipeline routes. The remainder of the outfall to the river is expected to cross an inactive oil refinery plus vacant marsh land.

b. Scenic Views. No scenic views should be affected. No landscaping, other than restoring existing surface conditions, is needed for any alternative.

c. Wind Patterns. Prevailing winds are described as being from the south-southeast, although the wind rose shows several prevalent directions.

The project will have no effect on any odors which may be produced at the Port Neches plant, since it does not include plant work. For Nederland, either wetland alternative would cause few odor problems because the wetland influent would be previously chlorinated, would enter in an relatively aerobic state, and would remain aerobic from oxygen diffused through wetland plant roots. The lagoon/wetland alternatives would create noticeable odor problems in the lagoons, but outside property would be protected by the required 500 foot buffer.

For Nederland or Groves, any plant expansion or relocated plant could present some odor problems, but such problems would be minimized by proper design and operation and would be very mild outside the plant sites.

Odors associated with collection system improvements would be very minor and may be outweighed by the elimination of wet-weather overflows.

No incineration is proposed in any sludge disposal methods.

d. Land Application. No land application of effluent is proposed. Sludge from all three cities is presently landfilled. Plant improvements for Nederland and Groves could render the sludge more suitable for land application.

e. Effects on Aquatic Life. The project should benefit the drainage ditch system by diverting all or part of the discharges and improving the quality of any remaining discharges. Rhodair Gully, or Johns Gully, if selected as a receiving stream, may suffer slightly, although the effects will be minimized by advanced secondary treatment. The Neches River (to which some flows will be diverted) and the Sabine-Neches Canal should not be affected measurably, since they already carry large quantities of industrial effluent.

Any drainageways which now experience periodic overflows or bypasses will benefit through elimination of such events.

Some species of aquatic life may thrive in any wetland facilities which may be constructed.

f. Effects on Water Uses. By reducing the amount of pollutants discharged or bypassed into the drainage ditch system, the project should benefit any downstream recreational usage of the waters. Rhodair Gully or Johns Gully could suffer slight adverse effects, but the level of treatment would minimize such effects. No effects on the Neches River or the Sabine-Neches Canal are anticipated.

g. Diversion of Flows. The 4.3 mgd now being diverted from the Neches basin would be reduced to 2.6 mgd or 0.8 mgd, depending on whether the Nederland and Groves North flows are redirected to the Neches. By the end of the 30 year planning period, these flows would increase to 2.8 mgd or 0.85 mgd. Note that if the TNRCC allows peak storm flows to continue to go to the ditch system, the amount of annual diversion will decrease by somewhat less than indicated.

h. Historical, Cultural, and Archeological Resources. Although no special investigation of any of the potential work areas has been made, the cities and the Engineer are not immediately aware of any historical or archeological resources in these areas.

The appropriate state historical agencies were notified of the study in its early stages and will notified of the selected alternatives at the appropriate time. The TWDB archeological staff may wish to conduct on-site surveys in connection with any state loan funding.

If any archeological resources are discovered during construction, work at the immediate site will be suspended pending archeological investigation.

i. Recreational Areas and Preserves. The joint Nederland/Star Enterprise wetland could, if selected, extend into a wildlife area on Star Enterprise property adjacent to its refinery. The industry presently operates this wildlife area on a voluntary basis. An examination of local maps shows no other recreational areas or preserves which could be affected by any alternative project elements.

j. Noise Levels. Main noise sources from existing plants are as follows:

· Nederland: Blowers and centrifuge, followed by pumps, aeration units, clarifiers, sludge thickener, and grit removal.

· Port Neches: Blowers and belt press, followed by pumps, clarifiers, and grit removal.

· Groves North: Pumps and clarifiers.

· Groves South: Blowers, followed by pumps and clarifiers.

For Nederland, any of the lagoon/wetland alternatives would eliminate all local plant noise except for the lift station, which may become louder. The wetland alternatives would essentially leave the plant as is except for adding a transfer lift station. The plant upgrading alternatives would increase the noise level by a varying amount, with the most noise form the 5/5 alternative and the least from the dual process alternative.

Offsite noise sources for various Nederland alternatives include a effluent lift station for the joint wetland with Star Enterprise and a new collection system lift station.

Noise for the Port Neches plant may increase by the construction of an effluent lift station. It is not certain whether the station will fall within the Port Neches plant, the Groves North Plant, or neither.

Noise for either of the Groves plants would increase to a moderate extent by upgrading the trickling filter process, mainly by adding aerobic digestion. The noise would increase much more from an expansion with activated sludge. Conversion of either plant to a Groves regional plant would maximize the noise at that plant, but would eliminate noise at the other plant (except for an upgraded transfer lift station in the case of the Groves North Plant). A regional plant in the east part of town would shift the noise location. The Taft Street Lift Station would have an increased noise level under any alternatives, but more so for any regional plant alternatives.

An intercity regional plant would constitute the greatest single source of noise, but it would be at an isolated location and would eliminate either three or all of four of the existing plants.

For any of the alternatives, noise problems would be minimized through mufflers, housing, or other design features.

k. Access Control. All existing and potential plant sites (including wetland and lagoon/ wetland sites) are (or will be) surrounded by fences with lockable gates. The isolated locations of some sites will also discourage trespassing.

l. Insect Nuisance. All existing plants contain trickling filters and clarifiers, which create psychoda fly problems, along with houseflies at skimmer troughs. However, the filter fly nuisance should be considerably less at the Nederland and Port Neches plants with synthetic media than at the two Groves plants with rock media.

Solids from bar screens and degritting mechanisms can also be present a horsefly problem, but the problem can be minimized by covering the materials until their removal from the site.

The trickling filters at Port Neches would remain in service under all alternatives except a three-city regional plant. The filters at Nederland would be eliminated under all alternatives except upgrading the plant similar to the existing dual process. The Groves filters would be either converted to synthetic media or eliminated in favor of activated sludge.

An intercity regional plant would eliminate trickling filters in two or all three cities.

Psychoda flies in the clarifier can be controlled with an occasional dose of chlorine, while houseflies can be controlled with lime.

Harmful insects in wetlands, especially mosquitoes, could be controlled by various non-chemical means, including periodic draining of cells. Pesticides could be applied, but their use must be according to federal regulations.

m. Floodplains. Flooding is no problem at any existing plants, since the sites are protected by the hurricane levee or above flooding. Portions of the potential Nederland plant sites on Rhodair Gully and Johns Gully are within floodplains, but can be protected by levees. Other potential plant sites are protected by the hurricane levee.

n. Air Quality. The proposed collection system upgrading/rehabilitation should improve air quality at points within the sewer systems slightly by eliminating the periodic overflows. Other linework should have no effect on air quality. All alternatives for treatment plant improvements should have little effect on air quality outside the respective sites.

o. <u>Energy and Chemical Consumption</u>. Energy consumption is much higher for activated sludge units than for trickling filters, facultative lagoons, or wetland units. Likewise, aerobic sludge digestion requires more energy than anaerobic digesters. Of the existing treatment plants, the most energy\intensive units are the activated sludge and aerobic digesters at Nederland. The various aerated grit chambers also entail significant energy use.

For the Nederland plant improvements, energy usage would be maximized by upgrading the existing plant with activated sludge to tertiary treatment standards, followed by an activated sludge plant providing an advanced secondary treatment level. Upgrading the activated sludge process while retaining the trickling filter track in parallel would increase energy usage to a lesser degree.

If the existing plant is retained and supplemented by a lagoon or wetland facility downstream, energy consumption would remain near the existing level with the addition of a transfer lift station. A complete lagoon or wetland facility would allow abandonment of the existing plant, but the transfer station would still be required.

For Port Neches, the addition of a transfer station pumping to the Neches River would be the only major increase in energy usage. In the case of plant expansion or a regional plant at the Groves North Plant site, the transfer station costs would increase but would be shared by Groves.

For Groves, the greatest energy usage would result from activated sludge treatment, whether at a Groves regional plant or by expansions of the North and South Plants. Energy usage for upgraded trickling filter processes at both plants or at a combined plant would result in a lesser increase in energy usage, with most of the increase due to changing from anaerobic to aerobic digestion.

A Groves regional plant may require slightly more energy than North and South Plants with the same process because of transfer pumping. Note also that effluent pumping would increase according to the distance to the receiving stream (farthest for North Plant, closer for east side, closest for South Plant).

The intercity regional plant, being activated sludge, may require more energy than other alternatives. For two cities, the energy usage would be reduced somewhat. The use of primary clarifiers may reduce the impact somewhat.

The Nederland collection system improvements, as well as any improvements to the Taft Avenue lift station in Groves for the South Plant, will require energy consumption for pumping.

For the various offsite plant options for Nederland, pumping would very according to distance from the existing plant.

For any plant alternative where discharge of peak flows to a nearby stream is a possibility, such a discharge would reduce pumping requirements.

· Chlorine and dechlorination agent usage, as well as polymer usage for sludge dewatering, will increase slightly for all alternatives except those involving lagoon/ wetland units as complete treatment plants. No chlorination or sludge processing would be required for those facilities.

p. Coastal Zones. All alternatives would be of some benefit to the coastal zone by reducing stream pollution.

q. Effects on Wildlife. Any constructed wetland units would attract much more wildlife than do the sites in their existing states. Wetland cells would primarily attract mammals, reptiles, and amphibians. If an open water area is also constructed on the site (as in the case of the Beaumont wetland), such an area would attract birds.

r. Effects on Utilities. Any large-area facility such as a lagoon or wetland plant would be designed to minimize any problems for existing pipelines, power lines, and canals crossing the site. All existing rights of protection contained in easement agreements would be honored. Coordination would be made with utility owners during construction.

D. SECONDARY IMPACTS OF VARIOUS ALTERNATIVES

1. Land Uses. The project can facilitate residential growth within the various cities by providing the necessary wastewater treatment capacity and, in some cases, transportation capacity. The project would allow the communities to make efficient use of various other facilities already available or programmed, such as water supply and highway improvements. Industrial growth could also possibly be stimulated, but availability of wastewater service is generally only a minor factor in the type of industrial development in the area.

   The amount of residential growth projected by the TWDB is relatively moderate, varying between 4.88% for Groves to 11.58% for Port Neches over the next 30 years. This growth will mainly occur by developing existing open land and by infilling of existing residential areas. There is little forested land in the developable areas of any of the three cities.

   Any construction of new plants, including lagoons or wetlands, should have little direct effect on neighboring land except for any land within the 150 foot or 500 foot buffer zones. Such land would require easements to prevent residential construction. However, any lagoon or wetland sites would contain all or most of such buffer zones within their boundaries.

   Treatment plant construction in underdeveloped areas could affect neighboring land development indirectly through fragmenting of the remaining available land.

2. Air Quality. Automobile usage within the planning area should increase somewhat from development. Such increase will be small in relation to existing local and through traffic. Possible new requirements for biannual emission testing would reduce the impact of automobile exhausts. It should be noted that automobile fumes are a relatively small source of air pollution in relation to industrial emissions. Also, much local air quality problems are suspected to result from air currents from the Houston area to the west.

3. Water Quality. Growth in the Midcounty area should have no effect on the quality of the water supply from the Neches River and Pine Island Bayou upstream from the area.

4. Effect on Public Services. Water usage will increase somewhat with growth, but the increase should be offset slightly by water conservation measures. The amount of increased usage should not present a major problem because of the large drainage area of the Neches River and the high rainfall within its basin.

5. Economic Impacts. The increase in user fees and/or taxes is expected to be within the residents' ability to pay. Section 8 of the main report contains information on the amount of increases in user fees.

6. Land Use Changes Versus Land Use Plans. Any future development within any of the cities will be in conformance with zoning plans.

7. Impacts of Growth on Sensitive Areas. No growth in floodplains other than infilling is anticipated from the project because of floodplain ordinances. Also, no development of land with significant wetland characteristics is expected, since each plat is scrutinized (by applicable local governments) for any local problems prohibitive to development.

   There are no known developable areas within the planning area comprising critical habitats, or environmentally sensitive, other than floodplains and wetlands.

ENVIRONMENTAL INFORMATION

ENVIRONMENTAL INFORMATION

A. DESCRIPTION OF STUDY AREA

The planning area for the regional wastewater study consists of three adjacent Jefferson County cities along with their wastewater service areas: Nederland, Port Neches, and Groves. Also construed as part of the study area are all potential project elements which may lie outside the actual service areas. All such elements lie in Jefferson County within a few miles of the three cities.

The three cities are located between Beaumont and Port Arthur. The 1994 populations of the cities are estimated at 16,549; 13,479; and 16,967 respectively.

The Nederland service area includes the existing City, the adjacent Jefferson County Airport, and several residential areas within the ETJ of the City. For planning purposes, the entire ETJ which the City plans to annex in the future is included, except for Jefferson County WCID No. 10, whose existing sewer system will remain in service.

The Port Neches service area includes the entire City *(exclusive of undevelopable Neches River marsh land)*, plus several industrial plants which are encircled by the City, some of which receive sewer service from the City. The Groves service area *(divided between the North and South Plants)* includes the entire City plus the Fairlea addition within Port Arthur.

Potential project elements outside the service areas include:

- The Groves South Plant (within Port Arthur).
- Two alternative sites for a new plant *(wetland or lagoon/wetland)* for Nederland.
- A potential wetland for Nederland as a joint venture with Star Enterprise (also within Port Arthur).
- Outfalls and portions of influent transfer facilities for the sites above.
- Part of the outfall from the existing Nederland plant to the Neches River.
- Portions of two of the three potential outfall routes from the Port Neches and/or Groves North plants to the river.
- Approximately half of the outfall from the Groves regional plant *(east side of town)*.
- Most of the outfall from the intercity regional plant.

B. CURRENTLY EXISTING ENVIRONMENT WITHOUT THE PROPOSED PROJECT

1. Geological Elements

a. Topography. The planning area lies in the Gulf Prairies and Marshes, a short distance south of the Piney Woods region.[1] The area is bounded for a short distance by the Neches River on its northeast side. The remainder of the northeast side is bounded by open marshlands within the Neches River floodplain. Other areas adjacent to the planning area include open land, industrial plants, Central Gardens (Jefferson County WCID No. 1), and the City of Port Arthur.

The planning area itself consists largely of solid residential areas in the three cities *(including a Port Arthur subdivision served by Groves)*. Much of these areas are covered with trees planted years ago, especially in Groves. Other portions of the planning area include open areas, industrial plants, industrial waste sites, and the Jefferson County Airport.

Natural ground elevations range from 4 to 20 feet above mean sea level, except that stream outfalls may be lower.[2] Jefferson County is flat with natural drainage divides poorly defined. Most of the county, including the planning area, lies inside the drainage basins of the Neches River and Taylor Bayou.

b. Soil Types

(1) Associations. According to a USDA soil survey released in 1965[3], most of the study area lies in the Beaumont-Morey association. The Morey-Crowley-Hockley association covers a small residential area in Nederland and Port Neches; some areas west of Nederland associated with potential treatment plant locations; and a narrow strip along the edge of the study area next to the Neches River marshes. The marshes themselves are in the Salt water marsh-Tidal marsh association. The part of Port Arthur extending from the Groves South Plant to its outfall, as well as the potential Nederland/Star Enterprise wetland site, are in the Harris-Made land association. Other potential elements fall in associations as shown in Table E-1.

All existing wastewater treatment plants other than the Groves South Plant are in the Beaumont-Morey association.

(2) General Characteristics. Most soils in the study area are clay, acid soils with poor internal drainage. The soils where some project elements may be located (spoil areas or marsh) are saline. See Table E-1 for further information.

The soils in Jefferson County are relatively impermeable. Permeability of surface layers in the study area varies from 0.05 to 2.5 inches per hour (except for the more permeable Salt water marsh). Below the surface this rate drops to zero to 0.8 inches per hour.

Because of the flat topography in Jefferson County, erosion is not a major problem. The only soil type noted for occasional erosion problems is that portion of the Acadia silt loam with 1% to 5% slopes. This soil occurs in a narrow strip along the boundary between the planning area and the Neches River marshes. This soil can be protected with a vegetative cover if necessary.

Soils with a high shrink-swell potential can also present potential erosion problems on steep slopes, as on the sides of ditches and embankments. Soils of this type within the study area include the Beaumont clay; the Harris clay; and subsurface layers of the Acadia, Crowley, and Morey silt loams and the Salt water marsh.

The soils at the existing Nederland, Port Neches, and Groves North plants are Morey silt loam, except for Beaumont clay in the north corner of the Port Neches plant. The Groves South Plant appears to fall partially in Harris clay and partially in Made land.

See Table E-2 for soil types at other project locations.

Prime agricultural land is not a consideration in the study area. The only soil types in the study area which can be classified as prime *(and then only if drained and not in an urban area)* are the Beaumont clay and the Morey silt loam[4]. Most portions of the study area with these soil types are within either incorporated areas, industrial sites, or developed residential areas. The only possible exceptions would be presently undeveloped portions of the Nederland ETJ, most of which are subject to future residential development regardless of whether the project is implemented.

c. Geologic Structures. The soils in the Gulf Coast Region are underlain by sedimentary material for several thousand feet below the surface. The sedimentary formations are divided into three major groups according to their dates of deposition: 1) Pleistocene, or during the glacial and interglacial periods of the ice age; 2) Holocene, during the irregular rise in sea level occurring after the ice age; and 3) Modern, during the last 4500 years of relatively stable sea level.

The Pleistocene deposits, particularly those deposited by the Trinity River, underlie almost all of Jefferson County. Where rivers cut through these deposits during times of lowered sea level, the eroded Pleistocene deposits have been replaced by Holocene and Modern deposits from these rivers. The Pleistocene formations generally underlie the soils of the coastal prairie, with Holocene and Modern formations occurring in floodplains and coastal strips.[5]

The formations crop out in belts parallel to the coast and dip toward the Gulf at angles much steeper than the land surface. The younger formations dip about twenty feet per mile. Since all the formations thicken downdip, the older formations dip more steeply. There are several aquifers underlying the Gulf Coast Region; the most important aquifer in the Jefferson County area is the Chicot Aquifer.[6]

Natural processes presently operating in the coastal regions include erosion, deposition, compaction, and subsidence. Measurable amounts of subsidence and sedimentation have occurred in the Jefferson County area in recent years, although the rate of subsidence is relatively minor.[5]

2. Hydrological Elements

a. Receiving Streams

(1) Drainage Ditches. The existing Nederland, Port Neches, and Groves North plants all discharge into various branches of the drainage network owned and maintained by Jefferson County Drainage District 7. Table E-3 lists the various receiving streams.

The ditches are all concrete lined at the discharge points, but downstream segments are unlined. The ditches are located inside an area protected from hurricane surges by a Corps of Engineers levee. All ditches drain to points just inside the levee, with all flows then pumped to the opposite side of the levee by DD7 pump stations. All ditches receiving effluent from the plants above lead to the Alligator Pump Station west of Port Arthur, which pumps the flows into the east distributary branch of Taylor Bayou. That stream flows into the Intracoastal Waterway (Segment 0702 of Neches-Trinity Coastal Basin).

The ditch system also receives various industrial discharges, including approximately 10-20 mgd from the Star Enterprise refinery at Port Arthur.

For each of the domestic plants above, the concrete lined ditches at the discharge points are classified as intermittent, with the downstream ditches being perennial.

Stream flow in the ditches, although adequate to prevent upstream flooding, is reportedly cyclic at some locations because of intermittent pump operation. During normal operation, the water is allowed to build up to considerable depth before the pumps come on. This backwater problem has caused problems for several dischargers, including the City of Port Neches.[7]

The ditch system was not a classified stream segment several years ago when the permits for the planning area were renewed. The permits allowed secondary effluent limits (20 mg/l for BOD_5 and suspended solids, no ammonia limit).

However, the TNRCC has performed subsequent stream studies of the system. Consequently, the draft permits recently issued for Nederland and Port Neches called for no discharge beginning three years after permit issuance. Even with the variances which the Cities were able to obtain on the basis of further studies, the future limits would require tertiary treatment (5 mg/l for BOD_5 and suspended solids, 2 mg/l ammonia). The Groves North Plant, which is up for renewal in 1995, has received a draft permit with similar limits.

The series of ditches is now presumed to be suitable for intermediate quality aquatic life (after an initial presumption of high quality aquatic life). The Intracoastal Waterway is designated for contact recreation and high quality aquatic life, and is used primarily for navigation.

(2) Sabine-Neches Canal. The existing Groves South Plant discharges into a portion of the Intracoastal Canal also known as the Sabine-Neches Canal. This deepened segment of the canal passes between Port Arthur on the mainland and Pleasure Island, a long, narrow island paralleling the edge of Sabine Lake.

The Sabine-Neches Canal was dredged earlier in the century along the edge of Sabine Lake (a natural lake near the coast receiving flows from the Sabine and Neches Rivers). The canal serves as a ship channel for various ports along the Neches in Jefferson County and the Sabine in Orange. The dredged material was placed in the lake just outside the canal to form Pleasure Island, which is used for recreational purposes.

Sabine Lake and the Sabine-Neches Canal, being subject to tidal influence, are saline. The segment is designated for contact recreation and high quality aquatic life. Its primary use is navigation. Secondary effluent standards are applicable to this segment and are expected to remain so for the foreseeable future.

(3) Neches River. Several of the alternatives include diversion of wastewater flows to the Neches River -- either through a regional plant or through outfalls from existing city plants. The Neches River is a less sensitive stream than any segments of the Taylor Bayou system and therefore can receive secondary effluent without major problems.

The Neches River is a major river draining 10,100 square miles of East Texas. All potential river outfalls will fall in Segment 601 of the Neches, the lowermost segment of the stream. Segment 601 is tidally affected, with varying degrees of salinity, and is included in the Sabine-Neches estuary. During certain times of the year, the river flow drops to an amount equal to the various surface water diversions in the area, making Segment 601 a no-flow segment.[8]

Existing major improvements to Segment 601 include dredging of a ship channel (currently 40 to 45 feet deep from the Gulf of Mexico to central Beaumont). A permanent salt water barrier just north of Interstate 10 is proposed as a replacement for the existing seasonal barrier.

The Neches River receives many domestic discharges throughout its basin. A number of industrial discharges also enter the river, the most significant of which are located in or near Segment 601. Nonpoint source pollution throughout the basin includes runoff from forest and agricultural land, urban runoff, occasional pipeline breaks, and septic tank leachate.

Desirable uses for Segment 601 are listed by the TNRCC as contact recreation and intermediate quality aquatic habitat. Other uses which are made include industrial cooling water and navigation. Major withdrawals of surface water just upstream from Segment 601 include two domestic intakes for the City of Beaumont and several pumping stations feeding into the Lower Neches Valley Authority (LNVA) canal system. The canals, which cover a large portion of Jefferson County, supply water for irrigation (including rice and soybean farming) and for domestic and industrial water supply (including water for the planning area). The intakes are located upstream from the local industrial waste discharge points.

Segment 601 is presently subject to secondary effluent standards (20 mg/l for BOD_5 and suspended solids, no ammonia limit). Secondary standards are expected to remain in effect for the foreseeable future.

(4) Rhodair Gully and Johns Gully. Some of the alternatives involve discharge of effluent from Nederland into Rhodair Gully or Johns Gully, two local streams. Rhodair Gully rises in Jefferson County a few miles north of Nederland and flows into Taylor Bayou (Segment 701, Neches-Trinity Coastal Basin) west of Port Arthur. The stream receives flows from several domestic and industrial dischargers, including the Jefferson County WCID No. 10. Johns Gullys within a tank farm several miles west of Nederland and flows into Hillebrandt Bayou (Segment 704), a tributary of Taylor Bayou. Available information shows one industrial discharge within the tank farm, probably storm water.

The entire Taylor Bayou watershed, including Rhodair and Johns Gullys, is composed of sluggish coastal streams. Taylor Bayou (above tidal) is designated for contact recreation and intermediate quality aquatic life.

It is anticipated that Rhodair Gully or Johns Gully will have advanced secondary effluent standards (10 mg/l BOD_5, 15 mg/l suspended solids, 3 mg/l ammonia) applicable to the City of Nederland.

b. Aquifers. Several aquifers underlie the Gulf Coast area and supply it with fresh water. In order from the oldest to the youngest, they are the Oakville Sandstone, sands in the Lagarto Clay, the Goliad Sand, the Willis Sand, the Lissie Formation and sands, and sands and gravels in the Recent alluvium.[6]

The principal aquifer in the Jefferson County area is the Chicot Aquifer which includes the Lissie and Willis formations. Although this aquifer supplies large quantities of water in Hardin County (particularly for three large wells for the City of Beaumont), only small to moderate fresh water supplies can be obtained in Jefferson County. Some industries in eastern Jefferson County reportedly use partially saline well water for cooling and firefighting purposes.[6] However, all of the cities and all or most industries in the study area take their water supply from the LNVA canal system.

c. LNVA Canal System. The LNVA operates a canal system throughout most of the north two thirds of Jefferson County to supply irrigation, domestic, and industrial water. The intakes are located on the Neches River and on its tributary, Pine Island Bayou, north of Beaumont. There is little or no need for irrigation water in the area from Beaumont to Port Arthur, but the canal system in this area supplies large amounts of domestic and industrial water. Major customers include the three cities in the planning area; the City of Port Arthur; Jefferson County WCID No. 10; and a number of industries including duPont, several chemical plants, and the Fina, Star Enterprise, and Chevron refineries

d. Interbasin Transfer of Water. All water in the local LNVA canal system comes from the Neches River basin. That portion of the water supplying Nederland, Port Neches, and Groves (except for the Groves South Plant) is presently being returned to the Taylor Bayou drainage area (Neches-Trinity Coastal Basin). This diverted water amounts to an annual average of approximately 3.5 mgd after excluding the amounts of effluent attributable to local infiltration/inflow. The flow from the Groves South Plant (0.81 mgd $\pm$ of return flow) enters the Sabine-Neches Canal, also classified as part of the Neches-Trinity Coastal Basin.

The interbasin transfer through the Port Neches plant will be limited to peak storm flows* under any of the alternatives, since most effluent from this plant will be rerouted to the Neches River. The transfer through the Nederland plant will (a) continue and increase slightly if the existing discharge point is retained, or if flow is diverted to Rhodair Gully or Johns Gully; or (b) be limited if the flow is diverted to the Neches River.

Transfer through the Groves North Plant will (a) be shifted to a Groves regional plant in the east or south part of town, or (b) be limited* in other cases. Transfer through the Groves South Plant will (a) continue in case of plant expansion or a Groves regional plant at this location; (b) be shifted to a Groves regional plant on the east side of town; or (c) be shifted and limited* in case of a Groves regional plant at the North site.

*To minimize costs of pumping treated effluent to the Neches River, it is proposed to pump only the flows up to the design *(maximum monthly average)* flows for each plant so diverted. Peak storm flows in excess of this amount will be discharged into the adjacent drainage ditch(es) as are all flows at present. The ditch system should be able to receive such flows without an unacceptable impact on aquatic life, since the effluent will be diluted by high flows from rainfall. If the city(ies) cannot negotiate this type of arrangement with the TNRCC, all flows from the plant(s) in question must be diverted to the river.

In the case of an intercity regional plant, alln transfer from three (or all four) of the plants will cease.

No interbasin agreement is necessary for continuing, discontinuing, or modifying the interbasin transfer, since the LNVA has jurisdiction over both the Neches-Trinity Coastal Basin and the lower portion of the Neches.

3. Floodplains and Wetlands

The developed portions of Nederland, Port Neches, and Groves are above flooding or are protected by a flood levee encompassing the urbanized area from Nederland through Port Arthur. All rainfall and effluent flows in this area are drained through storm sewers and/or to ditches to Jefferson County DD7 pump stations located at various points along the levee.

All existing wastewater plants for the three cities, including the Groves South Plant in Port Arthur, are protected against the 100 year flood by the above mentioned levee. The regional plant sites *(intercity and Groves east)* and the joint Nederland/Star Enterprise facility would be similarly protected.

Floodplain areas within the planning area include a portion of the Nederland ETJ on the west side of U. S. 69 (along Rhodair Gully). Floodplains affecting potential project elements include portions of the Nederland sites on Rhodair and Johns Gullys, as well as potential outfalls into the Neches River.

The planning area in itself does not contain significant amounts of wetlands, except for certain undevelopable areas *(very narrow strips along streams and various ponds representing industrial waste sites)*. However, the adjacent area along the Neches River is covered with vast marshes which extend from Sabine Lake to a point north of Beaumont. The marsh area is several miles wide in most places and extends well into Orange County across the river. The marsh area narrows to almost nothing on the west side of the river at Port Neches, however.[2] Also, most or all of the joint Nederland/Star Enterprise facility appears to fall in a marshy area.

The marshes are predominantly salt and brackish water up to a point just upstream from the planning area. From that point north, fresh water marshes take over. From the downstream edge of Beaumont north, the marshes are forested.[2, 9]

Some portions of the marshland have reportedly been covered with spoil material and thus removed from wetland status.

Narrow strips of wetlands occur along Rhodair and Johns Gullys, which flow through vacant areas for most of their length.

4. Climatic Elements

a. General. The climate of the study area is best described as being semitropical, with a mixture of tropical and temperate zone conditions. Sea breezes prevent extremely high temperatures in the summer, except on rare occasions. The area lies far enough south so that cold air masses of winter are moderate in severity, but still provide the stimulating effects of seasonal change.[3] The Gulf of Mexico dominates the climate of the climate of the region and accounts for the high humidity and high average rainfall. The mean annual relative humidity is approximately 83 per cent, while the average annual temperature at Port Arthur is about 69° F.[3]

The average rainfall for the study area, distributed evenly throughout the year, is 60.0 inches as determined from the National Weather Service records (1962-1987). The prevailing wind direction is south-southeasterly, averaging 11 mph. Except during infrequent tropical disturbances and severe thunderstorms, the wind seldom exceeds 45 mph, and exceeds 30 mph only about 40 days in any one year. The area enjoys approximately 308 clear or partly cloudy days each year.[3]

Winter temperatures are exceptionally mild. In January, the coldest month, the mean temperature is 53.3° F, with the minimum dropping to 32° F or below only four or five times during the month. Daily maximum temperatures average 64.3° F in the winter. The approximate dates of the first and last killing frosts are December 2 and March 2. Fog, most frequent in midwinter and rare in the summer, usually dissipates before noon, but occasionally under stagnant conditions lasts into the afternoon. The prevailing winds during the period from September through January are northerly.

Summers are warm and humid, with a growing season averaging 250 days. The month of July has a mean temperature of 84°F. Daytime maximum temperatures are moderated by the prevailing off-shore winds; these prevailing winds are southerly during the period from February to August.

Rainfall is abundant during the summer months; the excessive amounts of rain will occur over short periods of time. Thunderstorms are most frequent during July and August. The most persistent rains are generally associated with warm fronts and stationary fronts during the colder season and with dissipating cyclones during the summer and early fall.[10]

b. Air Quality. The Jefferson-Orange County area has been classified by the EPA as a nonattainment area because it cannot meet the EPA standards for ozone concentration in the atmosphere (0.12 ppm, 1 hr.). Consequently, the area faces possible sanctions from the EPA if it cannot attain compliance by 1999. An immediate consequence of local air quality problems may be a requirement for vehicle emissions testing beginning in 1995.[11]

5. Biological Elements

a. Plant Communities

(1) General. The study area falls within the Gulf Prairies and Marshes.[1] The land appears to have been mainly open land before local residents planted trees around their homes. In the case of Groves, large areas were planted in pecan orchards and later developed into residential lots.

The relatively high areas which are undeveloped are generally open. The area between the planning area and the Neches River is open marshland, much of which is covered with salt tolerant vegetation.[3] The nearest large forested areas are located several miles away in Orange County and in Jefferson County due south of Beaumont.

(2) Sabine-Neches Estuary. The Sabine-Neches Estuary extends from the Gulf of Mexico to the salt water barrier on the Neches River and adjoins the planning area. This area is recognized as a sensitive and unique ecosystem. The principal ecological areas are downstream from Beaumont, especially near the Highway 87 twin bridges over the Neches River.

Plant life along the estuary includes marsh grasses, tallow and willow trees, sedge, bulrush, and marshay millet.[12]

b. Animal Communities

(1) General. Animal life in open areas of Jefferson County includes ducks, quail, doves, geese, prairie chickens, raccoons, mink, squirrels, nutria, muskrats, and deer.[3] Aquatic animal life in inland areas includes turtles, moccasins, frogs, and alligators.

(2) Sabine-Neches Estuary. Aquatic life in the estuary includes gar, mullet, crabs, blue catfish, saltwater catfish, shrimp, croakers, common water snakes, and Rangia cuneata (a brackish clam). Land animals include nutria, muskrats, raccoons, opossums, rats, mice, beavers, skunks, and moccasins. The estuary contains over 200 species of birds, over half of which are aquatic species. Birdlife includes cranes, rails, snipes, herons, egrets, ducks, coots, gulls, terns, and waders.[12]

c. Habitats of Endangered Species. During a period from August 26-31, 1994, the Engineer sent letters (with fact sheet and maps) to various agencies, including state and federal wildlife agencies. Results to date (regarding wildlife) are as follows:

(1) U. S. Fish and Wildlife Service: Letter of September 1, 1994 indicated no federally listed or proposed threatened or endangered species in vicinity of study area.

(2) Texas Parks and Wildlife Department (Resource Protection Division): Letter of September 7, 1994 transmitted lists of endangered or threatened species possibly occurring in Jefferson County, as well as some special habitats listed by quadrangle map section. The letter indicated that a closer review could be performed after project alternatives are narrowed.

One species which has been listed in previous environmental reports for the area is the bald eagle. This bird tends to winter along major rivers and reservoirs,[13] which in the case of the study area would include only the Neches River.

6. Cultural Resources. Agencies contacted in August 1994 regarding cultural or historic resources were the Texas Historical Commission, the Texas Antiquities Committee, and the Texas Water Development Board (Engineering Division, Staff Archeologist). None of these agencies has responded. However, the agencies can be contacted again once the scope of the project has been better defined.

The TWDB, in a previous reconnaissance report for a Beaumont project, indicated that cultural remains (from Indian villages) could be expected mainly along major watercourses.[14] In the case of the study area, the Neches River would be the most likely location for such resources, possibly followed by Rhodair and Johns Gullys. It should be noted, however, that many cultural remains along the river may have been disturbed over the years in the course of repeated channel dredging and other activities.

The Spindletop Oil Field, several miles to the northwest of the study area, is included in a National Historic Landmark along with the Lucas Gusher. This oil well ushered in the petroleum age at the beginning of the century.

Points of interest include several museums in Nederland and Port Neches. Recreational facilities include a golf course and a swimming pool in Groves, as well as parks in all three cities. Another golf course is located in Port Arthur near Nederland. Many hunting, fishing, and boating opportunities are within easy driving distance.

7. Economic Conditions. Nederland, Port Neches, and Groves collectively make up an area known as Midcounty. The Midcounty area is part of the Golden Triangle which encompasses Beaumont and Port Arthur in Jefferson County and Orange in Orange County. The side extending from Beaumont to Port Arthur, including Midcounty, is a highly industrialized area extending the length of eastern Jefferson County in a broad strip parallel to the Neches River. Dominant industries in the area include petroleum refining and chemical and plastics industries, with two large paper mills a short distance north of the Triangle. Shipyards and a steel mill are also located in the Triangle.

In recent years, a portion of Jefferson County south of Beaumont has become the home of various state, federal, and county correctional facilities. Upon completion of all currently proposed units, the area will house approximately 12,000 inmates. This area is located only a few miles outside the planning area.

Agriculture in the Midcounty area is almost nonexistent. Agriculture in other portions of Jefferson County consists mainly of rice and soybean production.

For Jefferson County, the per capita income for 1989 was $16,375. Average weekly wage rate was $446.53 in 1990, with retail sales over $1.8 billion and tax value over $10 billion.[1]

The petroleum industry was born in Jefferson County, a few miles outside the planning area, at the beginning of the century. Over the years the area became highly dependent on the oil industry and various related industries, including refining, chemical and plastics manufacturing, and fabrication of oil field equipment. The local economic growth reached its peak in the early 1980's during a period of high demand for oil and refined products.

However, in response to high oil prices, engines were made more fuel efficient, reducing the worldwide demand for fuel. Local refineries cancelled expansion plans and laid off thousands of workers. Local shipyards declined, and oil prices fell in 1986 upon the collapse of the OPEC price controls.

Local employment has gradually improved since then, despite several additional plant closings and cutbacks. Factors contributing to improved conditions include diversification efforts, the growth of service industries, tax abatements, plant construction for environmental purposes, and the selection of Jefferson County for state and federal prison facilities.

The Jefferson County Airport, adjacent to Nederland, serves the entire Southeast Texas area with several commercial airlines. Highways through the Midcounty area include a federal highway, three state highways, and several farm roads. Several branches of the KCS Railroad pass through the area. Local ship channels include the Neches River and the connecting channels through Sabine Lake and Sabine Pass to the Gulf of Mexico. Ports include industrial ports in Port Neches, as well as the nearby Ports of Port Arthur and Beaumont. The Intracoastal Waterway, passing within two miles of the Midcounty area, provides for barge traffic.

Education through high school is provided by the Nederland Independent School District, the Port Neches-Groves ISD, and (for a small area) by the Port Arthur ISD, as well as by nearby parochial schools. Higher education is available at Lamar University, with campuses in Beaumont, Port Arthur, and Orange.

General hospitals include one each in Nederland and Groves, as well as two in nearby Port Arthur and three in Beaumont.

The 1994 city populations are estimated at 16,549 for Nederland, 13,479 for Port Neches, and 16,967 for Groves. Projected populations in fifteen years (2009) are 24,240 for Nederland, 14,517 for Port Neches, and 17,538 for Groves.[15] For the end of the study period (2024), the projected city populations are 24,816 for Nederland, 15,040 for Port Neches, and 17,794 for Groves.

Sewered populations are close to city populations for Port Neches and Groves. However, the Nederland figures vary considerably since the City proposes future annexation of an area including over 5000 residents of Jefferson County WCID No. 10, while leaving the District sewer system in service. Population served by the Nederland sewer system is estimated at 17,650 for 1994, 18,674 for 2009, and 19,127 for 2024. It should be noted that Port Neches and Groves have little or no opportunity for future annexation because of adjacent cities.

8. Land Use. All three cities have zoning. Actual land use at this time can be generally described as follows:

   a. Nederland. Roughly 90% of the existing City is residential, with the remainder commercial, public, and a relatively small amount of vacant land. The outside service area includes the Jefferson County Airport. Of the remaining existing potential service area *(which excludes Jefferson County WCID No. 10, with its own system)*, roughly 20% is residential, with commercial land very small in comparison. The remainder of the area is vacant.

   b. Port Neches. The portion of the City designated as part of the planning area is approximately 50% residential, 10% commercial and public, 10% vacant developable, and the remainder undevelopable industrial waste sites. A large portion of the City is excluded from the planning area as marsh land. There are substantial amounts of industrial sites included in the planning area, excluded from the City but surrounded by it.

      Most residential and commercial development in the City is concentrated in the northwestern portion adjacent to Nederland. The area next to Groves is mainly vacant with unincorporated industrial sites interspersed.

   c. Groves. Approximately 90% of the City is residential, with the remainder commercial, public (including a golf course), and a minor amount of vacant land. The Groves service area includes a residential area within Port Arthur, which is declining in population because of an ongoing buyout by an adjacent industry.

   d. Future Growth. Groves has only limited space for future growth, being surrounded by other cities and containing little vacant land. Port Neches is almost surrounded, and most of its vacant areas are undevelopable marsh land or industrial waste sites. Nederland has somewhat more capacity, having annexed a corridor around several square miles of its ETJ. Much of that area is occupied by Jefferson County WCID No. 10 with its own sewer system serving a population over 5000, and by industries. However, the ETJ also contains considerable open and residential areas outside the District.

9. Other Programs

   a. Economic Development. A number of privately and publicly sponsored programs were developed in Southeast Texas in the late 1980's for the purpose of attracting new industries to the area. The immediate goal was to replace the thousands of jobs which were lost during that decade as the result of plant closings and production cutbacks. Some of the programs for attracting new industry included a low interest loan program in which local citizens accepted a low rate of interest on savings; City revolving loan funds (in Beaumont and Port Arthur) for small businesses; several job training programs; and agencies providing various information to new or expanding businesses, including export assistance.

Along with the efforts to locate potential industries, the local governments in the area offered tax abatements for new industrial facilities or for expansion of existing facilities. Several governments, including Jefferson County, developed specific policies for the duration and extent of abatements according to the construction cost and/or the number of temporary or permanent jobs created.

The county and several local governments submitted a proposal several years ago for a state prison location on a site between Beaumont and Port Arthur. The site was selected by the state government, and one prison unit is already in service. Another unit is nearing completion, with several other units scheduled within the next few years. Similar proposals were submitted to the federal government, and a 4000 bed federal prison is under construction west of the state facilities.

Other recent programs for economic development include establishment of foreign trade zones, enterprise zones, and economic redevelopment zones.

The immediate goal of these various programs was to provide employment for local residents who lost their previous jobs or who were entering the job market. Beyond that goal, additional net job creation could induce the Southeast Texas area to grow beyond the peak population which was reached in the 1980's. Such future growth would affect the sizing of the necessary wastewater system improvements for the three cities in the planning area.

The economical development programs have been relatively successful in the last several years, although some plant closings have continued to occur. As a result, the TWDB has increased its population projections for Jefferson County and for the three cities.[15]

The size of the communities, however, does not control the basic need for the improvements. The work is necessary for several reasons including new stringent stream standards for three of the four existing plants; excessive infiltration/inflow in the sewage collection systems, resulting in excessive flows (for three of the plants) and overflows; and deteriorating condition of several treatment plant units.

b. Drainage. Most of Jefferson County suffers from poor natural drainage because of the flat topography and low elevation. Drainage for the three cities in the study, as well as the Port Arthur area, is enhanced by the efforts of Jefferson County Drainage District No. 7. The District operates a network of improved drainage ditches, many of which are concrete lined.
Surrounding the urbanized area on three sides is a storm levee constructed by the Corps of Engineers and designed to protect against the effects of tidal surges during hurricanes. The drainage system takes the local storm water to various points just inside the levee and then pumps it to the opposite side of the levee with storm water pump stations. Some pumps must operate on a daily basis because of large volumes of domestic and industrial treatment plant effluent.

Drainage from most of the cities in the planning area is tributary to the Main Outfall Ditch and is pumped by the Alligator pump station. The intermittent operation of the existing pumps has resulted in cyclic high levels in the lateral ditch which receives effluent from the Port Neches and Groves North Plants. The high stream levels create hydraulic problems in the Port Neches plant, thus reducing effective flow capacity. The District has been seeking funding to upgrade its pumping facilities to eliminate this problem.[7, 16]

c. Miscellaneous Programs. Although most highway improvements within the planning area appear to be complete, a master plan for future westward highway loop extensions has been prepared. The future highways would link the Midcounty area with Interstate 10 to Houston. Imminent widening of State Highway 73 west of Port Arthur will also improve access to the area.

Other programs which contribute to the quality of life in the Midcounty cities include low rent housing programs; mosquito control by a county agency; and the higher education provided at Beaumont and Port Arthur by Lamar University.

TABLE E-1

SOIL ASSOCIATIONS FOR POTENTIAL PROJECT ELEMENTS

CITY OR OTHER OWNER	DESCRIPTION	ASSOCIATION(S)	SOIL TYPES
Nederland	Relocated plant, Site A (Rhodair Gully)	Morey-Crowley-Hockley (apparently entire site)	Mainly Beaumont clay, with Morey silt loam near NW° corner
	Relocated plant, Site B (Johns Gully)	Beaumont-Morey; possibly extends into Morey-Crowley-Hockley	Morey silt loam
	Wetland as joint venture with Star Enterprise	Harris-Made land	Beaumont clay and Harris clay
	Transfer lines to relocated plant sites	Mainly Beaumont-Morey; line to Site B passes through Morey-Crowley-Hockley; line to joint wetland passes into Harris-Made land	Beaumont clay and Morey silt loam
	Outfall to river	Beaumont-Morey; then Morey-Crowley-Hockley (?); then narrow strip of Salt water marsh-Tidal marsh	In Nederland: Morey silt loam, edges of Beaumont clay; through Unocal site: Morey silt loam, possibly Beaumont clay, Acadia silt loam, Harris clay, and/or Made land
	Collection system improvements	Beaumont-Morey	Beaumont clay and Morey silt loam
Port Neches/ Groves	Outfall from Port Neches and/or Groves North to river	Beaumont-Morey most of way; ends with a strip of Salt water marsh-Tidal marsh; Routes A & C *(but not B)* also cross narrow strip of Morey-Crowley-Hockley between other two associations	Rt. A: Morey silt loam w/ small areas Crowley silt loam, narrow strip Acadia silt loam, Made land in river marshes; Rt. B similar to Rt. A; Rt. C: Bmt. clay most of way, Crowley silt loam last part of Spur 136, then narrow strip Acadia silt loam

Groves	Regional plant *(east side)*	Beaumont-Morey	Beaumont clay
	Transfer line between North and South Plants	Beaumont-Morey	Morey silt loam, Beaumont clay, Harris clay
	Transfer lines to Groves regional *(east side)*	Beaumont-Morey	Morey silt loam, Beaumont clay
	Outfall from South Plant	Harris-Made land	Harris clay, Made land
	Outfall from Groves regional *(east side)*	Mostly Beaumont-Morey; crosses narrow strip of Harris-Made land and/or Salt water marsh-Tidal marsh just before discharge point	Beaumont clay, Morey silt loam, Harris clay (possibly), Made land
Regional	Treatment plant	Appears to overlap Beaumont-Morey and Morey-Crowley-Hockley	Morey silt loam, Beaumont clay
	Transfer lines from all existing plants	Beaumont-Morey	Nederland: Morey silt loam, some Bmt. clay; PN/GN°: Morey silt loam; Groves S°: Bmt. clay, Morey silt loam, Crowley silt loam; joint: Morey silt loam
	Outfall to Neches River	Narrow strip of Morey-Crowley-Hockley; then Salt water marsh-Tidal marsh	Acadia silt loam, Made land, possibly Salt water marsh

TABLE E-2

SOIL TYPES

The Beaumont-Morey soils are marked by a flat, uniform topography with few natural drains. Water stands for long periods after heavy rains. The Beaumont series consists of gray to dark-gray poorly drained, acid soils with a clay texture throughout their profile. The Morey series consists of deep, gray to dark-gray, poorly drained, acid soils with a tight silty clay loam subsoil.

The Crowley series, which occurs within the Beaumont-Morey association in the study area, consists of deep, light-gray to grayish-brown, acid soils with a thick horizon of silt loam. These soils are imperfectly or somewhat poorly drained. Runoff is slow, and internal drainage is very slow. Made land (spoil areas for dredged materials) generally contains saline soils. Such soils will support vegetation a few years after their construction, but are not suitable for cultivation even though they may be high and well drained.

Salt water marsh contains a 16 to 36 inch layer of organic peat and muck over a clay or silty clay, with a water table within six inches of the surface. This soil is not covered every day by tides and will support the weight of grazing cattle, unlike the Tidal marsh in coastal areas.

The Harris-Made land association consists of flats with salt-tolerant vegetation. The flats are covered mainly with Harris soils, which are dark, wet, poorly drained, saline clay soils. The surface layer (usually 20 inches thick) is neutral to alkaline, sticky when wet, and very hard when dry. In the study area, this association is made up mainly of Made land, which also occurs in the Salt water marsh-Tidal marsh association.

The Acadia soils, which occur in narrow strips along the west edge of the Salt water marsh-Tidal marsh association, are deep, dark-colored, acid, poorly drained soils. The surface layer consists of silt loam, with tight silty clay loam and clay underneath.[3]

TABLE E-3

RECEIVING STREAM DESCRIPTIONS
JEFFERSON COUNTY DRAINAGE DISTRICT 7

Nederland: Main Ditch C, then to Main Ditch B, then to Main Outfall Canal, then through Alligator Pump Station, then to Taylor Bayou (east distributary branch), then to Intracoastal Waterway (Segment 0702 of Neches-Trinity Coastal Basin)

Port Neches: Main Ditch A-3, then to Main Ditch A, then to Main Outfall Canal, then through Alligator Pump Station, then to Taylor Bayou (east distributary branch), then to Intracoastal Waterway (Segment 0702 of Neches-Trinity Coastal Basin)

Groves North: [Same as Port Neches; both plants are adjacent to each other]

NOTE: All drainage ditches as well as the pump station are owned and operated by Jefferson County DD7.

REFERENCES

1. Texas Almanac, 1992-1993.

2. Quadrangle maps, U. S. Geological Survey: Beaumont East, Port Acres, Port Arthur North, and Terry.

3. USDA Soil Conservation Service, Soil Survey, Jefferson County, Texas, February 1965.

4. USDA Soil Conservation Service, Beaumont, Texas.

5. Environmental Geological Atlas of the Texas Coastal Zone: Beaumont-Port Arthur Area, Bureau of Economic Geology, University of Texas, 1973.

6. Groundwater Resources of Chambers and Jefferson Counties, Texas, J. B. Wesselman, Report 133, Texas Water Development Board, August, 1971.

7. City of Port Neches, correspondence to TWC, Beaumont, December 18, 1992.

8. Work Package No. 2, Water Quality Data Base, part of South East Texas 208 Areawide Waste Treatment Management Plan, Lower Neches Valley Authority for South East Texas Regional Planning Commission, October 1976.

9. Environmental Impact Assessment, South East Texas 208 Areawide Waste Treatment Management Plan, Water Resources Engineers, a Camp, Dresser & McKee firm, Austin, Texas, April 1978.

10. Environmental Assessment for Sewage Treatment Plant Improvements and Interceptor Construction, City of Beaumont, Texas, Schaumburg & Polk, Inc., June 1978.

11. Texas Natural Resource Conservation Commission, Region 10, Beaumont, conversation with Marion Everhart of Air Program, December 8, 1994.

12. Texas Highway Department, District 20, Beaumont, Environmental Assessment for Neches River Bridge on State Highway 87, 1970's.

13. U. S. Fish and Wildlife Service, Houston, Texas, Consultation No. 2-13-88-I-168, regarding Beaumont SRF, September 9, 1988.

14. Chris Jurgens, Texas Water Development Board Archeologist, "Archeological Reconnaissance at the City of Beaumont, Jefferson County, Texas," State Revolving Fund Project #2109, January 17, 1989.

15. Texas Water Development Board, population projections for Jefferson County, Nederland, Port Neches, and Groves, revised draft (transmitted to Schaumburg & Polk, October 6, 1994); Nederland projections for 2009 and 2024 were revised by Engineer to reflect anticipated future annexation.

16. Jefferson County Drainage District No. 7, Port Arthur, Texas.

PERTINENT ENVIRONMENTAL CORRESPONDENCE

Schaumburg & Polk, Inc.
CONSULTING ENGINEERS

8865 College St., Suite 100
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Phone (409) 866-0341
FAX (409) 866-0337

August 26, 1994

Mr. Frederick Werner
U. S. Department of the Interior
Fish and Wildlife Service
Division of Ecological Services
17629 El Camino Real, Suite 211
Houston, Texas 77058

Re: Regional Wastewater Study
Cities of Nederland, Port
Neches, and Groves
Jefferson County, Texas

Dear Mr. Werner:

We are conducting a regional wastewater study for the above referenced cities to develop alternatives for solving various pressing wastewater management problems. The cities are faced to varying degrees with extremely stringent effluent standards; infiltration/inflow; need for additional plant capacity; and other collection system and treatment plant improvements.

As the attached fact sheet and maps show, the study will consider various alternatives including a new regional plant to serve all three cities. The regional alternative, as well as several alternatives for separate facilities, includes an outfall to the Neches River.

The study is expected to develop a program of wastewater improvements phased over a number of years. The initial phases of the program will be very urgent because of impending effluent standards. It may be necessary to begin design on some of the facilities as early as late 1994, even before finalization of the study.

The study must present and discuss environmental considerations for the various alternatives to be presented. The study is due in draft form by November 30, with the final report by January 31, 1995.

Letter
SPI No. 4004.0
DF:627\WERNER.LET\Regional Wastewater Study--
Environmental Letters
082694

The areas which appear to be of most concern to your agency are the potential project elements involving new treatment plant and wetland locations, as well as any outfalls to the Neches River. The most significant work in existing wetlands would be those portions of any outfalls to the river passing through the floodplain of the river.

Please provide any comments which may be appropriate at this stage, including any information on endangered species in the area of the potential project elements.

The Texas Parks and Wildlife Department is also being contacted concurrently regarding the project.

In light of the information in the fact sheet and the project schedule as discussed above, we request your initial comments as soon as possible.

Please contact me or Gary Graham, P. E. of this office if you have any questions.

Sincerely,
Schaumburg & Polk, Inc.

Jeffrey G. Beaver, P. E.
Vice President

JGB/DE

encl.

cc (w/encl.): U. S. Fish and Wildlife, Beaumont
City of Nederland
City of Port Neches
City of Groves

Letter
SPI No. 4004.0
DF:627\WERNER.LET\Regional Wastewater Study–
Environmental Letters
082694

Schaumburg & Polk, Inc.
CONSULTING ENGINEERS

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Phone (409) 866-0341
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August 26, 1994

Mr. Bob Spain, Chief
Texas Parks and Wildlife Department
Resource Protection Division
Habitat Assessment Branch
4200 Smith School Road
Austin, Texas 78744

Re: Regional Wastewater Study
Cities of Nederland, Port
Neches, and Groves
Jefferson County, Texas

Dear Mr. Spain:

We are conducting a regional wastewater study for the above referenced cities to develop alternatives for solving various pressing wastewater management problems. The cities are faced to varying degrees with extremely stringent effluent standards; infiltration/inflow; need for additional plant capacity; and other collection system and treatment plant improvements.

As the attached fact sheet and maps show, the study will consider various alternatives including a new regional plant to serve all three cities. The regional alternative, as well as several alternatives for separate facilities, includes an outfall to the Neches River.

The study is expected to develop a program of wastewater improvements phased over a number of years. The initial phases of the program will be very urgent because of impending effluent standards. It may be necessary to begin design on some of the facilities as early as late 1994, even before finalization of the study.

The study must present and discuss environmental considerations for the various alternatives to be presented. The study is due in draft form by November 30, with the final report by January 31, 1995.

The areas which appear to be of most concern to your agency are the potential project elements involving new treatment plant and wetland locations, as well as any outfalls from treatment plants (or wetland units) to the Neches River. The most significant work in existing wetlands would be those portions of any outfalls to the river passing through the floodplain of the river.

Also, the actual outfall structure may be of concern to your agency. Potential outfall locations would be located in the segment of the river which serves as a deepened ship channel. Our company has been in contact with your agency earlier this year regarding a similar, but larger, outfall from a new plant for the City of Beaumont to a point upstream from the potential outfalls addressed in this letter.

Please let us know:

a. Any comments which may be appropriate at this stage, regarding endangered species in the area of the potential project elements.

b. Any approval from your agency which might be required for an outfall within this area of the river.

b. Any special requirements by your agency for such outfalls.

The Corps of Engineers and the Texas General Land are being contacted concurrently regarding the possible need for their authorizations for this proposed river outfall. The U. S. Fish and Wildlife Service is also being contacted in a similar regard and also with respect to any possible impacts on endangered species.

In light of the information in the fact sheet and the project schedule as discussed above, we request your initial comments as soon as possible.

Please contact me or Gary Graham, P. E. of this office if you have any questions.

Sincerely,
Schaumburg & Polk, Inc.

Jeffrey G. Beaver

Jeffrey G. Beaver, P. E.
Vice President

JGB/DE

encl.

cc (w/encl.): Charles Stutzenbaker (TP&WD, Pt. Arthur)
City of Nederland
City of Port Neches
City of Groves

Schaumburg & Polk, Inc.
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August 26, 1994

Ms. Shannon Breslin
Texas Parks and Wildlife Department
Resource Protection Division
Texas Natural Heritage Program
4200 Smith School Road
Austin, Texas 78744

Re: Regional Wastewater Study
Cities of Nederland, Port
Neches, and Groves
Jefferson County, Texas

Dear Ms. Breslin:

We are conducting a regional wastewater study for the above referenced cities to develop alternatives for solving various pressing wastewater management problems. The cities are faced to varying degrees with extremely stringent effluent standards; infiltration/inflow; need for additional plant capacity; and other collection system and treatment plant improvements.

As the attached fact sheet and maps show, the study will consider various alternatives including a new regional plant to serve all three cities. The regional alternative, as well as several alternatives for separate facilities, includes an outfall to the Neches River.

The study is expected to develop a program of wastewater improvements phased over a number of years. The initial phases of the program will be very urgent because of impending effluent standards. It may be necessary to begin design on some of the facilities as early as late 1994, even before finalization of the study.

The study must present and discuss environmental considerations for the various alternatives to be presented. The study is due in draft form by November 30, with the final report by January 31, 1995.

Letter
SPI No. 4004.0
DF:627\BRESLIN.LET\Regional Wastewater Study--
Environmental Letters
082694

The areas which appear to be of most concern to your agency are the potential project elements involving new treatment plant and wetland locations, as well as any outfalls from treatment plants (or wetland units) to the Neches River. The most significant work in existing wetlands would be those portions of any outfalls to the river passing through the floodplain of the river.

Please provide any comments which may be appropriate at this stage, including any information on endangered species in the area of the potential project elements.

The U. S. Fish and Wildlife Service and also the Habitat Assessment Branch of your agency are also being contacted with regard to any possible impacts on endangered species.

In light of the information in the fact sheet and the project schedule as discussed above, we request your initial comments as soon as possible.

Please contact me or Gary Graham, P. E. of this office if you have any questions.

Sincerely,
Schaumburg & Polk, Inc.

Jeffrey G. Beaver

Jeffrey G. Beaver, P. E.
Vice President

JGB/DE

encl.

cc (encl.): City of Nederland
City of Port Neches
City of Groves

Letter
SPI No. 4004.0
DF:627\BRESLIN.LET\Regional Wastewater Study–
Environmental Letters
082694

FACT SHEET

Schaumburg & Polk, Inc. has been retained to perform a regional wastewater study for three cities in Jefferson County--Nederland, Port Neches, and Groves. The study is funded in part by a planning grant from the Texas Water Development Board. The study will investigate various means of addressing the needs of these cities for wastewater facility improvements, including a possible regional wastewater treatment plant to serve all three cities. Also covered in the study are possible improvements to the existing collection systems.

The study was prompted by the recent stream studies conducted in the vicinity by the Texas Water Commission (now the Texas Natural Resource Conservation Commission). Three of the four wastewater treatment plants serving the three cities presently discharge into a drainage ditch system operated by Jefferson County Drainage District No. 7. The ditch system leads to the outfall of Taylor Bayou, a sensitive coastal stream. During recent permit renewals, Nederland and Port Neches were told initially that they would receive permits calling for no discharge beginning three years after permit issuance. As a result of more detailed studies, this requirement was amended, but those cities would still be required to meet advanced effluent quality standards after three years. The North Plant in Groves is expected to face similar requirements upon permit renewal in 1995.

The South Plant in Groves, which discharges into the Intracoastal Waterway, is expected to retain its existing secondary effluent standards for the foreseeable future. However, the plant contains some deficiencies which need to be resolved.

Although the project alternatives have not yet been finalized, they can be tentatively summarized as follows:

A. TREATMENT FACILITIES

1. Possible regional plant to serve all three cities, discharging into Neches River. *(It is expected that secondary standards as existing for the various plants will be required for discharges of any or all of the effluent for the three cities into the Neches.)*

2. Individual Improvements *(If the regional plant is not selected)*:

   a. Nederland *(Various alternatives)*:

      (1) Expand existing plant and upgrade to tertiary standards, continuing to discharge into drainage ditch.

      (2) Construct a wetland treatment facility to provide further effluent treatment before discharging into Rhodair Gully or Neches River.

      (3) Abandon existing WWTF and construct a lagoon/pond/wetland facility for discharge into Rhodair Gully or Neches River.

(4) Abandon existing WWTF and construct a new mechanical facility for discharge into Rhodair Gully or Neches River.

b. Port Neches *(Various alternatives)*:

(1) Upgrade existing plant to advanced standards, continuing to discharge into drainage ditch.

(2) Divert effluent to Neches River.

c. Groves:

(1) North Plant *(Various alternatives)*:

(a) Upgrade existing plant to advanced standards, continuing to discharge into drainage ditch.

(b) Upgrade existing plant and divert effluent to Neches River.

(2) South Plant: Upgrade existing plant.

B. COLLECTION SYSTEM, TRANSPORTATION, AND OUTFALL ELEMENTS

1. In the event of a regional wastewater plant, lift stations and force mains would be constructed to transport raw wastewater from each city to the regional plant. An outfall would also be constructed from that plant to the Neches River.

2. For upgrading of the Port Neches and Groves North plants and diversion to the Neches River, a common outfall from those two plants *(which are adjacent to each other)* to the river would be constructed.

3. In the case of a relocated plant for Nederland, appropriate transportation facilities would be constructed to transport all raw influent to the new plant. An outfall from the new plant to Rhodair Gully or to the Neches River would also be constructed.

4. In the case of a constructed wetland for Nederland (following existing plant), appropriate transportation facilities for partially treated effluent would be constructed from the existing or relocated plant to the wetland. An outfall from the wetland to Rhodair Gully or to the Neches River would also be constructed.

5. For all three cities, selected segments of the collection systems will be rehabilitated, upgraded, or relieved by new facilities to reduce infiltration/inflow problems and/or to eliminate overloading.

Attached are two maps showing the three cities, the existing wastewater treatment plants, the existing discharge points, and the existing and potential receiving streams.

ADDRESS LIST

__August 26, 1994__:

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U. S. Army Corps of Engineers
Galveston District
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Copies to:

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Chief, Enforcement Section
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Area Engineer
P. O. Box 157
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Division of Ecological Services
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Ms. Shannon Breslin
Texas Parks and Wildlife Department
Resource Protection Division
Texas Natural Heritage Program
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Austin, Texas 78744

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LaPorte, Texas 77571-5048

August 29, 1994:

Federal Emergency Management Agency
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Mr. Richard Grabowski
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Denver, Colorado 80225

Mr. Norman Thomas
EPA 6E F
U. S. Environmental Protection Agency
1445 Ross Avenue, Suite 1200
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U. S. Environmental Protection Agency
Municipal Permitting Section (MW-P)
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Texas Antiquities Committee
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Austin, Texas 78711-2276

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Mr. Randy Wilburn, P. E.
Texas Natural Resource Conservation Commission
Wastewater Permits Section
Watershed Management Division
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Copy to:

Mr. Keith Anderson
Texas Natural Resource Conservation Commission
Region 10
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Mr. Sasha Earl, P. E.
Texas Natural Resource Conservation Commission
Plans and Specifications Review Section
Watershed Management Division
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Copy to:

Mr. Keith Anderson
(Address above)

Mr. Don Kelley
South East Texas Regional Planning Commission
P. O. Drawer 1387
Nederland, Texas 77627

Mr. Tom Hebert
Lower Neches Valley Authority
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Beaumont, Texas 77704

Mr. Victor Bateman
Jefferson County Environmental Control
2748 Viterbo Road, Box 4
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Mr. Robert Stroder
County Engineer, Jefferson County
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Beaumont, Texas 77701

Mr. Bill Macon
Jefferson County Drainage District 7
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August 31, 1994

Mr. Mike Kieslich
Chief, Planning Division
U. S. Army Corps of Engineers
Galveston District
P. O. Box 1229
Galveston, Texas 77553-1229

Re: Regional Wastewater Study
Cities of Nederland, Port
Neches, and Groves
Jefferson County, Texas

Dear Mr. Kieslich:

We are conducting a regional wastewater study for the above referenced cities to develop alternatives for solving various pressing wastewater management problems. The cities are faced to varying degrees with extremely stringent effluent standards; infiltration/inflow; need for additional plant capacity; and other collection system and treatment plant improvements.

As the attached fact sheet and maps show, the study will consider various alternatives including a new regional plant to serve all three cities. The regional alternative, as well as several alternatives for separate facilities, includes an outfall to the Neches River.

The study is expected to develop a program of wastewater improvements phased over a number of years. The initial phases of the program will be very urgent because of impending effluent standards. It may be necessary to begin design on some of the facilities as early as late 1994, even before finalization of the study.

The study must present and discuss environmental considerations for the various alternatives to be presented. The study is due in draft form by November 30, with the final report by January 31, 1995.

Letter
SPI No. 4004.0
DF:627\KIESLICH.LET\Regional Wastewater Study--
Environmental Letters
083194

An examination of the alternatives listed in the fact sheet suggests that your agency would be concerned primarily with the potential outfalls from one or more treatment plants to the Neches River. Potential outfall locations would be located in the segment of the river which serves as a deepened ship channel. Our company has been in contact with the Corps earlier this year regarding a similar, but larger, outfall from a new plant for the City of Beaumont to a point upstream from the potential outfalls addressed in this letter.

Potential outfall routes may cross wetland areas before reaching the river. Our previous communications with the Corps indicate that the linework outside the river can be performed under a nationwide permit. However, the actual outfall into the river may require an individual Corps permit.

Please let us know:

a. **What type of Corps approval would be required for an outfall within this area of the river, such as an individual Section 10 or 404 permit, or other form of notice?**

b. **Any Corps requirements such as minimum submergence and maximum distance that pipe can extend into the channel.**

The Texas Parks and Wildlife Department and the Texas General Land Office are being contacted concurrently regarding the possible need for their authorizations for any outfalls into the Neches River resulting from this study.

In light of the information in the fact sheet and the project schedule as discussed above, we request your initial comments by September 30 if possible.

Please contact me or Gary Graham, P. E. of this office if you have any questions.

Sincerely,
Schaumburg & Polk, Inc.

Jeffrey G. Beaver, P. E.
Vice President

JGB/DE

encl.

cc: City of Nederland
City of Port Neches
City of Groves

United States Department of the Interior

TAKE PRIDE IN AMERICA

FISH AND WILDLIFE SERVICE
Division of Ecological Services
17629 El Camino Real, Suite 211
Houston, Texas 77058

September 1, 1994

Jeffrey G. Beaver
Schaumburg & Polk, Inc.
8865 College Street, Suite 100
Beaumont, Texas 77707

Dear Mr. Beaver:

This responds to your August 26, 1994 letter requesting information on Federally listed threatened or endangered species which may be in your project area. The project involves a regional wastewater study as a means of solving various wastewater management problems faced by the Cities of Nederland, Port Neches, and Groves in Jefferson County, Texas.

A review of U.S. Fish and Wildlife Service (Service) files and your project maps indicates that no federally listed or proposed threatened or endangered species are known to occur in the vicinity of your study area.

The impacts of any proposed projects should also be evaluated pursuant to Executive Orders 11988, Floodplain Management, and 11990, Protection of Wetlands. These Executive Orders were issued to avoid, to the extent possible, the long and short term adverse impacts associated with the occupancy and modification of floodplains. The first Executive Order requires justifying any structures located in a floodplain; the second requires each agency to avoid undertaking or providing assistance for new construction located in wetlands unless there is no practicable alternative and the proposed action includes all practicable measures to minimize harm to wetlands.

Please note that the Service provides technical, and in some cases, financial assistance, for the construction of wetlands which utilize the treated effluent water discharged from wastewater treatment plants. While the Service cannot fund the construction of a wetland treatment facility, we would be interested in exploring the possibility of expanding and/or altering such a facility to meet both of our needs. The utilization of effluent water, which already meets discharge standards, for the creation of wetlands provides valuable wildlife habitat while further filtering the effluent water. General information on this program is enclosed.

If we can be of further assistance, please contact Edith Erfling at 713/286-8282.

Sincerely,

Frederick T. Werner
Chief, Regulatory Activities

enclosure

4004.0 - ENV. Assessment

DEPARTMENT OF THE ARMY
GALVESTON DISTRICT, CORPS OF ENGINEERS
P.O. BOX 1229
GALVESTON, TEXAS 77553-1229

RECEIVED SEP 09 1994

Sept. 7, 1994

REPLY TO
ATTENTION OF:

Enforcement Section

SUBJECT: D-6043; Jurisdictional Determination, Cities of Nederland, Port Neches, and Groves, Jefferson County, Texas

Jeffrey G. Beaver, P.E.
Vice President
Schaumburg & Polk, Inc.
8865 College Street, Suite 100
Beaumont, Texas 77707

Dear Mr. Beaver:

We acknowledge receipt of your August 26, 1994, letter, requesting a jurisdictional determination for a wastewater treatment outfall for the Cities of Nederland, Port Neches, and Groves, in Jefferson County, Texas. The above number has been assigned to your request; please reference this number in all future correspondence with our office pertaining to this request. Should you have any questions or require additional information, please contact me at the letterhead address or by telephone at (409) 766-3933.

Sincerely,

John Davidson
Project Manager, North Unit
Enforcement Section

RECEIVED SEP 12 1994

TEXAS PARKS AND WILDLIFE DEPARTMENT

4200 Smith School Road • Austin, Texas 78744 • 512-389-4800

September 7, 1994

Jeffrey G. Beaver, P.E.
Schaumberg & Polk, Inc.
8865 College Street, Suite 100
Beaumont, Texas 77707

Dear Mr. Beaver:

In response to your August 26, 1994 request for information on sensitive species and natural communities within or near the regional wastewater study for the Cities of Nederland, Port Neches, and Groves in Jefferson County, we offer the following comments. A search of the Texas Natural Heritage Program (TXNHP) Information System produced the following printouts. Please find enclosed a list of presently computerized records, an incomplete list of rare vertebrates, and a list of state endangered and threatened species that possibly occur in Jefferson County. This information is very general. For future reference, the TXNHP is able to do individual project reviews. This allows us to provide the most up to date and site specific information available. When project alternatives are narrowed, we would welcome the opportunity to review your project in greater detail.

The Heritage Program information included here is based on the best data currently available to the state regarding threatened, endangered, or otherwise sensitive species. However, these data do not provide a definite statement as to the presence or absence of special species or natural communities within your project area, nor can these data substitute for an on-site evaluation by qualified biologists. This information is intended to assist you in avoiding harm to species that occur on your site.

This letter does not constitute a review of fish and wildlife impacts that might result from the activity for which this information is provided. Should you need an impact review of this type from the Texas Parks and Wildlife Department, contact the Habitat Assessment Branch of the Resource Protection Division, attention Mr. Bob Spain, or contact him at 512/389-4725. All requests for reviews must be in writing.

REGIONAL WW STUDY
-.04.0 - ENVIRONMENTAL ASSESSMENT

Please contact the Texas Parks and Wildlife Department's Heritage Program before publishing printout data or otherwise disseminating any specific locality information. Thank you for contacting us. Please feel free to call me at 512/448-4311 if you have questions.

Sincerely,

Shannon Breslin, Assistant Data Manager
Texas Natural Heritage Program
Resource Protection Division

Enclosures

SLB:sb

DEPARTMENT OF THE ARMY
GALVESTON DISTRICT, CORPS OF ENGINEERS
P.O. BOX 1229
GALVESTON, TEXAS 77553-1229

REPLY TO
ATTENTION OF:

SEP 27 1994

North Evaluation Section

SUBJECT: D-6061; Construction of an Outfall Structure on the Neches River

Mr. Jeffery G. Beaver
Schaumburg & Polk, Inc.
Suite 100
8865 College Street
Beaumont, Texas 77707

RECEIVED SEP 29 1994

Dear Mr. Beaver:

Thank you for your letter concerning the construction of an outfall structure on the Neches River, near Nederland, Port Neches and Groves, Jefferson County, Texas.

In your letter, you asked, "what type of Corps approval would be required, such as Section 10 or 404 permit, or other form of notice?" The section of the Neches River that you are interested in would require a Section 10 permit and possibly a Section 404 permit for the construction of an outfall structure. The outfall may require an individual permit, which requires the submission of an application for a permit; or, it might qualify for Nationwide Permit 7, which requires you submit a notification to us. When you submit an application with detailed plans, we will be able to determine which type of permit this project will require. Enclosed is an application packet to assist you.

You also asked, "what minimum submergence and maximum distance the outfall pipe can extend into the channel?" There is no minimum submergence required for outfall pipes. However, the outfall structure and/or pipe can not be within 50 feet of the upper cut of the channel.

1 Nederland WW Study
4004.0 - Environmental Assessment

Should you have any questions, please contact the Project Manager, Mona G. Coleman at the above letterhead address or by telephone at 409/766-3936.

Sincerely,

Bruce H. Bennett

Bruce H. Bennett
Leader, North Evaluation Section

Enclosure

TEXAS
PARKS AND WILDLIFE DEPARTMENT
4200 Smith School Road • Austin, Texas 78744 • 512-389-4800

October 18, 1994

RECEIVED OCT 21 1994

Mr. Jeffrey G. Beaver, P.E.
Schaumberg & Polk, Inc.
8865 college Street, Suite 100
Beaumont, Texas 77707

Re: Information Request Concerning a Regional Wastewater Study for the Cities of Nederland, Port Neches, and Groves, Jefferson County, Texas

Dear Mr. Beaver:

Thank you for coordinating with this agency in your planning activities concerning this regional study. You have requested preliminary information regarding fish, wildlife, and plant resources for preparation of an analysis of alternatives to solve specific wastewater problems within these communities. We anticipate alternatives may include construction of facilities and activities which will potentially adversely impact natural resources. Activitities which will have probable adverse environmental impact to fisheries, wildlife species, or habitats include: removal of vegetation cover, landform alteration including buildling of levees, trenching, ditching, rebuilding on the floodplain, construction anywhere in a previously undeveloped area, use of pesticides in the project area, allowing undue noise and associated disturbance in the project area, destruction of inert microhabitats (snags, brush, oxbows, fallen logs, sand dunes, river banks, etc.), instituting management practices which hinder the mobility of species, and laying down of impervious material. Air, land, and water resources may be potentially impacted.

The project(s) should be designed so discharges will not be toxic to fish and wildlife resources. If any wetlands are constructed for treatment purposes, they must not attract wildlife to toxic areas or release any exotic plant/animal life to receiving waters.

If the project(s) affect tidal areas, an easement from the General Land Office may be required.

Nederland WW Study
4-004.C - Environmental Assessment

The U.S. Army Corps Engineers should be consulted to determine permit requirements relative to jurisdictional wetlands. Once the jurisdictional determination/delineation is completed, a U.S. Army Corps of Engineers/U.S. Environmental Protection Agency (EPA) "Section 404" permit may be required for land alteration activities affecting waters of the United States, including wetlands.

Compensation may be required for any encroachment into high value habitat areas. Should mitigation be required, habitat compensation plans should contain detailed descriptions of the proposed compensation areas. Detailed drawings should include plan-view locations of the encroachments and cross-section details including design features, construction, planting lists, and maintenance and monitoring schedules.

The Texas Parks & Wildlife Department's Legal Division should be consulted to evaluate activities involving the disturbance or taking of material from the beds or bottoms of State owned streambeds and bay bottoms. In addition, the Wildlife & Fisheries Division requires a permit for the placement (planting) of aquatic plants in waters of the State (as in habitat restoration projects).

The U.S. Fish & Wildlife Service should be consulted to assist in the evaluation of the proposed land alteration activities which may affect federally-listed rare, threatened, or endangered wildlife species.

Information relating to the potential for occurrence of threatened/endangered species near the project area from our Texas Natural Heritage Program has already been provided.

Project plans should include measures to prevent erosion and sediment runoff from disturbed areas and sedimentation into wetlands. Runoff control measures should be maintained until disturbed areas have been revegetated. Landscaping and revegetation should utilize existing drainage patterns and appropriate trees, grasses and shrubs native to the immediate area. Planting vegetation with value for wildlife would further enhance the aesthetics of the area.

We appreciate the opportunity to review and comment on this project.

Sincerely,

Jack Bauer

Jack Bauer
Conservation Scientist
Project Coordinator

JB:dab

DEPARTMENT OF THE ARMY
GALVESTON DISTRICT, CORPS OF ENGINEERS
P.O. BOX 1229
GALVESTON, TEXAS 77553-1229

REPLY TO ATTENTION OF:

DEC 20 1994

Enforcement Section

SUBJECT: D-6043; Jurisdictional Determination, Regional Wastewater Study, Jefferson County, Texas

Jeffrey G. Beaver, P.E.
Vice President
Schaumberg & Polk, Inc.
8865 College Street, Suite 100
Beaumont, Texas 77707

RECEIVED DEC 22 1994

Dear Mr. Beaver:

This concerns your August 26, 1994, letter requesting a jurisdictional determination for a regional wastewater study in Jefferson County. The study involves alternatives to improve the discharge of wastewater in the cities of Nederland, Port Neches, and Groves, Texas.

Based on the information you provided and a December 15, 1994, desk determination, we have determined that the installation of an outfall structure in the Neches River requires an individual permit pursuant to Section 10 of the Rivers and Harbors Act, provided there is not an associated intake and the outfall is located below the mean high water line. Should the outfall structure have an associated intake structure, the outfall could be authorized by Nationwide Permit 7, which requires notification to the District Engineer. The discharge of fill material in association with the outfall is subject to Section 404 of the Clean Water Act and may be authorized by a nationwide permit depending on the amount and location of the fill material. In the Neches River, the outfall must be a minimum of 50 feet from the top edge cut of the channel, however, there is not a minimum submergence requirement. The installation of an outfall into Rhodair Gully does not require a permit provided the outfall is located north of State Highway 365. Should final plans include an outfall in the Neches River or south of State Highway 365 in Rhodair Gully, you must submit an application detailing the project.

Nederland
4009.0
Environmental
Assessment

This verification is valid for a period of 5 years from the date of this letter unless new information warrants a revision of the determination prior to the expiration date. Please reference the determination number D-6043 in future correspondence pertaining to this subject. If you have any questions concerning this matter, please contact Mr. John Davidson, at the letterhead address or by telephone at (409) 766-3933.

Sincerely,

For Casey Cutler
Unit Leader, North Unit
Enforcement Section

Enclosure

SECTION 6 - WATER CONSERVATION

Task III. *Develop a Water Conservation Plan*

A Water Conservation Plan has been prepared for each of the three cities as a separate bound document.

SECTION 7 - INSTITUTIONAL CONSIDERATIONS

Task IV. *Evaluate Institution Considerations*

If a regional treatment facility had proven to be cost effective, appropriate managing entities would have been evaluated, and the "best fit" would have been recommended for creation. This study determined a regional facility not to be the most cost effective alternative and does not recommend regionalization. Since each city will continue to operate their own facilities no other institutions were considered for creation and use.

SECTION 8 - FINANCIAL PLANS

Task V. *Prepare appropiate financial plans to implement recommended alternatives.*

Financial plans for the three cities to finance construction of wastewater improvements and pay for increased operation and maintenance cost associated with those improvements over present levels are presented in a very simplified form in this section. The cities of Nederland and Groves have retained the services of a financial advisor and are currently working to identify to best financial plan to fund the necessary construction. We confined our development of financial plans to a very straight forward, very simple approach for the three cities. For the improvements recommended to each city developed the capital cost requirements and the operational and maintenance cost requirements on per a month basis. We then distributed the monthly cost for capital and for operational and maintenance over the wastewater connections in each city. This yields a monthly increase in cost over and above current rates which may be anticipated as being necessary to fund construction of the recommended improvements.

For the City of Port Neches, their share of the jointly operated lift station with Groves for discharge of their combined effluents to the Neches River, capital cost is $1,275,000.00. Anticipating Port Neches' preferred method of finance to be CO's, we used 7 percent for twenty years. This is $9,885.00 a month. We anticipate $5,400.00 a month in O & M expenses as their share of this facility for a total of $15,300.00 per month, divided by 4,932 connections, yields an increased cost to the wastewater customers of Port Neches of $3.10 per month.

For the City of Nederland the recommended alternative has a capital cost of $4,828,000.00. We anticipate an increase in O & M cost for additional blowers and also for operation of a lift station in order to pump their effluent to the Neches River at $100,000 a year. We anticipate financing of the capital cost through an SRF loan. We calculated a 6 percent loan for twenty years, this would require $34,589.00 per month to repay that debt, combined with the $8,334.00 per month for O & M, for a funding need of $42,923.29 per month for 6,261 connections. This would be an increased cost per month over present rates of $6.86 per month.

For the City of Groves the total of the recommended alternatives has a capital cost of $13,175,000.00. O & M costs additional to those cost already experienced are about $50,000 per year. Monthly costs were determined by amortizing $13,175,000.00 at 6 percent over twenty years, yields a cost for this money of $94,389.00 per month, O & M cost of $4,167.00 per month, yields a total of $98,556.00 per month for 6,916 connections The increased monthly bill for wastewater in the City of Groves to fund these improvements is $14.25 per month.

It is our understanding from conversations with the cities of Nederland and Groves that their intentions are to use Texas Water Development Board SRF financing to complete construction of the improvements required. The cost saving available to these cities through this program is almost too attractive to pass up, considering the amounts of money needed.

PORT NECHES

Capital Amortization 20 years @ 7%

$1,275,000 @ 7% (Market) for 20 years	$ 9,885.06/month
	$ 118,620.74/year
$65,000.00 per year O & M	$ 5,416.67/month
	$ 65,000.00/year

Monthly Cost

Debt Payment	$ 9,885.06
O & M	$ 5,416.67
TOTAL	$ 15,301.73/month

$\frac{\text{15,30173 cost/month}}{\text{4932 connnections}}$ = $ 3.10/month/connection

SPI No. 4004.01/10101.0/10201.0
DF: \B:\FNCLPLN.WPD
04/04/95

Schaumburg & Polk, Inc.
CONSULTING ENGINEERS

NEDERLAND

Captial Requirements	$4,828,000.00
O & M Cost (Additional)	$ 100,000.00/year
	$ 8,334.00/month
$4,828,000 @ 6% (SRF) for 20 years	$ 34,589.29/month
TOTAL	$ 415,071.50/year

Monthly Cost

Debt Payment	$ 34,589.29/month
O & M	$ 8,334.00/month
TOTAL	$ 42,923.29/month

$\frac{\$42,923.00 \text{ cost/month}}{6,261 \text{ connections}}$ = $6.86/connection/month

GROVES

RECOMMENDED ALTERNATIVES

PN/G-2	$ 634,000.00
G-1	$ 4,093,000.00
G-4	$ 8,448,000.00
	$13,175,000.00

Captial Required	$13,175,000.00
O & M Cost (Additional)	$ 50,000.00/year
	$4,167.00/month
13,175,000 @ 6% (SRF) for 20 years	$ 94,389.79/month
TOTAL	$ 1,132,677.50/year

Monthly Cost

Debt Payment	$ 94,389.79
O & M	$ 4,167.00
TOTAL	$ 98,556.79

$98,556.00 cost/month / 6,916 connections = $14.25/connection/month

APPENDIX A - Existing Wastewater Treatment Facilities

A1 - City of Nederland WWTF

A2 - City of Port Neches WWTF

A3 - City of Groves North WWTF

A4 - City of Groves South WWTF

Note: For each plant, excerpts from existing and/or proposed draft TNRCC permits are included.

A1 - CITY OF NEDERLAND WWTF

A1 - CITY OF NEDERLAND WWTF

A. General

The existing treatment units are analyzed according to current TNRCC design criteria for secondary treatment to determine the ratable plant sizing. The analysis is based on assumed secondary treatment requirements.

The plant is presently permitted for 3.8 mgd design flow (maximum monthly average) at secondary standards (20 mg/l BOD_5 and TSS) with a DO requirement of 2 mg/l. The permitted two hour peak flow is 8350 gpm (equivalent to 12.024 mgd). The permit is phased to allow no discharge after late 1996. However, the City has been granted a variance which can lead to an amended permit with parameters somewhere between no discharge and the existing secondary.

The plant was reportedly designed for an influent BOD_5 strength of 240 mg/l. However, the City began an influent testing program in February of 1992 with results considerably less than that value. The maximum influent strength as reported in the 1992 permit renewal application (based on testing to that date) was 100 mg/l. An average influent strength of 200 mg/l BOD_5 will be used in the analysis to determine rated capacity of the various treatment units.

The Nederland plant contains three parallel treatment tracks between preliminary treatment and chlorination. Two of these tracks consist of identical contact stabilization plants, while the third track is a trickling filter process. Sludge is digested aerobically. Treatment units are as described in the following sections.

B. Preliminary Treatment (Before splitting into tracks)

1. Mechanical Bar Screen. One screen, 7 ft. 5 in. ± length, 5 ft. total width, 30 bars, 3/8 in. width on 1 3/8" centers, with mechanical cleaning mechanism; design liquid depth 6 ft. maximum.

   Required: *Some form of screening; bar openings minimum ½" for mechanical screens; velocities @ design flow minimum 2 ft./sec through channel, < 3 ft./sec. through screen.*

   Analysis: Bar openings = 1"
   Channel Velocity = 5.88 cfs / (5 ft. x 6 ft.) = 0.2 fps
   Screen Velocity = 5.88 cfs / (30 x 1/12 ft. x 6 ft.) = 0.4 fps

2. Influent Lift Station. Four pumps, submersible type, installed in dry pit, each 2900 gpm capacity for firm capacity of 8700 gpm. (Two of the pumps are two speed with a slower speed of 900 gpm, with pump speed automatically adjusted as a function of wet well level. The 2900 gpm rated capacities are based on an average pumping head

between high and low wet well levels.)

Required: *Firm pumping capacity (largest pump out of service) must be adequate to pump peak flow to destination; three or more pumps (or duplex pumps with automatic variable speed control) required.*

Analysis: Firm capacity = 8700 gpm = 12.528 mgd peak flow.

[Although the rated pumping capacity slightly exceeds the permitted peak flow, it should be noted that the station pumps various internal flows (filtrate, drainage, certain supernatant, etc) as well as influent flows. It should also be noted that in addition to the varying pump speed, the actual pumping rate will vary according to liquid depth.]

3. Aerated Grit Chamber. 20 ft. x 20 ft. chamber, 13 ft. water depth (less 5 ft. x 12 ft. x 4.5 ft. splitter box for effluent), plus hopper bottom with 1:1 slope (reported basin volume of 6240 ft^3); two air diffusers (112 cfm total) with 30" draft tube; concentrated grit/liquid mixture sent to degritter for final grit separation.

Required: *Grit removal recommended; if removal units are provided, must have method of removing grit from unit, and any unit with single chamber must have bypass.*

Analysis: Grit removal by grit pump below; piping allows flow to bypass grit chamber if needed. This unit also provides preaeration.

4. Grit Pump. One vortex type pump, 250 gpm (pumps grit/liquid mixture from aerated grit chamber to degritter).

5. Degritter. Hydrocyclone (10.5 ft. long) and grit classifier/washer (L shaped, approx. 5 ft. x 25 ft. plus 4 ft. x 3 ft. (dewaters grit from aerated grit chamber).

C. Contact Stabilization Process. (Two identical tracks in parallel.)

1. Aeration Chambers. Segment of annular ring in each tank, 100 ft. O. D. x 60 ft. I. D. x 15 ft. depth, Contact zone is 94°, Reaeration zone is 144°, fine bubble diffusers. Diffuser capacity is reportedly 2150 cfm total (before adding reserve capacity) for all contact and reaeration chambers totaled.

Required: *Total volume (contact + reaeration) must be 1000 ft^3 per 50 lb. BOD_5/day. This volume should be divided with a ratio of 1 to 2 parts reaeration per part of contact zone. Diffused aeration, if used, must be designed for 1800 SCF per lb. BOD_5 (unless otherwise justified by improved diffuser efficiency). The diffuser system must be capable* *of providing 150% of design requirements.*

Analysis: Each Contact chamber = $\pi(50^2-30^2)(15)(94/360) = 19{,}687\ ft^3$
Total Contact volume = $2 \times 19{,}687\ ft^3 = 39{,}374\ ft^3$

Each Reaeration chamber = $\pi(50^2-30^2)(15)(144/360) = 30{,}159\ ft^3$
Total Reaeration volume = $2 \times 30{,}159\ ft^3 = 60{,}318\ ft^3$

Ratio of Reaeration to Contact = 144:94 = 1.5:1

Total aeration tank volume = $99{,}692\ ft^3$

Allowable loading = $(99{,}692\ ft^3)(50\ lb\ BOD_5/day/1000\ ft^3)$
= $4{,}985\ lb\ BOD_5/day$

At an influent strength of 200 mg/l, the total design flow capacity based on aeration volume is 2.99 mgd.

From the 1983 facility plan amendment (for the plant upgrading which included upgrading the package plants to their present state), the rated diffuser efficiency was 9.5% with a 36.8% reduction in air requirements. The amendment reported the air requirements (for both contact zones and both reaeration zones combined) as 2150 cfm total. It is assumed that this diffuser capacity (plus the required 50% reserve) was provided. (2150 cfm)[1/(1 - 0.368)](1440 min./day)/(1800 SCF per lb. BOD_5) = capacity for 2722 lb. BOD_5/day. At an influent strength of 200 mg/l, the total design flow capacity based on available aeration system is 1.63 mgd.

2. <u>Secondary Clarifiers</u>. Inner circle of each tank, 60 ft. diam. x 10 ft. side water depth, 14 ft. diam. feedwell, bottom slope toward center.

Required: *Maximum surface loading at Peak flow of 1400 gal./day/ft², and at Design flow of 700 gal./day/ft². Side water depth must be at least 10 ft. for surface areas of 1250 ft² or more. Effective detention times (based on liquid volume above a 3 ft. sludge blanket) must be 1.3 hr. @ Peak flow and 2.6 hr. @ Design flow.*

Analysis: <u>Allowable flow based on surface area</u>:
Effective surface area of each clarifier = $\pi(30^2 - 7^2) = 2673\ ft^2$
Total effective surface area = $2 \times 2673\ ft^2 = 5346\ ft^2$

Allowable Peak flow = $(5346)(1400)/10^6 = 7.48$ mgd
Allowable Design flow = $(5346)(700)/10^6 = 3.74$ mgd

Side water depth is adequate.

Allowable flow based on detention time:
Detention time is based on effective surface area and side water depth less three ft. $(5346)(10 - 3) = 37{,}422\ ft^3 \times 7.48\ gal./ft^3 = 279{,}917$ gal.

Allowable Peak flow = 279,917 gal./(1.3 hr.)(1 day/24 hr.)
= 5.17 mgd
Allowable Design flow = 279,917 gal./(2.6 hr.)(day/24 hr.)
= 2.58 mgd

The flows based on detention govern, since they are less than the flows based on surface area.

D. Trickling Filter Process. (Treated as one track, although it includes two parallel filters.)

1. Primary Clarifier. 65 ft. diam. x 12 ft. side water depth; bottom slopes to center @ 6:1; volume includes influent feedwell (6 ft. diam.), effluent trough; mechanical sludge collection.

Required: *Maximum surface loading at Peak flow of 1800 gal./day/ft^2, and at Design flow of 1000 gal./day/ft^2. Side water depth must be at least 7 ft.*

Analysis: Allowable flow based on surface area:
Effective surface area of clarifier = $\pi(32.5^2 - 3^2) = 3290\ ft^2$

Allowable Peak flow = $(3290)(1800)/10^6$ = 5.92 mgd
Allowable Design flow = $(3290)(1000)/10^6$ = 3.29 mgd

Side water depth is adequate.

Primary clarifier is considered to remove 35% of raw influent BOD_5.

2. Trickling Filter No. 1. Octagon, 48 ft. (as measured between midpoints of opposite sides), 4.5 ft. square center pier, 8 ft. media depth, synthetic media, rotary distributor (4 arm).

Required: *Sizing as recommended by filter media manufacturer; must reduce the influent BOD_5 from 65% of raw concentration to 20 mg/l per permit requirements.*

Analysis: Media area based on regular octagon, 48 ft. side to side.

Side:width ratio = $1:(1 + 2\sqrt{0.5}) = 1:2.414$

Side = width/2.414 = 48/2.414 = 19.88 ft.

Gross area = (Side x width) + 2[side^2 x ($\sqrt{0.5}$)(1 + $\sqrt{0.5}$)]
= 1909 ft^2

Net area (excluding 4.5 ft. square center pier) = 1889 ft^2

Media volume = 1889 x 8 = 15,112 ft^3.

Required efficiency = 85%. Per media manufacturer's (Munters Media #27060) curve @ 85% efficiency, allowable loading = 62 -63 lb. BOD_5/day/1000 ft^3.

lb. BOD_5/day = (62 lb. BOD_5/day/1000 ft^3)(15,112 ft^3)
= 937 lb. BOD_5/day

At an influent strength of 200 mg/l BOD_5, and allowing for 35% reduction through the primary clarifier, the allowable flow rate is 0.864 mgd.

3. <u>Trickling Filter No. 2</u>. 62 ft. diam., 4.5 ft square center pier, 8 ft. media depth, synthetic media, rotary distributor (4 arm).

Required: *See Trickling Filter 1 above.*

Analysis: Gross media area = $\pi(31^2)$ = 3019 ft^2.

Net area (deduct center pier) = 2999 ft^2

Media volume = 2999 x 8 = 23,992 ft^3.

Required efficiency = 85%. Per media manufacturer's (Munters Media #27060) curve @ 85% efficiency, allowable loading = 62 -63 lb. BOD_5/day/1000 ft^3.

lb. BOD_5/day = (62 lb. BOD_5/day/1000 ft^3)(23,992 ft^3)
= 1,488 lb. BOD_5/day

At an influent strength of 200 mg/l BOD_5, and allowing for 35% reduction through the primary clarifier, the allowable flow rate is 1.371 mgd.

4. Recirculation Pumps. Two pumps, 1000 gpm, recirculate portion of filter effluent back to filters.

Required: *Recirculation for low flow periods, sufficient to keep media wetted as recommended by manufacturer and to keep rotor arms turning.*

5. Final Clarifier. 70 ft. diam. x 12 ft. side water depth; bottom slopes to center @ 12:1.75; volume includes influent feedwell (approx. 10 ft. diam.), effluent trough; mechanical sludge collection.

Required: *Maximum surface loading at Peak flow of 1600 gal./day/ft^2, and at Design flow of 800 gal./day/ft^2. Side water depth must be at least 10 ft. for surface areas of 1250 ft^2 or more. Effective detention times (based on liquid volume above a 3 ft. sludge blanket) must be 1.1 hr. @ Peak flow and 2.2 hr. @ Design flow.*

Analysis: Allowable flow based on surface area:
Effective surface area of clarifier = $\pi(35^2 - 5^2) = 3770$ ft^2

Allowable Peak flow = $(3770)(1600)/10^6 = 6.03$ mgd
Allowable Design flow = $(3770)(800)/10^6 = 3.02$ mgd

Side water depth is adequate.

Allowable flow based on detention time:
Detention time is based on effective surface area and side water depth less three ft. $(3770)(12 - 3) = 33{,}930$ ft^3 x 7.48 gal./ft^3 = 253,796 gal.

Allowable Peak flow = 253,796 gal./(1.1 hr.)(1 day/24 hr.)
= 5.54 mgd
Allowable Design flow = 253,796 gal./(2.2 hr.)(day/24 hr.)
= 2.77 mgd

The flows based on detention govern, since they are less than the flows based on surface area.

E. Effluent Works. (Receives combined flows from both contact stabilization plants and trickling filter process.)

1. Chlorine Contact Chamber. Inside dimensions 98 ft. 6 in. x 38 ft. 2 in. including partitions and baffles; minimum liquid depth 6.25 ft. (6 ft. in final compartment); hopper bottoms in two 18.5 ft. x 18.75 ft. portions of chamber; fine bubble diffusers for mixing.

Required: *Detention time of 20 minutes @ peak flow.*

Analysis: Existing volume approximately 23,000 ft^3
23,000 ft^3/20 min. = 1150 cfm = 19.2 cfs = 12.39 mgd

2. Chlorine Feed Equipment. Two systems, each 500 lb./day feed capacity (vacuum operated) including one standby; flow proportioned; chlorine gas from one ton size containers.

Required: *Feed equipment must be able to provide more than the highest dosage to be required at any time. Dosage must be adequate to maintain a chlorine residual of at least 1 mg/l after 20 minutes detention, prior to dechlorination.*

Analysis: In standard practice, feed equipment is designed to feed 10 ppm of Cl_2 in order to assure a 1 mg/l residual.

(500 lb./day)/(10 ppm)(8.345 lb./gal.) = 5.99 mgd
If both feeders can be used simultaneously during peak flows, they would have a theoretical capacity for 11.98 mgd peak.

3. Dechlorination. Liquid ammonium bisulfate, 3000 gal. storage tank, one metering pump with 96 gal./day capacity; injection and reaction occur in a transitional area between chlorination and flow measurement. This dechlorination area is structurally an extension of the chlorine contact chamber, 8 ft. x 10 ft. 8 in. rectangle plus an adjacent trapezoidal area, 5 ft. long, width transitional from 8 ft. to 3 ft. 11 in.

Required: *The effluent, after chlorination and 20 minutes detention time, must be dechlorinated to less than 0.1 mg/l. For most dechlorination agents, 1 minute detention is generally considered adequate.*

Analysis: Approximately 691 ft^3 detention volume

4. Flow Measurement. 24 inch parshall flume; continuously indicating, recording, and totalizing flow meter calibrated to read up to 15 mgd.

Required: *Continuous effluent measurement required, with capacity for maximum expected peak flow.*

Analysis: Existing effluent measurement is adequate for peak flows up to 15 mgd.

5. Postaeration. Postaeration is accomplished by a cascading effect as the effluent drops from the flow measurement device to the effluent line.

F. Sludge Processing.

1. Sludge Pumps (Final Clarifier). Two, submersible, 100 gpm, pumping secondary sludge to thickener.

2. Sludge Thickener. 38 ft. diam. x 14 ft. side water depth, bottom slopes to center @ 4:1; mechanical sludge collection with pickets; supernatant to Trickling Filter No. 1.

Required: *Aerobic digesters should be provided with sludge thickening.*

3. Sludge Pumps (Thickener). Two, 250 gpm, self priming centrifugal screw type, pumping thickened sludge to digestion.

4. Aerobic Digesters. Two, one in each contact stabilization plant in annular area; 100 ft. O. D. x 60 ft. I. D. x 122° x 15 ft. depth (reported 25,554 ft^3 each), fine bubble diffusers. Available information shows a diffuser capacity of 1533 cfm.

Requirement: *Minimum solids retention time of 15 days (may be calculated as 20 ft^3 for each lb. influent BOD_5 per day). Diffused air requirement is 30 cfm per 1000 ft^3 of volume.*

Analysis: Volume for each digester = $\pi(50^2\text{-}30^2)(15)(122/360)$ = 25,552 ft^3. Total digestion volume = 51,108 ft^3 total.

Allowable BOD_5 = 51,108/20 = 2555 lb. BOD_5/day.

At an influent strength of 200 mg/l, this volume is sufficient for 1.53 mgd design flow. [The use of the thickener prior to digestion may increase digester capacity]

Required .aeration = (30 cfm/1000 ft^3)(51,104 ft^3) = 1533 cfm required. The 1983 facility plan amendment suggests that the diffuser capacity is equal to the required amount.

5. Centrifuge Facility. One sludge grinder; two sludge metering pumps, progressive cavity, 60 gpm; one polymer feed pump (for 6% solution); two 200 gallon polymer mixers; one polymer metering pump; one horizontal centrifuge, 60 gpm with 20 hp motor and mixing tank to introduce polymer into sludge.

6. Drying Beds. Two sets of open sand beds, 76 ft. x 220 ft. and 50 x 100 ft.; used for standby only.

G. Blowers. Four blowers, 1500 cfm each, supplying air for contact stabilization (contact and reaeration); aerobic digesters; aerated grit chamber; chlorine contact chamber mixing; and airlift pumps. Existing firm capacity = 4500 cfm.

Required: *Blowers must be able to meet maximum aeration requirements with largest unit out of service.*

H. Plant Capacity

		ADF	PEAK
1.	Influent Lift Station		12.528 mgd
2.	Contact Stabilization Process		
	Aeration Chambers	2.99 mgd	
	Final Clarifiers	2.58 mgd	5.17 mgd
3.	Trickling Filter Process		
	Primary Clarifier	3.29 mgd	5.92 mgd
	Trickling Filter No. 1	0.86 mgd	
	Trickling Filter No. 2	1.37 mgd	
	Final Clarifier	2.77 mgd	5.54 mgd
4.	Effluent Works		
	Chlorine Contact Chamber		12.39 mgd
	Chlorine Feed Equipment		11.98 mgd
	Flow Measurement		15.0 mgd
5.	Sludge Processing		
	Aerobic Digesters	1.53 mgd	

TEXAS WATER COMMISSION

IN THE MATTER OF THE	§	
APPLICATION OF THE CITY OF	§	BEFORE THE
NEDERLAND FOR A RENEWAL OF	§	
PERMIT NO. 10483-002	§	TEXAS WATER COMMISSION

AN ORDER FOR A TEMPORARY VARIANCE FROM TEXAS SURFACE WATER QUALITY STANDARDS

On this the 8th day of December, 1993, the Texas Water Commission ("Commission" or "TWC"), at a hearing pursuant to notice properly and timely given, considered the application of the City of Nederland, ("Applicant" or "Nederland"), for an temporary variance pursuant to 31 Texas Administrative Code ("TAC") §307.2(d)(4).

Having heard the argument of the parties, the Commission is satisfied that the applicant has satisfied the requirements of 31 TAC §307.2(d)(4), therefore, the Commission finds that the temporary variance should be approved.

FINDINGS OF FACT

1. Preliminary evidence indicates that a site specific water quality standards amendment for a series of perennial canals in Jefferson County which are tributaries of Taylor Bayou from a classification of high "presumed" quality aquatic life use to intermediate quality aquatic life use is appropriate.

2. The City of Nederland's treatment plant, Permit No. 10483-002, is an existing permitted discharge facility.

3. The City of Nederland applied for a temporary variance during the permit renewal application process.

4. Notice of the temporary variance request was included in the public notice of the permit application.

5. The variance shall not exceed a time period of three years.

CONCLUSIONS OF LAW

1. The above facts are conditions sufficient to issue this order pursuant to 31 TAC §307.2(d)(4).

2. Issuance of this order will effectuate the purposes of Chapter 26 of the Texas Water Code.

NOW, THEREFORE, BE IT ORDERED BY THE TEXAS WATER COMMISSION THAT:

1. The City of Nederland is granted a temporary variance to existing water quality standards for a series of perennial canals in Jefferson County which are tributaries of Taylor Bayou.

2. The City of Nederland will conduct a study of the perennial canals in Jefferson County into which its treatment plant discharges treated domestic wastewater effluent to show whether a site specific amendment to water quality standards is justified.

3. If the Commission adopts the site specific standards for a series of perennial canals in Jefferson County which are tributaries of Taylor Bayou, the City of Nederland shall apply for a permit amendment to meet revised water quality standards.

4. If the Commission does not approve the site specific standard prior to the expiration of the variance period, then final effluent limits based on existing water quality standards shall remain in effect.

5. This temporary variance shall expire three years from the date of issuance of this Order.

6. The Chief Clerk of the Commission is directed to forward a copy of this Order to the Applicant and all other parties and to issue the Order and cause it to be recorded in the files of the Commission.

Issued this date: December 14, 1993

TEXAS WATER COMMISSION

John Hall

John Hall, Chairman

ATTEST:

Gloria A. Vasquez

Gloria Vasquez, Chief Clerk

PERMIT NO. 10483-002
(corresponds to
NPDES PERMIT NO. TX0026476)

This is a renewal of Permit No. 10483-002, approved January 5, 1988.

TEXAS WATER COMMISSION
Stephen F. Austin State Office Building
1700 N. Congress Ave.
Austin, Texas 78711

PERMIT TO DISPOSE OF WASTES
under provisions of Chapter 26
of the Texas Water Code

City of Nederland

whose mailing address is

P.O. Box 967
Nederland, Texas 77627

is authorized to treat and dispose of wastes from the wastewater treatment facilitie

located immediately east of the intersection of Hardy Avenue and Avenue D, east of th main drainage canal in the City of Nederland in Jefferson County, Texas

to an intermittent concrete lined ditch; thence into a series of perennial canals thence into Taylor Bayou; thence into the Intracoastal Waterway in Segment No. 0702 o the Neches-Trinity Coastal Basin

only in accordance with effluent limitations, monitoring requirements and othe conditions set forth herein, as well as the rules of the Texas Water Commissio ("Commission"), the laws of the State of Texas, and other orders of the Commission The issuance of this permit does not grant to the permittee the right to use privat or public property for conveyance of wastewater along the herein described discharg route. This includes property belonging to but not limited to any individual partnership, corporation or other entity. Neither does this permit authorize an invasion of personal rights nor any violation of federal, state, or local laws o regulations. It is the responsibility of the permittee to acquire property rights a may be necessary to use the herein described discharge route.

This permit and the authorization contained herein shall expire at midnight, five year after the date of Commission approval.

ISSUED DATE: DEC 14 1993

ATTEST: Gloria A. Vasquez

John Hall
For the Commission

INTERIM EFFLUENT LIMITATIONS AND MONITORING REQUIREMENTS

Outfall Number 001

1. During the period beginning upon the date of issuance and lasting through March 31, 1996*, the permittee is authorized to discharge subject to the following effluent limitations:

   The daily average flow of effluent shall not exceed 3.8 million gallons per day (MGD); nor shall the average discharge during any two-hour period (2-hour peak) exceed 8,350 gallons per minute (gpm).

Effluent Characteristic	Discharge Limitations				Minimum Self-Monitoring Requirements	
	Daily Avg mg/l(lbs/day)	7-day Avg mg/l	Daily Max mg/l	Single Grab mg/l	Report Daily Avg. & Daily Max. Measurement Frequency	Sample Type
Flow, MGD	Report	N/A	Report	N/A	Continuous	Totalizing meter
Biochemical Oxygen Demand (5-day)	20(634)	30	45	65	Two/week	Composite
Total Suspended Solids	20(634)	30	45	65	Two/week	Composite

2. The effluent shall contain a chlorine residual of at least 1.0 mg/l after a detention time of at least 20 minutes (based on peak flow), and shall be monitored daily by grab sample. The permittee shall dechlorinate the chlorinated effluent to less than 0.1 mg/l chlorine residual and shall monitor daily by grab sample after the dechlorination process. An equivalent method of disinfection may be substituted only with prior approval of the Commission.

3. The pH shall not be less than 6.0 standard units nor greater than 9.0 standard units and shall be monitored once per week by grab sample.

4. There shall be no discharge of floating solids or visible foam in other than trace amounts and no discharge of visible oil.

5. Effluent monitoring samples shall be taken at the following location(s): Following the final treatment unit.

6. The effluent shall contain a minimum dissolved oxygen of 2.0 mg/l and shall be monitored twice per week by grab sample.

* See Other Requirement No. 1, Page 8.

OTHER REQUIREMENTS

1. FINAL PHASE - During the period beginning upon April 1, 1996 and lasting through the date of expiration, no discharge of pollutants into waters in the State is authorized and the following provisions apply:

Conditions of the permit: No discharge of pollutants to surface water in the State is authorized.

Character: Treated Domestic Sewage Effluent

Volume: 30-day Average - 3.8 MGD from the treatment system

Quality: The following degree of treatment shall be required:

A.

Parameter	30-day Average	Effluent Concentrations (Not to Exceed) Single Grab
BOD_5, mg/l	20	65
TSS, mg/l	20	65

The pH shall not be less than 6.0 standard units nor greater than 9.0 standard units.

The effluent shall be chlorinated in a chlorine contact chamber to a residual of 1.0 mg/l with a minimum detention time of 20 minutes.

B. Monitoring Requirements:

Parameter	Monitoring Frequency	Sample Type
Flow, MGD	Five/week	Instantaneous
BOD_5, mg/l	One/month	Grab
pH	One/month	Grab
Chlorine, mg/l	Five/week	Grab

The monitoring shall be done after the final treatment unit. These records shall be maintained on a monthly basis and be available at the plant site for inspection by authorized representatives of the Commission for at least three years.

2. This Category B facility shall be operated and maintained by a chief operator or operator in responsible charge holding a valid Class B certificate of competency or higher issued pursuant to Chapter 31 TAC Texas Administrative Code Section 325. All shift supervisors and other plant operators shall be certified in accordance with the provisions of the Chapter therein.

OTHER REQUIREMENTS

1. FINAL PHASE - During the period beginning upon April 1, 1996 and lasting through the date of expiration, no discharge of pollutants into waters in the State is authorized and the following provisions apply:

Conditions of the permit: No discharge of pollutants to surface water in the State is authorized.

Character: Treated Domestic Sewage Effluent

Volume: 30-day Average - 3.8 MGD from the treatment system

Quality: The following degree of treatment shall be required:

A.

	Effluent Concentrations (Not to Exceed)	
Parameter	30-day Average	Single Grab
BOD_5, mg/l	20	65
TSS, mg/l	20	65

The pH shall not be less than 6.0 standard units nor greater than 9.0 standard units.

The effluent shall be chlorinated in a chlorine contact chamber to a residual of 1.0 mg/l with a minimum detention time of 20 minutes.

B. Monitoring Requirements:

Parameter	Monitoring Frequency	Sample Type
Flow, MGD	Five/week	Instantaneous
BOD_5, mg/l	One/month	Grab
pH	One/month	Grab
Chlorine, mg/l	Five/week	Grab

The monitoring shall be done after the final treatment unit. These records shall be maintained on a monthly basis and be available at the plant site for inspection by authorized representatives of the Commission for at least three years.

2. This Category B facility shall be operated and maintained by a chief operator or operator in responsible charge holding a valid Class B certificate of competency or higher issued pursuant to Chapter 31 TAC Texas Administrative Code Section 325. All shift supervisors and other plant operators shall be certified in accordance with the provisions of the Chapter therein.

3. Within one year of permit issuance, the permittee shall submit to the Texas Water Commission, Wastewater Permits Section, Watershed Management Division and the District Office of the Texas Water Commission a study that investigates the possibility of substituting reclaimed water for potable water and/or freshwater where such substitution would be both appropriate and cost effective pursuant to Chapter 31 TAC Section 305.126(b). At a minimum, the study shall include:

   a. a water supply and demand assessment for the area served;
   b. an inventory of potential areas where reclaimed water may be appropriately substituted for potable water and/or freshwater;
   c. an inventory of potential uses of reclaimed water;
   d. an analysis of the market for reclaimed water and the conditions necessary to serve that market (eg. quantity, quality, selling price, distribution system); and
   e. a preliminary cost-benefit analysis for the treatment and use of reclaimed water compared with the continued use of potable water and/or freshwater, water supply augmentation, water conservation, and/or cost of treatment and disposal of treated wastewater.

   Forty-five (45) days prior to implementation of an approved Use of Reclaimed Water program, the permittee shall provide written notice to the Austin Office, Watershed Management Division, Enforcement Support Unit and District Office of the Commission. The sampling and monitoring required under Chapter 31 TAC Section 310.10 to 310.13 shall be submitted by the 25th of each month.

4. The permittee shall submit within two years from the date of permit issuance an amendment application providing information about the no discharge facility to the Texas Water Commission, Municipal Permitting, Watershed Management Division.

5. The permittee shall obtain approval from the Watershed Management Division, Plans and Specs Review Unit of an engineering report and/or plans and specifications that clearly show how the treatment system will meet the final permitted no discharge requirements on Page 8, prior to construction.

6. The permittee shall comply with the following sludge requirements:

   A. The permittee is authorized to dispose of sludge at a co-disposal landfill or commercial land application site permitted by the Texas Water Commission. <u>The disposal of sludge by land application on property owned, leased or under the direct control of the permittee is a violation of the permit</u>.

   B. The permittee shall use only those sewage sludge disposal practices that comply with the federal regulations for landfills and solid waste disposal established in 40 CFR Part 257 and 258 and in accordance with all the applicable rules of the Texas Water Commission.

A2 - CITY OF PORT NECHES WWTF

A2 - CITY OF PORT NECHES WWTF

A. General

The existing treatment units are analyzed according to current TNRCC design criteria for secondary treatment to determine the ratable plant sizing. The analysis is based on assumed secondary treatment requirements.

The City of Port Neches has a single wastewater treatment facility (WWTF) serving the City. The plant is presently permitted for 4.98 mgd design flow (maximum monthly average) at secondary standards (20 mg/l BOD_5 and TSS) with a DO requirement of 5 mg/l. The permitted two hour peak flow is 26.0 mgd.

The WWTF is utilizes the fixed film treatment process for treatment of the wastewater flows. The existing treatment units treat to meet secondary effluent limits as required by the City's discharge permit. The major wastewater treatment units consist of the headworks including a comminutor, manually cleaned bar screen, and flow measuring device; aerated grit basin including a grit classifier; primary clarifier; trickling filters including one (1) primary and two (2) secondary; two (2) final clarifiers; two (2) stormwater clarifiers; and chlorination facilities. Under normal operation, the two stormwater clarifiers follow the two final clarifiers. During storm flow, several automatic gates and valves divert normal plant flow around the stormwater clarifiers. The stormwater flows are directed around the main treatment units to the stormwater clarifiers for solids settling prior to disinfection and discharge.

Sludge treatment units consist of the grit classifier; primary and secondary digesters; drying beds; and belt press. Dried sludge is disposed of at a landfill.

B. Preliminary Treatment

1. Comminutor/Manual Bar Screen. One comminutor and one manually cleaned bar screen. Effective capacity = 9.0 mgd.

   Required: *Some form of screening; where shredders are used, a backup unit or manually cleaned bar screen shall be provided.*

2. Aerated Grit Chamber. Two, 22' x 22' chambers, 13'-6" water depth, plus hopper bottom with 1:1 slope (report capacity of 13,100 ft^3). Each chamber is equipped with an air diffuser with 36" draft tube; Two 260 cfm blowers. Concentrated grit/liquid mixture is sent to a grit classifier.

   Required: *Grit removal recommended; if removal units are provided, must have method of removing grit from unit, and any unit with single chamber must have bypass.*

3. Primary Clarifiers. Two 60 ft. diam. x 8.5 ft. side water depth, total surface area is 5655 ft^2, bottom slopes to center.

Required: *Maximum surface loading at Peak flow of 1800 gal./day/ft^2, and at Design flow of 1000 gal./day/ft^2. Side water depth must be at least 7 ft.*

Analysis: Effective surface area of clarifier = 2 x $\pi(30^2)$ = 5655 ft^2

Allowable Peak flow = (5655)(1800)/10^6 = 10.18 mgd
Allowable Design flow = (5655)(1000)/10^6 = 5.66 mgd

Side water depth is adequate.

Primary clarifier is considered to remove 35% of raw influent BOD_5.

C. Secondary Treatment

1. Primary Trickling Filter. 60 ft. diam. x 5.25 ft. media depth, synthetic media, 2827 ft^2 surface area, Two 1680 gpm recirculation pumps.

Required: *Sizing as recommended by filter media manufacturer; must reduce the influent BOD_5 from 65% of raw concentration to 20 mg/l per permit requirements.*

Analysis: Gross media area = $\pi(30^2)$ = 2827 ft^2

Media volume = 2827 x 5.25 = 14,842 ft^3 {0.34 acre-ft.}

Per Texas Water Commission letter, dated September 28, 1989, the Port Neches WWTF is designed to treat 3157 lb. of BOD_5 per day.

Hydraulic Loading = 4,980,000 gpd / 2827 ft^2
= 1,762 gpd/ft^2

Organic Loading $= \dfrac{65\%(3157 \text{ lb. } BOD_5 \text{ per day})}{14{,}842 \text{ ft}^3}$
= 138.3 lb BOD/day/1000 ft^3

Calculate the efficiency of the trickling filters based on the NRC formula.

$E_1 = 1 / [1 \pm m(i)^n]$

Where:

n	=	0.5
m	=	0.0085
i	=	W/VF
W	=	lb. BOD to first stage of filter
V	=	ac-ft of trickling filter media
F	=	recirculation factor

$F = [1 \pm R/I] / [1 \pm (1 - f)R/I]^2$

Where:

R	=	rate of recirculation *(assume 5.33mgd)*
I	=	rate of raw influent
f	=	weighing factor, generally taken as 0.9 for domestic sewage

$$F = \frac{1 + (5.33/4.98)}{[1 + (1 - 0.9)(5.33 / 4.98)]^2}$$

$$= 1.7$$

$$E_1 = \frac{1}{1 + 0.0085 \{2052 / [(0.34)(1.7)]\}^{0.5}}$$

$$= 0.663 \text{ (or 66.3\%)}$$

2. Secondary Trickling Filters. Two, 60 ft. diam x 5.25 ft. media depth, synthetic media, 5655 ft^2 total surface area, two 3125 gpm load pumps per each filter, and two 1300 gpm recirculation pumps per each filter.

Required: *See Primary Trickling Filter above.*

Analysis: Gross media area = 2 x $\pi(30^2)$ = 5655 ft^2

Media volume = 5655 x 5.25 = 29,689 ft^3 {0.68 acre-ft.}

Hydraulic Loading = 4,980,000 gpd / 5655 ft^2
= 881 gpd/ft^2

Organic Loading $= \dfrac{65\%(3157 \text{ lb. } BOD_5 \text{ per day})(33.7\%)}{29{,}689 \text{ ft}^3}$
= 23.3 lb BOD/day/1000 ft^3

Calculate the efficiency of the trickling filters based on the NRC formula.

$$E_2 = 1 / [1 + (m / 1 - E_1)(i)^n]$$

Where:	n	= 0.5
	m	= 0.0085
	i	= W_2/VF
	W_2	= lb. BOD to second-stage filter
	V	= ac-ft of trickling filter media
	F	= recirculation factor

$$F = [1 + R/I] / [1 + (1 - f)R/I]^2$$

Where:	R	= rate of recirculation *(assume 2.43 per J&N Report)*
	I	= rate of raw influent
	f	= weighing factor, generally taken as 0.9 for domestic sewage

$$F = \frac{1 + (2.43)}{[1 + (1 - 0.9)(2.43)]^2}$$

$$= 2.22$$

$$E_1 = \frac{1}{1 + (0.0085 / 1 - 0.663) \{692 / [(0.68)(2.22)]\}^{0.5}}$$

$$= 0.649 \text{ (or 64.9 \%)}$$

Effluent BOD_5 = (692 lb./day)(1 - 0.649)
= 242.9 lb./day

3. Final Clarifiers. Two, 60 ft. diam. x 10 ft. side water depth, total surface area of 5655 ft^2, bottoms slope to center.

Required: *Maximum surface loading at Peak flow of 1600 gal./day/ft^2, and at Design flow of 800 gal./day/ft^2. Side water depth must be at least 10 ft. for surface areas of 1250 ft^2 or more.*

Analysis: Surface area of each clarifier = $\pi(30^2)$ = 2827 ft^2
Total surface area of clarifiers = 2 x $\pi(30^2)$ = 5655 ft^2

Allowable Peak flow = $(5655)(1600)/10^6$ = 9.05 mgd

Allowable Design flow = $(5655)(800)/10^6 = 4.52$ mgd
Side water depth is adequate.

D. Stormwater Clarifiers. The stormwater clarifiers are operated as second stage final clarifiers during flows of 9 mgd or less, and will receive direct stormwater when flows exceed 9 mgd during storm events.

1. Stormwater Clarifiers. Two, 80 ft. x 12 ft. side water depth, total surface area of 10,053 ft^2, bottoms slope to center.

Required: *Maximum surface loading at Peak flow of 1600 gal./day/ft^2, and at Design flow of 800 gal./day/ft^2. Side water depth must be at least 10 ft. for surface areas of 1250 ft^2 or more.*

Analysis: Surface area of each clarifier = $\pi(40^2) = 5027$ ft^2
Total surface area of clarifiers = $2 \times \pi(40^2) = 10{,}053$ ft^2

Allowable Peak flow = $(10{,}053)(1600)/10^6 = 16.08$ mgd
Allowable Design flow = $(10{,}053)(800)/10^6 = 8.04$ mgd

Side water depth is adequate.

E. Effluent Works.

1. Chlorine Contract Chamber. Two chambers, total tank volume of 12,533 ft^3, average water depth of 6.3 ft.

Required: *Detention time of 20 minutes @ peak flow.*

Analysis: Existing volume 12,533 ft^3*
12,533 ft^3/20 min. = 627 cfm = 10.4 cfs = 6.75 mgd*
* Recently enlarged per permit requirements for 9 mgd.

2. Dechlorination.

Required: *The effluent, after chlorination and 20 minutes detention time, must be dechlorinated to less than 0.1 mg/l. For most dechlorination agents, 1 minute detention is generally considered adequate.*

3. Post-Aeration. Diffused aeration in a portion of chlorine chamber structure.

4. Flow Measurement.

Required: *Continuous effluent measurement required, with capacity for*

maximum expected peak flow.

F. Sludge Processing.

1. First Stage Digester. 36,320 ft^3, heating and mixing equipment.

Required: *Minimum solids retention time of 15 days required for unheated anaerobic digesters. 19.0 $ft^3/lb\ BOD_5/day$ required.*

Analysis: Volume = 36,320 ft^3

Allowable BOD_5 = 36,320 ft^3 / 19.0 $ft^3/lb\ BOD_5/day$
= 1912 lb. BOD_5/day.

Per Texas Water Commission letter, dated September 28, 1989, the Port Neches WWTF is designed to treat 3157 lb. of BOD_5 per day. Therefore, at a design flow of 4.98 mgd this equals 76 mg/l BOD_5.

At 76 mg/l BOD_5, the first stage digester would be rated for a flow of 3.01 mgd.

2. Second Stage Digester. 33,120 ft^3.

Requirement: *Minimum solids retention time of 30 days required for unheated anaerobic digesters. 26.5 $ft^3/lb\ BOD_5/day$ required.*

Analysis: Volume = 33,120 ft^3

Allowable BOD_5 = 33,120 ft^3 / 26.5 $ft^3/lb\ BOD_5/day$
= 1250 lb. BOD_5/day.

At 76 mg/l BOD_5, the second stage digester would be rated for a flow of 1.97 mgd.

3. Belt Filter Press.

G. Plant Capacity

		ADF	PEAK
1.	Primary Clarifier	5.66 mgd	10.18 mgd
2.	Final Clarifier	4.52 mgd	9.05 mgd
3.	Stormwater Clarifiers	8.04 mgd	16.08 mgd
4.	Chlorine Contact Chamber		6.75 mgd
5.	Anaerobic Digesters	4.98 mgd	

TEXAS NATURAL RESOURCE CONSERVATION COMMISSION

IN THE MATTER OF THE APPLICATION OF THE CITY OF PORT NECHES FOR RENEWAL Of PERMIT NO. 10477-004	§ § § §	BEFORE THE TEXAS NATURAL RESOURCE CONSERVATION COMMISSION

AN ORDER FOR A TEMPORARY VARIANCE FROM
TEXAS SURFACE WATER QUALITY STANDARDS

On this the 11th day of May, 1994, the Texas Natural Resource Conservation Commission ("Commission" or "TNRCC"), at a hearing pursuant to notice properly and timely given, considered the application of the City of Port Neches, ("Applicant" or "Port Neches"), for an temporary variance pursuant to 30 Texas Administrative Code ("TAC") §307.2(d) (4).

Having heard the argument of the parties, the Commission is satisfied that the applicant has satisfied the requirements of 30 TAC §307.2(d)(4), therefore, the Commission finds that the temporary variance should be approved.

FINDINGS OF FACT

1. The TNRCC Water Quality Standards Team has determined that the criteria for the perennial Jefferson County Drainage District Canals in Segment No. 0702 of the Neches-Trinity Coastal Basin should be lowered to intermediate quality aquatic life uses. This change in criteria will require a revision to the Water Quality Standards and approval from EPA during the 1994 triennial revision of the standards.

2. The City of Port Neches's plant is an existing permitted discharge facility.

3. The City of Port Neches applied for a temporary variance during the permit renewal application process.

4. Notice of the temporary variance request was included in the public notice of the permit application.

5. The variance shall not exceed a time period of two years.

CONCLUSIONS OF LAW

1. The above facts are conditions sufficient to issue this order pursuant to 30 TAC §307.2(d)(4).

2. Issuance of this order will effectuate the purposes of Chapter 26 of the Texas Water Code.

NOW, THEREFORE, BE IT ORDERED BY THE TEXAS NATURAL RESOURCE CONSERVATION COMMISSION THAT:

1. The City of Port Neches is granted a temporary variance to existing water quality standards of the perennial Jefferson County Drainage District Canals in Segment No. 0702 of the Neches-Trinity coastal Basin.

2. The City of Port Neches will evaluate several options that would result in compliance with new effluent limitations. These options include 1) upgrading the treatment system to advanced levels, 2) rerouting the effluent to the Lower Neches River Tidal Segment 0601 and 3) joining a regional wastewater treatment system in the area.

3. If the Commission adopts the site specific standards for the perennial Jefferson County Drainage District Canals in Segment No. 0702, the City of Port Neches shall apply for a permit amendment to meet revised water quality standards.

4. If the Commission does not approve the site specific standard prior to the expiration of the variance period, then final effluent limits based on existing water quality standards shall remain in effect.

5. This temporary variance shall expire two years from the date of issuance of this Order.

6. The Chief Clerk of the Commission is directed to forward a copy of this Order to the Applicant and all other parties and to issue the Order and cause it to be recorded in the files of the Commission.

Issued this date: MAY 13 1994

TEXAS NATURAL RESOURCE CONSERVATION COMMISSION

John Hall

John Hall, Chairman

ATTEST:

Gloria A. Vasquez

Gloria Vasquez, Chief Clerk

C: LEON
Rudy

PERMIT NO. 10477-004
(corresponds to
NPDES PERMIT NO. TX0022926)

This is a renewal of Permit No. 10477-004, approved December 13, 1988.

TEXAS NATURAL RESOURCE CONSERVATION COMMISSION
Stephen F. Austin State Office Building
1700 N. Congress Ave.
Austin, Texas 78711

PERMIT TO DISPOSE OF WASTES
under provisions of Chapter 26
of the Texas Water Code

City of Port Neches

whose mailing address is

P.O. Box 758
Port Neches, Texas 77651

is authorized to treat and dispose of wastes from the Main Plant Wastewater Treatment Facilities

located approximately 1 mile northwest of the intersection of State Highway 347 and State Highway 73 in the 6100 block of Georgia Street in Jefferson County, Texas

to a concrete lined Jefferson County Drainage District No. 7 (DD7) drainage canal; thence to DD7 Canal A; thence to Alligator Bayou; thence to Taylor Bayou; thence to DD7 Main Outfall Canal; thence to the Intracoastal Waterway in Segment No. 0702 of the Neches-Trinity Coastal Basin

only in accordance with effluent limitations, monitoring requirements and other conditions set forth herein, as well as the rules of the Texas Natural Resource Conservation Commission ("Commission"), the laws of the State of Texas, and other orders of the Commission. The issuance of this permit does not grant to the permittee the right to use private or public property for conveyance of wastewater along the herein described discharge route. This includes property belonging to but not limited to any individual, partnership, corporation or other entity. Neither does this permit authorize any invasion of personal rights nor any violation of federal, state, or local laws or regulations. It is the responsibility of the permittee to acquire property rights as may be necessary to use the herein described discharge route.

This permit and the authorization contained herein shall expire at midnight, five years after the date of Commission approval.

ISSUED DATE: MAY 13 1994

ATTEST: Gloria A. Vasquez

John Hall
For the Commission

RECEIVED 6/8/94

INTERIM EFFLUENT LIMITATIONS AND MONITORING REQUIREMENTS

Outfall Number 001

1. During the period beginning upon the date of issuance and lasting through the April 30, 1997*, the permittee is authorized to discharge subject to the following effluent limitations:

The daily average flow of effluent shall not exceed 4.98 million gallons per day (MGD); nor shall the average discharge during any two-hour period (2-hour peak) exceed 6,250 gallons per minute (gpm).

Effluent Characteristic	Discharge Limitations				Minimum Self-Monitoring Requirements	
	Daily Avg mg/l(lbs/day)	7-day Avg mg/l	Daily Max mg/l	Single Grab mg/l	Report Daily Avg. & Daily Max. Measurement Frequency	Sample Type
Flow, MGD	Report	N/A	Report	N/A	Continuous	Totalizing meter
Biochemical Oxygen Demand (5-day)	20(832)	30	45	65	One/day	Composite
Total Suspended Solids	20(832)	30	45	65	One/day	Composite

2. The effluent shall contain a chlorine residual of at least 1.0 mg/l after a detention time of at least 20 minutes (based on peak flow), and shall be monitored daily by grab sample. The permittee shall dechlorinate the chlorinated effluent to less than 0.1 mg/l chlorine residual and shall monitor daily by grab sample after the dechlorination process. An equivalent method of disinfection may be substituted only with prior approval of the Commission.

3. The pH shall not be less than 6.0 standard units nor greater than 9.0 standard units and shall be monitored once per week by grab sample.

4. There shall be no discharge of floating solids or visible foam in other than trace amounts and no discharge of visible oil.

5. Effluent monitoring samples shall be taken at the following location(s): Following the final treatment unit.

6. The effluent shall contain a minimum dissolved oxygen of 5.0 mg/l and shall be monitored once per day by grab sample.

* See Other Requirement No. 1, Page 9.

OTHER REQUIREMENTS

1. FINAL PHASE - During the period beginning upon May 1, 1997 and lasting through the date of expiration, no discharge of pollutants into waters in the State is authorized and the following provisions apply:

Conditions of the permit: No discharge of pollutants to surface water in the State is authorized.

Character: Treated Domestic Sewage Effluent

Volume: 30-day Average - 4.98 MGD from the treatment system

Quality: The following degree of treatment shall be required:

A.

Parameter	30-day Average	Effluent Concentrations (Not to Exceed) Single Grab
BOD_5, mg/l	20	65
TSS, mg/l	20	65

The pH shall not be less than 6.0 standard units nor greater than 9.0 standard units.

The effluent shall be chlorinated in a chlorine contact chamber to a residual of 1.0 mg/l with a minimum detention time of 20 minutes.

B. Monitoring Requirements:

Parameter	Monitoring Frequency	Sample Type
Flow, MGD	Five/week	Instantaneous
BOD_5, mg/l	One/week	Grab
pH	One/week	Grab
Chlorine, mg/l	Five/week	Grab

The monitoring shall be done after the final treatment unit. These records shall be maintained on a monthly basis and be available at the plant site for inspection by authorized representatives of the Commission for at least three years.

2. This Category B facility shall be operated and maintained by a chief operator or operator in responsible charge holding a valid Class B certificate of competency or higher issued pursuant to 30 TAC Chapter 325. All shift supervisors and other plant operators shall be certified in accordance with the provisions of the Chapter therein. Note, Class D certificates are not renewable at any activated sludge facility, regardless of size, or any trickling filter or RBC facility with a permitted flow greater than 100,000 gallons per day.

3. Prior to May 1, 1997, the permittee shall submit an amendment request detailing how the permittee will meet the requirements of Page 9.

4. The permittee shall obtain approval from the Watershed Management Division, Plans and Specs Review Unit of an engineering report and/or plans and specifications that clearly show how the treatment system will meet the 1.0 mg/l chlorine residual and 20 minutes detention time required in the final permitted effluent limitations required on Page 9 of the permit prior to construction or January 1, 1995, whichever occurs first.

5. The permittee shall notify the Austin Office, Watershed Management Division, Enforcement Support Unit and the Region Office of the Texas Natural Resource Conservation Commission in writing at least forty-five (45) days prior to the completion of the chlorination facilities.

6. By April 1, 1995, the permittee shall submit to the Texas Natural Resource Conservation Commission, Municipal Permits Section, Watershed Management Division and the Region Office of the Texas Natural Resource Conservation Commission a study that investigates the possibility of substituting reclaimed water for potable water and/or freshwater where such substitution would be both appropriate and cost effective pursuant to Chapter 30 TAC Section 305.126(b). At a minimum, the study shall include:

   A. a water supply and demand assessment for the area served;
   B. an inventory of potential areas where reclaimed water may be appropriately substituted for potable water and/or freshwater;
   C. an inventory of potential uses of reclaimed water;
   D. an analysis of the market for reclaimed water and the conditions necessary to serve that market (eg. quantity, quality, selling price, distribution system); and
   E. a preliminary cost-benefit analysis for the treatment and use of reclaimed water compared with the continued use of potable water and/or freshwater, water supply augmentation, water conservation, and/or cost of treatment and disposal of treated wastewater.

   Forty-five (45) days prior to implementation of an approved Use of Reclaimed Water program, the permittee shall provide written notice to the Austin Office, Watershed Management Division, Enforcement Support Unit and Region Office of the Commission. The sampling and monitoring required under Chapter 30 TAC Section 310.10 to 310.13 shall be submitted by the 25th of each month.

7. The permittee shall operate the parallel peak flow treatment system in accordance with the following provisions:

   A. Influent to the wastewater treatment facility will be diverted to the peak flow clarifiers only when wet weather cause the influent flowrate to the treatment plant to exceed 6,250 gallons per minute (9 MGD).

B. The average discharge during any two-hour (2-hour peak) from the peak flow clarifiers shall not exceed 11,806 gpm (17 MGD). Subsequently, the total two-hour flow (2-hour peak) from the peak flow clarifiers and the wastewater treatment system shall not exceed 18,056 gpm (26 MGD).

C. When the peak flow clarifiers are treating influent due to wet weather, the combined effluent concentration shall meet all limitations on page 2 of the permit.

D. If the peak flow clarifiers are removed from service, these units shall be drained and the supernatant and sludge returned to the head of the treatment plant.

E. Provisions shall be made to allow for influent testing by grab or composite sampling at the head of the treatment plant for BOD_5 and TSS at the same frequency listed on page 2 of this permit.

F. A flow measurement device shall be installed and maintained for both the peak flow clarifier and wastewater treatment systems.

G. When raw influent is diverted directly to the peak flow clarifiers, the permittee shall monitor both the peak flow system effluent and the total combined effluent for BOD_5 and TSS by a 24-hour composite sample. The composite sample shall begin with one sample taken within 1/2 hour after starting to divert raw effluent to the peak flow clarifiers and end with one sample taken 1/2 hour before ceasing direct diversion to the peak flow clarifiers.

H. The peak flow clarifiers may be used as final clarifiers for the wastewater treatment system under the following conditions:

i. The peak flow clarifiers are preceded by the two existing 60 foot diameter final clarifiers.

ii. The sludge blanket in the peak flow clarifiers is maintained at a level of one (1) foot or less.

iii. Raw influent is not being diverted to the peak flow clarifiers.

I. Each time raw influent is diverted directly to the peak flow clarifiers, the permittee shall keep records which include the following information:

i. Date(s) of operation and length of time of diversion.

ii. Flow data during operation and total volume treated by both the peak flow and wastewater treatment systems.

iii. Composite or grab sample analysis results for BOD_5 and TSS for both peak flow system effluent and total combined effluent.

iv. Date and time when the peak flow clarifier is totally drained, as applicable.

v. The requirements found in Item 2 of page 2 of this permit are met for flows from the peak flow clarifiers and wastewater treatment system.

The above records shall be maintained on a monthly basis and be available at the plant site for inspection by authorized representative of the Commission for at least three years.

J. The existing final clarifiers shall be operated only as final clarifiers. Any change in the operational mode shall require prior approval by the Executive Director.

8. The permittee shall comply with the following sludge requirements:

A. The permittee is authorized to dispose of sludge at a co-disposal landfill or land application site permitted or registered by the Texas Natural Resource Conservation Commission.

B. The permittee shall use only those sewage sludge disposal practices that comply with the federal regulations for landfills and solid waste disposal established in 40 CFR Part 257 and 258 and in accordance with all the applicable rules of the Texas Natural Resource Conservation Commission.

C. The permittee shall handle and dispose of sewage sludge in accordance with all applicable state and federal regulations to protect public health and the environment from any reasonable anticipated adverse effects due to any toxic pollutants which may be present.

D. If an applicable "acceptable management practice" or numerical limitation for pollutants in sewage sludge promulgated under Section 405(d)(2) of the Clean Water Act is more stringent than the sludge pollutant limit or acceptable management practice in this permit, or controls a pollutant not listed in this permit, this permit may be modified or revoked and reissued to conform to the requirements promulgated under Section 405(d)(2). In accordance with 40 CFR 122.41, one year following promulgation of the technical sludge regulations (40 CFR 503), the facility must be in compliance with all requirements regardless of whether the permit is modified to incorporate these standards.

A3 - CITY OF GROVES NORTH WWTF

A3 - CITY OF GROVES NORTH WWTF

A. General

The existing treatment units are analyzed according to current TNRCC design criteria for secondary treatment to determine the ratable plant sizing. The analysis is based on assumed secondary treatment requirements.

The plant is presently permitted for 0.83 mgd design flow (maximum monthly average) at secondary standards (20 mg/l BOD_5 and TSS) with a DO requirement of 5 mg/l. The permitted two hour peak flow is 2000 gpm (equivalent to 2.88 mgd).

The Groves North treatment facility consist of a comminutor, bar screen, influent lift station, primary clarifier, trickling filter, final clarifier, chlorine contact, sludge digester, and sludge drying beds. Treatment units are as described in the following sections.

B. Preliminary Treatment.

1. Comminutor.

Required: *Some form of screening; where shredders are used, a backup unit or manually cleaned bar screen shall be provided.*

2. Influent Lift Station. Three self-priming pumps, each rated at 575 gpm at 35 ft. TDH and 20 ft. suction lift, for a firm capacity of 1150 gpm. One sludge pump rated at 75 gpm at 50 ft. TDH.

Required: *Firm pumping capacity (largest pump out of service) must be adequate to pump peak flow to destination; three or more pumps (or duplex pumps with automatic variable speed control) required.*

Analysis: Firm capacity = 1150 gpm = 1.656 mgd peak flow.

3. Primary Clarifier. 40 ft. diam. x 9 ft. side water depth, bottom slopes to center, 6 ft. diam. stilling well, mechanical sludge collection.

Required: *Maximum surface loading at Peak flow of 1800 gal./day/ft², and at Design flow of 1000 gal./day/ft². Side water depth must be at least 7 ft.*

Analysis: Allowable flow based on surface area:
Effective surface area of clarifier = $\pi(20^2 - 3^2) = 1228$ ft²

Allowable Peak flow = $(1228)(1800)/10^6 = 2.21$ mgd

Allowable Design flow = (1228)(1000)/10^6 = 1.23 mgd

Side water depth is adequate.

Primary clarifier is considered to remove 35% of raw influent BOD_5.

C. Secondary Treatment.

1. Trickling Filter. 100 ft. diam. x 6 ft. media depth, four 8" distributor arms, rock media, 7854 ft^2 surface area, recirculation pumps.

Required: *Typical design loadings for high rate rock media are 230-900 gpd/ft^2 hydraulic loading and 25-300 lb BOD/day/1000 ft^3 organic loading, and a BOD removal of 65-85%. The National Research Council formula may be used for calculation the efficiency of rock filters.*

Analysis: Gross media area = $\pi(50^2)$ = 7854 ft^2 {0.1803 acres}

Media volume = 7854 x 6 = 47,124 ft^3 {1.082 Acre-ft.}

Calculate loading rates based on 0.83 mgd permitted ADF:

Hydraulic Loading = 830,000 gpd / 7854 ft^2
= 106 gpd/ft^2

Organic Loading $= \dfrac{65\%(0.83 \text{ mgd} \times 8.345 \times 200 \text{ mg/l})}{47{,}124 \text{ ft}^3}$
= 19.1 lb BOD/day/1000 ft^3

Calculate the efficiency of the trickling filters based on the NRC formula.

$$E_1 = 1 / [1 \pm m(i)^n]$$

Where:

n	=	0.5
m	=	0.0085
i	=	W/VF
W	=	lb. BOD to first stage of filter
V	=	ac-ft of trickling filter media
F	=	recirculation factor

$F = [1 \pm R/I] / [1 \pm (1 - f)R/I]^2$

Where: R = rate of recirculation
I = rate of raw influent
f = weighing factor, generally taken as 0.9 for domestic sewage

$$F = \frac{1 + (2.16/0.83)}{[1 \pm (1 - 0.9)(2.16 / 0.83)]^2}$$

$$= 2.86$$

$$E_1 = \frac{1}{1 \pm 0.0085 \{900 / [(1.082)(2.859)]\}^{0.5}}$$

$$= 0.873 \text{ (or 87.3\%)}$$

Per the City of Groves 1981 Design Information, the BOD_5 Removal Efficiency of the trickling filter was listed as 84.65%. Since the removal efficiency calculated by the NRC formula exceeds the allowable, assume 84.65% efficiency is correct.

2. Final Clarifier. 40 ft. diam. x 6 ft. side water depth, bottom slopes to center.

Required: *Maximum surface loading at Peak flow of 1600 gal./day/ft^2, and at Design flow of 800 gal./day/ft^2. Side water depth must be at least 10 ft. for surface areas of 1250 ft^2 or more. Effective detention times (based on liquid volume above a 3 ft. sludge blanket) must be 1.1 hr. @ Peak flow and 2.2 hr. @ Design flow.*

Analysis: Allowable flow based on surface area:
Effective surface area of clarifier = $\pi(20^2) = 1257$ ft^2

Allowable Peak flow = $(1257)(1600)/10^6 = 2.01$ mgd
Allowable Design flow = $(1257)(800)/10^6 = 1.01$ mgd

Side water depth is not adequate.

Allowable flow based on detention time:
Detention time is based on effective surface area and side water depth less three ft. $(1257)(6 - 3) = 3771$ ft^3 x 7.48 gal./ft^3 = 28,207 gal.

Allowable Peak flow = 28,207 gal./(1.1 hr.)(1 day/24 hr.)
= 0.62 mgd

Allowable Design flow = 28,207 gal./(2.2 hr.)(day/24 hr.)
= 0.31 mgd

The flows based on detention govern, since they are less than the flows based on surface area.

D. Effluent Works.

1. Chlorine Contract Chamber. 112 ft. long x 5 ft. bottom width/14 ft. top width x 4.5 ft. deep, divided into two chambers by a center wall running the length of the chamber, total tank volume of 34,600 gallons (4626 ft^3). Chlorination equipment designed for 0 to 500 pounds per day of chlorine.

Required: *Detention time of 20 minutes @ peak flow.*

Analysis: Existing volume approximately 4,626 ft^3
4,626 ft^3/20 min. = 231 cfm = 3.9 cfs = 2.49 mgd

2. Flow Measurement. 90° V-notch weir.

E. Sludge Processing.

1. Digester. 60 ft. diam. x 12 ft. side water depth, 37,670 ft^3 of volume per 1981 plans.

Requirement: *Minimum solids retention time of 30 days required for unheated anaerobic digesters. 26.5 ft^3/lb BOD_5/day required.*

Analysis: Volume = 37,670 ft^3

Allowable BOD_5 = 37,670 ft^3 / 26.5 ft^3/lb BOD_5/day
= 1421 lb. BOD_5/day.

At an influent strength of 200 mg/l, this volume is sufficient for 0.85 mgd design flow.

2. Drying Beds. Total area of 14,304 ft^2.

F. Plant Capacity

		ADF	PEAK
1.	Influent Lift Station		1.656 mgd
2.	Primary Clarifier	1.23 mgd	2.21 mgd
3.	Final Clarifier	0.31 mgd	0.62 mgd
4.	Chlorine Contact Chamber		2.49 mgd
5.	Anaerobic Digesters	0.85 mgd	

PERMIT NO. 10094-02
(corresponds to
NPDES PERMIT NO. TX0024651)

This is a renewal of Permit
No. 10094-02, approved
September 24, 1985.

TEXAS WATER COMMISSION
Stephen F. Austin State Office Building
1700 N. Congress Ave.
Austin, Texas 78711

PERMIT TO DISPOSE OF WASTES
under provisions of Chapter 26
of the Texas Water Code

City of Groves

whose mailing address is

P.O. Box 846
Groves, Texas 77619

is authorized to treat and dispose of wastes from the North Wastewater Treatment Plant

located at the western corner of Georgia Avenue and Mockingbird Lane, approximately 1/2 mile northeast of the intersection of State Highway 347 and State Highway Spur 136 in Jefferson County, Texas

to Jefferson County Drainage District No. 7 Main A-3 Canal; thence to Main Canal; thence to the Main A Canal, the Alligator Bayou; thence to Taylor Bayou; thence to the Drainage District No. 7 Main Outfall Canal; thence into the Intracoastal Waterway in Segment No. 0703 of the Neches - Trinity Coastal Basin

only in accordance with effluent limitations, monitoring requirements and other conditions set forth herein, as well as the rules of the Texas Water Commission ("Commission"), the laws of the State of Texas, and other orders of the Commission. The issuance of this permit does not grant to the permittee the right to use private or public property for conveyance of wastewater along the herein described discharge route. This includes property belonging to but not limited to any individual, partnership, corporation or other entity. Neither does this permit authorize any invasion of personal rights nor any violation of federal, state, or local laws or regulations. It is the responsibility of the permittee to acquire property rights as may be necessary to use the herein described discharge route.

This permit and the authorization contained herein shall expire at midnight, five years after the date of Commission approval.

APPROVED, ISSUED AND EFFECTIVE this 22nd day of October 19 90.

ATTEST: Brenda W. Foster

For the Commission

10-22-95

FINAL EFFLUENT LIMITATIONS AND MONITORING REQUIREMENTS

Outfall Number 001

1. During the period beginning upon the date of issuance and lasting through the date of expiration, the permittee is authorized to discharge subject to the following effluent limitations:

   The daily average flow of effluent shall not exceed 0.83 million gallons per day (MGD); nor shall the average discharge during any two-hour period (2-hour peak) exceed 2,000 gallons per minute (gpm).

Effluent Characteristic	Discharge Limitations				Minimum Self-Monitoring Requirements	
	Daily Avg mg/l(lbs/day)	7-day Avg mg/l	Daily Max mg/l	Single Grab mg/l	Report Daily Avg. & Daily Max. Measurement Frequency	Sample Type
Flow, MGD	Report	N/A	Report	N/A	Continuous	Totalizing meter
Biochemical Oxygen Demand (5-day)	20(138)	30	45	65	One/week	Composite
Total Suspended Solids	20(138)	30	45	65	One/week	Composite

2. The effluent shall contain a chlorine residual of at least 1.0 mg/l and shall not exceed a chlorine residual of 4.0 mg/l after a detention time of at least 20 minutes (based on peak flow), and shall be monitored daily by grab sample. An equivalent method of disinfection may be substituted only with prior approval of the Commission.

3. The pH shall not be less than 6.0 standard units nor greater than 9.0 standard units and shall be monitored once per week by grab sample.

4. There shall be no discharge of floating solids or visible foam in other than trace amounts and no discharge of visible oil.

5. Effluent monitoring samples shall be taken at the following location(s): Following the final treatment unit.

6. The effluent shall contain a minimum dissolved oxygen of 5.0 mg/l and shall be monitored once per week by grab sample.

TEXAS NATURAL RESOURCE CONSERVATION COMMISSION

Protecting Texas by Reducing and Preventing Pollution

June 14, 1995

The Honorable Sylvester Moore, Mayor
City of Groves
P.O. Box 846
Groves, Texas 77619

Re: City of Groves - Renewal of Permit No. 10094-002

Dear Mayor Moore:

Attached for your review and comment is a copy of a draft proposed permit for the above-referenced operation. This draft is subject to further staff review and modification; however, we believe it generally includes the terms and conditions that are appropriate to your discharge. Please read the entire draft carefully because the following changes have been proposed since the permit was last issued:

1. Please note, that according to the analysis using the QUAL-TX model, an effluent set of 5 mg/l $CBOD_5$, 12 mg/l TSS, 3 mg/l NH_3-N and 6 mg/l DO will not meet the dissolved oxygen criterion established by the TNRCC Standards Team. Therefore, no discharge of pollutants into waters in the State is authorized in the final phase of the draft permit. (The series of perennial canals has been classified according to TNRCC implementation procedures for the Texas Surface Water Quality Standards and 30 TAC Chapter 307.4(H) and (K) with presumed high aquatic life use with 5.0 mg/l dissolved oxygen);
2. Regarding the proposed effluent limitations the City may request a standards revision for the discharge stream. Information regarding the discharge stream classification may be submitted to Mr. Charles Bayer of the Research of Environmental Assessment Section of the Water Planning and Assessment Division;
3. The sludge language in the draft permit has been modified since the last permit issuance; and
4. The expiration date on Page 1 of the draft permit is in accordance with the newly adopted rules of basin schedules in 30 TAC Chapter 305.71.

If you have any comments or questions, please contact me at (512) 239-4545 within two weeks from the date of this letter.

Sincerely,

Zdenek Matl

Zdenek Matl
Municipal Team, Permitting Section (MC 148)
Watershed Management Division

ZM:sp

Attachment
cc: TNRCC Region 10

PERMIT NO. 10094-002
(corresponds to
NPDES PERMIT NO. TX0024651)

This is a renewal of Permit No. 10094-002, approved October 22, 1990.

TEXAS NATURAL RESOURCE CONSERVATION COMMISSION
P. O. Box 13087
Austin, Texas 78711-3087

PERMIT TO DISPOSE OF WASTES
under provisions of Chapter 26
of the Texas Water Code

DRAFT
SUBJECT TO REVISION

City of Groves

whose mailing address is

P.O. Box 846
Groves, Texas 77619

is authorized to treat and dispose of wastes from the North Wastewater Treatment Facilities

located at the western corner of Georgia Avenue and Mockingbird Lane, approximately 0.5 mile northeast of the intersection of State Highway 347 and State Highway Spur 136 in Jefferson County, Texas

to ditch A-3A; thence into a series of perennial canals; thence into the Intracoastal Waterway in Segment No. 0702 of the Neches-Trinity Coastal Basin

only in accordance with effluent limitations, monitoring requirements and other conditions set forth herein, as well as the rules of the Texas Natural Resource Conservation Commission ("Commission"), the laws of the State of Texas, and other orders of the Commission. The issuance of this permit does not grant to the permittee the right to use private or public property for conveyance of wastewater along the herein described discharge route. This includes property belonging to but not limited to any individual, partnership, corporation or other entity. Neither does this permit authorize any invasion of personal rights nor any violation of federal, state, or local laws or regulations. It is the responsibility of the permittee to acquire property rights as may be necessary to use the herein described discharge route.

This permit and the authorization contained herein shall expire at midnight, October 1, 1998.

ISSUED DATE:

ATTEST:______________________________ ____________________________________
For the Commission

INTERIM EFFLUENT LIMITATIONS AND MONITORING REQUIREMENTS

Outfall Number 001

1. During the period beginning upon the date of issuance and lasting through September 30, 1998*, the permittee is authorized to discharge subject to the following effluent limitations:

   The daily average flow of effluent shall not exceed 0.83 million gallons per day (MGD); nor shall the average discharge during any two-hour period (2-hour peak) exceed 2,000 gallons per minute (gpm).

Effluent Characteristic	Discharge Limitations				Minimum Self-Monitoring Requirements	
	Daily Avg mg/l(lbs/day)	7-day Avg mg/l	Daily Max mg/l	Single Grab mg/l	Report Daily Avg. & Daily Max. Measurement Frequency	Sample Type
Flow, MGD	Report	N/A	Report	N/A	Five/week	Totalizing meter
Biochemical Oxygen Demand (5-day)	20(138)	30	45	65	One/week	Composite
Total Suspended Solids	20(138)	30	45	65	One/week	Composite

2. The effluent shall contain a chlorine residual of at least 1.0 mg/l and shall not exceed a chlorine residual of 4.0 mg/l after a detention time of at least 20 minutes (based on peak flow), and shall be monitored daily by grab sample. An equivalent method of disinfection may be substituted only with prior approval of the Commission.

3. The pH shall not be less than 6.0 standard units nor greater than 9.0 standard units and shall be monitored twice per month by grab sample.

4. There shall be no discharge of floating solids or visible foam in other than trace amounts and no discharge of visible oil.

5. Effluent monitoring samples shall be taken at the following location(s): Following the final treatment unit.

6. The effluent shall contain a minimum dissolved oxygen of 5.0 mg/l and shall be monitored once per week by grab sample.

* **See Other Requirement No. 1**

OTHER REQUIREMENTS

1. FINAL PHASE - During the period beginning upon October 1, 1998 and lasting through the date of expiration, no discharge of pollutants into waters in the State is authorized and the following provisions apply:

Conditions of the permit: No discharge of pollutants to surface water in the State is authorized.

Character: Treated Domestic Sewage Effluent

Volume: 30-day Average - 0.83 MGD from the treatment system

Quality: The following degree of treatment shall be required:

A.

Parameter	30-day Average	Effluent Concentrations (Not to Exceed) Single Grab
BOD_5, mg/l	20	65

The pH shall not be less than 6.0 standard units nor greater than 9.0 standard units.

The effluent shall be chlorinated in a chlorine contact chamber to a residual of 1.0 mg/l with a minimum detention time of 20 minutes.

B. Monitoring Requirements:

Parameter	Monitoring Frequency	Sample Type
Flow, MGD	Five/week	Instantaneous
BOD_5, mg/l	One/week	Composite
pH	Two/month	Grab
Chlorine, mg/l	Daily	Grab

The monitoring shall be done after the final treatment unit. These records shall be maintained on a monthly basis and be available at the plant site for inspection by authorized representatives of the Commission for at least three years.

2. This Category C facility shall be operated and maintained by a chief operator or operator in responsible charge holding a valid Class C certificate of competency or higher issued pursuant to 30 TAC Chapter 325. All shift supervisors and other plant operators shall be certified in accordance with the applicable provisions of Chapter 325. Note, Class D certificates are not renewable at any activated sludge facility, regardless of size, or any trickling filter or RBC facility with a permitted flow greater than 100,000 gallons per day.

3. The permittee shall submit within two years from the date of permit issuance an amendment application providing information about the no discharge facility to the Texas Natural Resource Conservation Commission, Permitting Section (MC 148), Watershed Management Division.

4. The permittee shall obtain approval from the Watershed Management Division, Permitting Section (MC 148)of an engineering report and/or plans and specifications that clearly show how the treatment system will meet the final permitted no discharge requirements required on Page 20 of the permit prior to construction or October 1, 1998, whichever occurs first.

A4 - CITY OF GROVES SOUTH WWTF

A4 - CITY OF GROVES SOUTH WWTF

A. General

The existing treatment units are analyzed according to current TNRCC design criteria for secondary treatment to determine the ratable plant sizing. The analysis is based on assumed secondary treatment requirements.

The plant is presently permitted for 2.29 mgd design flow (maximum monthly average) at secondary standards (20 mg/l BOD_5 and TSS) with a DO requirement of 5 mg/l. The permitted two hour peak flow is 4771 gpm (equivalent to 6.87 mgd).

The Groves South treatment facility consist of bar screens, preaeration units, primary clarifier, trickling filters, final clarifier, chlorination, dechlorination, anaerobic sludge digesters, and sludge drying beds. Treatment units are as described in the following sections.

B. Preliminary Treatment.

1. Bar Screens. Two fixed bar screens, 3 ft. side channels and screens, 1/2" bars with 1" openings between bars, screens at 45°, 3 ft. deep channels.

   Required: *Some form of screening; bar openings minimum ½" for mechanical screens; velocities @ design flow minimum 2 ft./sec through channel, < 3 ft./sec. through screen.*

2. Preaeration. Two basins, 22 ft. x 22 ft. x 13.5 ft. SWD, hopper bottoms.

3. Primary Clarifier. 60 ft. diam. x 10 ft. side water depth, bottom slopes to center, 9 ft. diam. stilling well, mechanical sludge collection.

   Required: *Maximum surface loading at Peak flow of 1800 gal./day/ft², and at Design flow of 1000 gal./day/ft². Side water depth must be at least 7 ft.*

   Analysis: Allowable flow based on surface area:
   Effective surface area of clarifier = $\pi(30^2 - 4.5^2) = 2764$ ft²

   Allowable Peak flow = $(2764)(1800)/10^6 = 4.98$ mgd
   Allowable Design flow = $(2764)(1000)/10^6 = 2.76$ mgd

   Side water depth is adequate.

   Primary clarifier is considered to remove 35% of raw influent BOD_5.

C. Secondary Treatment.

1. Trickling Filters. 3 @ 60 ft. diam. x 5.5 ft. media depth.

Required: *Typical design loadings for high rate rock media are 230-900 gpd/ft^2 hydraulic loading and 25-300 lb BOD/day/1000 ft^3 organic loading, and a BOD removal of 65-85%. The National Research Council formula may be used for calculating the efficiency of rock filters.*

Analysis: Gross media area = $(3)(\pi)(30^2) = 8482$ ft^2 {0.195 acres}

Media volume = 8482 x 5.5 = 46,651 ft^3 {1.071 Acre-ft.}

Calculate loading rates based on 2.29 mgd permitted ADF:

Hydraulic Loading = 2,290,000 gpd / 8482 ft^2
= 270 gpd/ft^2

Organic Loading $= \dfrac{65\%(2.29 \text{ mgd} \times 8.345 \times 200 \text{ mg/l})}{46{,}651 \text{ ft}^3}$
= 53.3 lb BOD/day/1000 ft^3

Calculate the efficiency of the trickling filters based on the NRC formula.

$$E_1 = 1 / [1 \pm m(i)^n]$$

Where:

n	=	0.5
m	=	0.0085
i	=	W/VF
W	=	lb. BOD to first stage of filter
V	=	ac-ft of trickling filter media
F	=	recirculation factor

$$F = [1 \pm R/I] / [1 \pm (1 - f)R/I]^2$$

Where:

R	=	rate of recirculation
I	=	rate of raw influent
f	=	weighing factor, generally taken as 0.9 for domestic sewage

$$F = \frac{1 + (2.0/2.29)}{[1 \pm (1 - 0.9)(2.0 / 2.29)]^2}$$

$$= \quad 1.58$$

$$E_1 = \frac{1}{1 \pm 0.0085\ \{2494 / [(1.071)(1.58)]\}^{0.5}}$$

$$= \quad 0.754 \text{ (or } 75.4\%)$$ *85% Required*

2. <u>Final Clarifier</u>. 60 ft. diam. x 8 ft. side water depth, bottom slopes to center, 9 ft. diam. stilling well, mechanical sludge collection.

Required: *Maximum surface loading at Peak flow of 1600 gal./day/ft², and at Design flow of 800 gal./day/ft². Side water depth must be at least 10 ft. for surface areas of 1250 ft² or more. Effective detention times (based on liquid volume above a 3 ft. sludge blanket) must be 1.1 hr. @ Peak flow and 2.2 hr. @ Design flow.*

Analysis: <u>Allowable flow based on surface area</u>:
Effective surface area of clarifier = $\pi(30^2) = 2827\ ft^2$

Allowable Peak flow = $(2827)(1600)/10^6 = 4.52$ mgd
Allowable Design flow = $(2827)(800)/10^6 = 2.26$ mgd

Side water depth is <u>not</u> adequate.

<u>Allowable flow based on detention time</u>:
Detention time is based on effective surface area and side water depth less three ft. $(2827)(8 - 3) = 14{,}135\ ft^3 \times 7.48\ gal./ft^3 = 105{,}730$ gal.

Allowable Peak flow = 105,730 gal./(1.1 hr.)(1 day/24 hr.)
= 2.31 mgd

Allowable Design flow = 105,730 gal./(2.2 hr.)(day/24 hr.)
= 1.15 mgd

The flows based on detention govern, since they are less than the flows based on surface area.

D. <u>Effluent Works</u>.

1. <u>Chlorine Contract Chamber</u>. 1,150 linear feet x 36" diameter effluent pipe @ 0.06% slope.

Required: *Detention time of 20 minutes @ peak flow.*

Analysis: Existing volume approximately 8,129 ft^3
8,129 ft^3/20 min. = 406 cfm = 6.8 cfs = 4.39 mgd

E. Sludge Processing.

1. Digester. Two 45 ft. diameter anaerobic digesters. Primary = 21.5 ft. SWD, Secondary = 19.5 ft. SWD.

Requirement: Minimum solids retention time of 30 days required for unheated anaerobic digesters. 26.5 ft^3/lb BOD_5/day required.

Analysis: Volume = 65,208 ft^3

Allowable BOD_5 = 65,208 ft^3 / 26.5 ft^3/lb BOD_5/day
= 2461 lb. BOD_5/day.

At an influent strength of 200 mg/l, this volume is sufficient for 1.47 mgd design flow.

2. Drying Beds. Total area of 26,400 ft^2.

F. Plant Capacity

		ADF	PEAK
1.	Primary Clarifier	2.76 mgd	4.98 mgd
2.	Final Clarifier	1.15 mgd	2.31 mgd
3.	Chlorine Contact Pipe		4.39 mgd
5.	Anaerobic Digesters	1.47 mgd	

PERMIT NO. 10094-001
(corresponds to
NPDES PERMIT NO. TX0024643)

This amendment supersedes and replaces Permit No. 10094-001 approved January 31, 1990.

TEXAS WATER COMMISSION
Stephen F. Austin State Office Building
1700 N. Congress Ave.
Austin, Texas 78711

PERMIT TO DISPOSE OF WASTES
under provisions of Chapter 26
of the Texas Water Code

City of Groves

whose mailing address is

P.O. Box 846
Groves, Texas 77619

is authorized to treat and dispose of wastes from the South Wastewater Treatment Facilities

located on Taft Avenue approximately 1 mile southeast of the intersection of Taft Avenue and State Highway 73 in Jefferson County, Texas

to the Sabine-Neches Canal in Segment No. 0703 of the Neches-Trinity Coastal Basin

only in accordance with effluent limitations, monitoring requirements and other conditions set forth herein, as well as the rules of the Texas Water Commission ("Commission"), the laws of the State of Texas, and other orders of the Commission. The issuance of this permit does not grant to the permittee the right to use private or public property for conveyance of wastewater along the herein described discharge route. This includes property belonging to but not limited to any individual, partnership, corporation or other entity. Neither does this permit authorize any invasion of personal rights nor any violation of federal, state, or local laws or regulations. It is the responsibility of the permittee to acquire property rights as may be necessary to use the herein described discharge route.

This permit and the authorization contained herein shall expire at midnight, January 31, 1995.

APPROVED, ISSUED AND EFFECTIVE this 9th day of March 19 92 .

ATTEST: Gloria A. Vazquez John Hall
For the Commission

S STP TWC permit 10094-01 expires: 1-31-95

INTERIM EFFLUENT LIMITATIONS AND MONITORING REQUIREMENTS

Outfall Number 001

1. During the period beginning upon the date of issuance and lasting through December 31, 1992, the permittee is authorized to discharge subject to the following effluent limitations:

   The daily average flow of effluent shall not exceed 2.29 million gallons per day (MGD); nor shall the average discharge during any two-hour period (2-hour peak) exceed 4771 gallons per minute (gpm).

Effluent Characteristic	Discharge Limitations				Minimum Self-Monitoring Requirements	
	Daily Avg mg/l(lbs/day)	7-day Avg mg/l	Daily Max mg/l	Single Grab mg/l	Report Daily Avg. & Daily Max. Measurement Frequency	Sample Type
Flow, MGD	Report	N/A	Report	N/A	Continuous	Totalizing meter
Biochemical Oxygen Demand (5-day)	20 (382)	30	45	65	Two/week	Composite
Total Suspended Solids	20 (382)	30	45	65	Two/week	Composite
Copper	Report(Report)	N/A	Report	N/A	Two/month	Composite

2. The effluent shall contain a chlorine residual of at least 1.0 mg/l after a detention time of at least 20 minutes (based on peak flow), and shall be monitored daily by grab sample. The permittee shall dechlorinate the chlorinated effluent to less than 0.1 mg/l chlorine residual and shall monitor daily by grab sample after the dechlorination process. An equivalent method of disinfection may be substituted only with prior approval of the Commission.

3. The pH shall not be less than 6.0 standard units nor greater than 9.0 standard units and shall be monitored once per week by grab sample.

4. There shall be no discharge of floating solids or visible foam in other than trace amounts and no discharge of visible oil.

5. Effluent monitoring samples shall be taken at the following location(s): Following the final treatment unit.

6. The effluent shall contain a minimum dissolved oxygen concentration of 2 mg/l and shall be monitored twice per week by grab sample.

FINAL EFFLUENT LIMITATIONS AND MONITORING REQUIREMENTS

Outfall Number 001

1. During the period beginning upon January 1, 1993 and lasting through the date of expiration, the permittee is authorized to discharge subject to the following effluent limitations:

The daily average flow of effluent shall not exceed 2.29 million gallons per day (MGD), nor shall the average discharge during any two-hour period (2-hour peak) exceed 4771 gallons per minute (gpm).

Effluent Characteristic	Discharge Limitations				Minimum Self-Monitoring Requirements	
	Daily Avg mg/l(lbs/day)	7-day Avg mg/l	Daily Max mg/l	Single Grab mg/l	Report Daily Avg. & Daily Max. Measurement Frequency	Sample Type
Flow	N/A	N/A	N/A	N/A	Continuous	Totalizing meter
Biochemical Oxygen Demand (5-day)	20 (382)	30	45	65	Two/week	Composite
Total Suspended Solids	20 (382)	30	45	65	Two/week	Composite
Copper	0.029(0.55)	N/A	0.061	N/A	Two/month	Composite

2. The effluent shall contain a chlorine residual of at least 1.0 mg/l after a detention time of at least 20 minutes (based on peak flow), and shall be monitored daily by grab sample. The permittee shall dechlorinate the chlorinated effluent to less than 0.1 mg/l chlorine residual and shall monitor daily by grab sample after the dechlorination process. An equivalent method of disinfection may be substituted only with prior approval of the Commission.

3. The pH shall not be less than 6.0 standard units nor greater than 9.0 standard units and shall be monitored once per week by grab sample.

4. There shall be no discharge of floating solids or visible foam in other than trace amounts and no discharge of visible oil.

5. Effluent monitoring samples shall be taken at the following location(s): following the final treatment unit.

6. The effluent shall contain a minimum dissolved oxygen concentration of 2 mg/l and shall be monitored twice per week by grab sample.

Shall report daily average and daily maximum limitation ← see Endorcement dated 1-7-93 attached to the back of this permit

John Hall, *Chairman*
Pam Reed, *Commissioner*
R. B. "Ralph" Marquez, *Commissioner*
Dan Pearson, *Executive Director*

TEXAS NATURAL RESOURCE CONSERVATION COMMISSION

Protecting Texas by Reducing and Preventing Pollution

June 8, 1995

The Honorable Sylvester Moore, Mayor
City of Groves
P.O. Box 846
Groves, Texas 77619-6048

Re: City of Groves - Amendment of Permit No. 10094-001

Dear Mayor Groves:

Attached for your review and comment is a copy of a draft proposed permit for the above-referenced operation. This draft is subject to further staff review and modification; however, we believe it generally includes the terms and conditions that are appropriate to your discharge. Please read the entire draft carefully because the following changes have been proposed since the permit was last issued:

1. Biomonitoring is required in the draft permit;
2. The sludge language in the draft permit has been modified since the last permit issuance;
3. According to the submitted information and a TNRCC evaluation, the limit of Copper has been deleted from the existing permit;
4. The expiration date on Page 1 of the draft permit is in accordance with the newly adopted rules of basin schedules in 30 TAC Chapter 305.71; and
5. This Category B facility shall be operated and maintained by a chief operator or operator in responsible charge holding a valid Class B certificate of competency or higher issued pursuant to 30 TAC Chapter 325.

If you have any comments or questions, please contact me at (512) 239-4545 within two weeks from the date of this letter.

Sincerely,

Zdenek Matl

Zdenek Matl
Municipal Team
Permitting Section (MC 148)
Watershed Management Division

ZM:sp

Attachment
cc: TNRCC Region 10

PERMIT NO. 10094-001
(corresponds to
NPDES PERMIT NO. TX0024643)

This permit supersedes and replaces Permit No. 10094-001, approved March 9, 1992.

TEXAS NATURAL RESOURCE CONSERVATION COMMISSION
P. O. Box 13087
Austin, Texas 78711-3087

PERMIT TO DISPOSE OF WASTES
under provisions of Chapter 26
of the Texas Water Code

DRAFT
SUBJECT TO REVISION

City of Groves

whose mailing address is

P.O. Box 846
Groves, Texas 77619-6048

is authorized to treat and dispose of wastes from the South Wastewater Treatment Facilities

located on Taft Avenue approximately 1 mile southeast of the intersection of Taft Avenue and State Highway 73 in Jefferson County, Texas

through a 36" pipe; thence under the Hurricane Protection Levee; thence to the Sabine-Neches Canal in Segment No. 0703 of the Neches-Trinity Coastal Basin

only in accordance with effluent limitations, monitoring requirements and other conditions set forth herein, as well as the rules of the Texas Natural Resource Conservation Commission ("Commission"), the laws of the State of Texas, and other orders of the Commission. The issuance of this permit does not grant to the permittee the right to use private or public property for conveyance of wastewater along the herein described discharge route. This includes property belonging to but not limited to any individual, partnership, corporation or other entity. Neither does this permit authorize any invasion of personal rights nor any violation of federal, state, or local laws or regulations. It is the responsibility of the permittee to acquire property rights as may be necessary to use the herein described discharge route.

This permit and the authorization contained herein shall expire at midnight, July 1, 1998.

ISSUED DATE:

ATTEST: ______________________

____
For the Commission

FINAL EFFLUENT LIMITATIONS AND MONITORING REQUIREMENTS

Outfall Number 001

1. During the period beginning upon the date of issuance and lasting through the date of expiration, the permittee is authorized to discharge subject to the following effluent limitations:

   The daily average flow of effluent shall not exceed 2.29 million gallons per day (MGD); nor shall the average discharge during any two-hour period (2-hour peak) exceed 4,771 gallons per minute (gpm).

Effluent Characteristic	Discharge Limitations				Minimum Self-Monitoring Requirements	
	Daily Avg mg/l(lbs/day)	7-day Avg mg/l	Daily Max mg/l	Single Grab mg/l	Report Daily Avg. & Daily Max. Measurement Frequency	Sample Type
Flow, MGD	Report	N/A	Report	N/A	Continuous	Totalizing meter
Biochemical Oxygen Demand (5-day)	20(382)	30	45	65	Two/week	Composite
Total Suspended Solids	20(382)	30	45	65	Two/week	Composite

2. The effluent shall contain a chlorine residual of at least 1.0 mg/l after a detention time of at least 20 minutes (based on peak flow), and shall be monitored daily by grab sample. The permittee shall dechlorinate the chlorinated effluent to less than 0.1 mg/l chlorine residual and shall monitor daily by grab sample after the dechlorination process. An equivalent method of disinfection may be substituted only with prior approval of the Commission.

3. The pH shall not be less than 6.0 standard units nor greater than 9.0 standard units and shall be monitored once per week by grab sample.

4. There shall be no discharge of floating solids or visible foam in other than trace amounts and no discharge of visible oil.

5. Effluent monitoring samples shall be taken at the following location(s): Following the final treatment unit.

6. The effluent shall contain a minimum dissolved oxygen of 2.0 mg/l and shall be monitored twice per week by grab sample.

B1 - City of Nederland

TABLE B-1
POPULATION PROJECTIONS
NEDERLAND, TEXAS (JEFFERSON COUNTY)

A	SETRPC Water Quality Management Plan - 1993				Texas Water Development Board Most Likely Series				Population					
YEAR	*B* Nederland	*C* City % Increase	*D* District % Increase	*E* Jefferson County WCID # 10	*F* Nederland	*G* City % Increase	*H* County % Increase	*I* Jefferson County	*J* City of Nederland Per TWDB	*K* District % Increase	*L* Jefferson County WCID No. 1	*M* Proposed Annexation Outside District	*N* Total City Population	*O* Total City Sewered Population
1950	3805							195,083						
1960	12,036							245,659						
1970	16,810							246,347						
1980	16,855				16,855			250,938						
1990	16,192				16,192			239,397						
1992	16,312			5000	16,370			243,251	16,370		5000			
1994	16,432	0.74	3.34	5167	16,549	1.09	1.58	247,104	16,549	4.92	5246	1406 (incl. 1101 sewered)	16,549	17,650
1995	16,492	0.37	1.61	5250	16,638	0.54	0.78	249,031	16,638	2.35	5369	1417 (incl. 1110 sewered)	16,638	17,748
2000	16,822	2.00	2.38	5375	17,084	2.68	3.87	258,665	17,084	3.19	5540	1472	24,096	18,556
2005	17,157	1.99	2.33	5500	17,123	0.23	1.81	263,340	17,123	0.27	5555	1499	24,177	18,622
2009	17,433	1.61	1.82	5600	17,154	0.18	1.42	267,079	17,154	0.20	5566	1520	24,240	18,674
2010	17,502	0.40	0.45	5625	17,162	0.05	0.35	268,014	17,162	0.06	5569	1525	24,256	18,687
2014					17,293	0.76	1.40	271,756	17,293		5611	1546	24,450	18,839
2020					17,489	1.13	2.07	277,368	17,489		5674	1578	24,741	19,067
2024					17,536	0.27	0.83	279,671	17,536		5689	1591	24,816	19,127
2030					17,606	0.40	1.24	283,125	17,606		5712	1611	24,929	19,217

TABLE B-1
POPULATION PROJECTIONS
NEDERLAND, TEXAS (JEFFERSON COUNTY)

A	SETRPC Water Quality Management Plan - 1993				Texas Water Development Board Most Likely Series				Population					
YEAR	B Nederland	C City % Increase	D District % Increase	E Jefferson County WCID # 10	F Nederland	G City % Increase	H County % Increase	I Jefferson County	J City of Nederland Per TWDB	K District % Increase	L Jefferson County WCID No. 1	M Proposed Annexation Outside District	N Total City Population	O Total City Sewered Population
1950	3805							195,083						
1960	12,036							245,659						
1970	16,810							246,347						
1980	16,855				16,855			250,938						
1990	16,192				16,192			239,397						
1992	16,312			5000	16,370			243,251	16,370		5000			
1994	16,432	0.74	3.34	5167	16,549	1.09	1.58	247,104	16,549	4.92	5246	1406 (incl. 1101 sewered)	16,549	17,650
1995	16,492	0.37	1.61	5250	16,638	0.54	0.78	249,031	16,638	2.35	5369	1417 (incl. 1110 sewered)	16,638	17,748
2000	16,822	2.00	2.38	5375	17,084	2.68	3.87	258,665	17,084	3.19	5540	1472	24,096	18,556
2005	17,157	1.99	2.33	5500	17,123	0.23	1.81	263,340	17,123	0.27	5555	1499	24,177	18,622
2009	17,433	1.61	1.82	5600	17,154	0.18	1.42	267,079	17,154	0.20	5566	1520	24,240	18,674
2010	17,502	0.40	0.45	5625	17,162	0.05	0.35	268,014	17,162	0.06	5569	1525	24,256	18,687
2014					17,293	0.76	1.40	271,756	17,293		5611	1546	24,450	18,839
2020					17,489	1.13	2.07	277,368	17,489		5674	1578	24,741	19,067
2024					17,536	0.27	0.83	279,671	17,536		5689	1591	24,816	19,127
2030					17,606	0.40	1.24	283,125	17,606		5712	1611	24,929	19,217

Population Projection Table
DF:628\C:\DOC\NED\4004-0\NEDPOP.TAB\Nederland
Regional Wastewater Study - TREATMENT PLANT ANALYSIS
062695

TABLE B-1 NOTES

1. The South East Texas Regional Planning Commission's Water Quality Management Plan, as updated in 1993, has for purposes of this report been superseded by the TWDB Most Likely Series (revised, in draft form, 1994). The revised projections are based on an increased inward migration rate for Southeast Texas as a result of recent employment growth.

2. Column B: Projections were provided for every five years through 2010, with other years interpolated.

3. Column E: Population was shown as (estimated) 5000 for 1992, with corresponding flow of 0.4 mgd. Populations for 1995, 2000, 2005, and 2010 were not shown directly in Plan, but were prorated on basis of flow projections in Plan.

4. Columns F and I: Projections were provided for every ten years through 2050. *(County population before 1980 is based on historic census figures.)* It is assumed for this report that the City projections do not reflect anticipated future annexations of Jefferson County WCID No. 10 and surrounding unincorporated areas.

5. Columns C, D, G, H, and K: Percent increase shown for each year is based on increase from the year on the row above.

6. Column L: Population for 1992 is taken at 5000 from SETRPC Plan. For subsequent years through 2010, population reflects a rate of increase at each stage equal to (Column D)(Column K)/(Column C). After that date, the rate of increase is taken to be equal to rate of City increase (Column G).

7. Column M: Total population for 1994 is based on 1994 house count.* Subsequent total population is taken as proportional to county population. For 1994 and 1995, sewered population is based on best available information on sewered areas. For 2000 and later, it is assumed that City sewer service will be extended to all houses in this area.

8. Columns N and O: Total City population is based on anticipated future annexation of District and surrounding areas. However, the District population is excluded from City sewered population since the City anticipates leaving the District's wastewater collection and treatment system intact. (For purposes of this table, annexation is assumed to occur between 1995 and 2000. For 1994 and 1995, only the sewered portion of the area outside District is included in sewered population. For 2000 and later, all annexed areas outside District are assumed to receive City sewer service.)

*See summary next page.

Table B-1 Notes (cont.)

Parkway Village (mobile home park):	187 active connections, management reports estimated 167 school children and 750 total population:	750
Other residential areas:	Total 230 customers, estimated 95% residential, assume 3 persons/residence:	656
	TOTAL	1406

Available information indicates that the following portions of the area outside the District are presently (1994) receiving sewer service (wholesale or retail) from the City:

Parkway Village (through mobile park owners):	750
Ridgecrest and Crestview subdivisions (area bounded by U. S. 69, Canal Avenue, 27th, and LNVA Canal; included in area outside District discussed above; 123 connections, assume 95% residential @ 3 persons/residence)	351
TOTAL	1101

FLOW PROJECTIONS
CITY OF NEDERLAND

Wastewater flows come primarily from the existing City, but a significant amount of flows come from two large areas outside the City. One of these areas is designated as the ETJ service area. This area is an area outside the City *(and also outside Jefferson County WCID No. 10)* which the City has indicated that it is likely to annex in the future. The area includes large amounts of vacant land, with most existing and projected development being residential. Portions of the ETJ service area already contribute waste flows to the City.

The other area is the Jefferson County Airport, which has no population.

Note that Jefferson County WCID No. 10 is excluded from City flow projections. Although the District may be annexed in the future along with the ETJ service area, the City anticipates leaving the existing District sewer system in service, with no effect on City flows.

Calculations are based mainly on population projections for the service area (Appendix B-1). As discussed above, this area includes the City and the ETJ service area. Approximately 70% of residents within the ETJ service area already receive City sewer service. It is assumed that the City will in the future annex the area and also extend sewer service to the remaining portions of the area.

A. Baseline Conditions (1994)

City sewered population 17,650 *(Including ETJ service area.)*

Annual average ADF (based on 24 month period ending August 1994) = 2.917 mgd *(Includes bypasses, based on quantities estimated and reported by City.)*

Maximum monthly ADF (May 1993) = 4.5 mgd *(Neglecting bypasses, since no reported bypasses occurred that month)*

Design ADF = (Maximum monthly ADF)(111%) *(Allowance for TNRCC 75/90 rule)*

Design ADF = 5.0 mgd *(4.5 mgd x 111%)*

Flow modelling of the interceptors leading to the treatment facility indicates that the maximum peak flow which could be presently transported to the treatment facility is 22.77 mgd. Therefore, the treatment facility should be designed with a two hour peaking factor of 5:1 (22.77 mgd:4.5 mgd).

Two hour peak (for design) should be **25.0 mgd**, based on 5 times the design ADF.

B. 15 Year Projections (2009)

City sewered population projected at 18,674. *(Including effects of extending service to the ETJ service area)*

Annual average ADF 2.995 mgd *(Based on following methodology:*

Assume that 37% of annual wastewater flows are comprised of storm water, as reported by City of Groves. Increase both residential and nonresidential return flows in proportion to population.

Increase storm flows at 15% the rate of residential growth, since future sewer extensions will be relatively watertight.)

Deduct 2% from future return flows because of water conservation measures.)

Existing 2.917 mgd = 63% return flows = 1.838 mgd

37% storm flows = 1.079 mgd

18,674 ÷ 17,650 = 1.0580 (5.8% increase)

1.838 mgd x 1.058 = 1.945 mgd

1.079 mgd x [1 + (.0580 x 0.15)] = 1.089 mgd

Deduct 2% of 1.945 mgd = (-) 0.039 mgd

NET TOTAL = 2.995 mgd

Maximum monthly ADF = 5.13 mgd *(Increasing in proportion to average annual ADF)*

5.0 mgd x (2.995 ÷ 2.917) = 5.13 mgd

Two hour peak (for design) should be **25.67 mgd**, based on 5 times the design ADF.

C. 30 Year Projections (2024)

City sewered population projected at 19,127. *(Including effects of extending service to the ETJ service area)*

Annual average ADF = 3.045 mgd *(Similar to 15 year projections)*

19,127 ÷ 17,650 = 1.0837 (8.37% increase)

1.838 mgd x 1.0837 = 1.992 mgd

1.079 mgd x [1 + (.0837 x 0.15)] = 1.093 mgd

Deduct 2% of 1.992 mgd = (-) 0.040 mgd

NET TOTAL = 3.045 mgd

Maximum monthly ADF = 5.22 mgd *(Increasing in proportion to average annual ADF)*

5.0 mgd x (3.045 ÷ 2.917) = 5.22 mgd

Two hour peak (for design) should be **26.10 mgd**, based on 5 times the design ADF.

B2 - City of Port Neches

TABLE B-2
POPULATION PROJECTIONS
PORT NECHES, TEXAS (JEFFERSON COUNTY)

A YEAR	*B* SETRPC Water Quality Management Plan - 1993	*C* Texas Water Development Board Most Likely Series	*D* Selected Population
1950	5488		5488
1960	8696		8696
1970	10,894		10,894
1980	13,944	13,944	13,944
1990	12,974	12,974	12,974
1992	13,114	13,227	13,227
1994	13,254	13,479	13,479
1995	13,324	13,606	13,606
2000	13,724	14,237	14,237
2005	14,124	14,392	14,392
2009	14,464	14,517	14,517
2010	14,549	14,548	14,548
2014		14,710	14,710
2020		14,953	14,953
2024		15,040	15,040
2030		15,171	15,171

TABLE B-2 NOTES

1. The South East Texas Regional Planning Commission's Water Quality Management Plan, as updated in 1993, has for purposes of this report been superseded by the TWDB Most Likely Series (recently adopted by Board action, January 1995). The revised projections are based on an increased inward migration rate for Southeast Texas as a result of recent employment growth.

2. Column B: Projections were provided for every five years through 2010, with other years interpolated.

3. Column C: Projections were provided for every ten years through 2050, with other years interpolated. It is assumed for this report that the City projections reflect no future annexations, and that there will in fact be no such annexations.

Table B-2 Notes (cont.)

4. Column D: Historical census figures (as quoted in SETRPC Plan) are used through 1990, then the TWDB projections (actual or interpolated) are used for all subsequent years. For purposes of this report, the City is assumed to serve all City residents and no residents outside the City.

FLOW PROJECTIONS
CITY OF PORT NECHES

Wastewater flows come from throughout the City. The sewered population is taken as equal to City population.

Calculations are based mainly on population projections for the City (Appendix B-2). It is assumed that all return flows will increase in proportion to the population (with slight adjustments for water conservation).

A. Baseline Conditions (1994)

City sewered population 13,479 *(Same as City population)*

Annual average ADF (based on 24 month period ending July 1994) *(Does not include any overflows, since the available monthly effluent reports do not show any overflows.)*

= 1.889 mgd main units

0.104 mgd storm water clarifiers

1.993 mgd TOTAL

Maximum monthly ADF = 3.03 mgd for main units (January 1993)

0.338 mgd for storm water clarifier (May 1994)

3.277 mgd for all units combined (June 1993)

Design ADF = (Maximum monthly ADF)(111%) *(Allowance for TNRCC 75/90 rule)*

Design ADF = 3.64 mgd *(3.277 mgd x 111%)*

Two hour peak: *An examination of monthly reports, including storm water clarifier usage, from September 1991 through July 1994, shows that the governing factor for peak 24 hr. flow is storm water clarifier usage. For each month that the storm water clarifiers were used as such, a summary report for that month was attached to the monthly reports showing various information for each date of usage. The flow-related data consisted of duration of usage, volume of storm water for that event, and total combined (24 hour) volume. A comparison with corresponding monthly operating reports indicates that the combined volume is the sum of (a) the 24 hr. flow through the main units, as shown on the monthly operating report for the following day, and (b) the flow diverted through the storm water clarifiers and not reflected on the monthly reporting forms.*

For calculation purposes, the storm flow is assumed to occur evenly throughout its duration as listed on the report, with the remainder of the combined flow passing through the main units at a constant rate throughout the day. The two hour peak would thus consist of the total of the two flow rates (storm flow plus other flow). The highest reliable value thus derived for the two hour peak occurred on June 13, 1994, as follows:

Duration of storm flow:	14 hr. 15 min.
Total storm flow:	5,615,000 gallons
Total daily combined flow:	11,871,000 gallons

5.615 mgd x 24/14.25 =	9.457 mgd
(11.571 - 5.615) mgd =	6.256 mgd
TOTAL	15.713 mgd two hour peak

To this peak historic plant flow should be added an amount for collection system overflows, which are known to be a serious problem in the Lee-Block neighborhood and which occur concurrently with the flows which activate the storm water clarifiers. The best available estimate of these flows is a previous engineering study which implied an overflow magnitude of 2.3 mgd.

Combined plant flow:	15.713 mgd
Manhole overflow:	2.3 mgd
TOTAL	18.013 mgd two hour peak[a]

[a] A higher value of 24.845 mgd was calculated similarly for June 20, 1992, but was considered unreliable because of limited transportation capacity as discussed below.

Peak reported flows may be unreliable because of two factors:

- *Observations by the City since completion of storm water clarifiers and related collection system work suggest that no more than 19 mgd can get to the plant because of limited gravity interceptor capacity upstream from the Park Lift Station. Overflows occur upstream from the gravity line, apparently because of inadequate line depth.*

- *The effluent meter for the main units (from which the reported effluent flows from main units are derived) reportedly functions inaccurately when the receiving stream level is high. The backwater problem results from operating practices at the downstream pump station operated by Jefferson County Drainage District No. 7. The problem reportedly needs pump station upgrading to correct the problem, and the District has been seeking funding.*

The design two hour peaking factor should be 5.5:1 (18.013 mgd:3.277 mgd).

Two hour peak (for design) should be **20.02 mgd**, based on 5.5 times the design ADF.

B. <u>15 Year Projections (2009)</u>

City sewered population projected at 14,710 *(Same as City population)*

Annual average ADF = 2.091 mgd *(Based on following methodology:*

Assume that 37% of annual wastewater flows are comprised of storm water, as reported by City of Groves. Increase both residential and nonresidential return flows in proportion to population.

Increase storm flows at 15% the rate of residential growth, since future sewer extensions will be relatively watertight.)

Deduct 2% from future return flows because of water conservation measures.)

Existing 1.993 mgd = 63% return flows = 1.256 mgd
37% storm flows = 0.737 mgd

14,710 ÷ 13,479 = 1.0913 (9.13% increase)

1.256 mgd x 1.0913 = 1.371 mgd

0.737 mgd x [1 + (.0913 x 0.15)] = 0.747 mgd

Deduct 2% of 1.371 mgd = (-) 0.027 mgd

<u>NET TOTAL = 2.091 mgd</u>

Maximum monthly ADF = 3.82 mgd *(Increasing in proportion to average annual ADF)*

3.64 mgd x (2.091 ÷ 1.993) = 3.82 mgd

Two hour peak (for design) = **21.01 mgd**, based on 5.5 times the design ADF.

C. 30 Year Projections (2024)

City sewered population projected at 15,040 *(Same as City population)*

Annual average ADF = 2.123 mgd *(Similar to 15 year projections)*

15,040 ÷ 13,479 = 1.1158 (11.58% increase)

1.256 mgd x 1.1158 = 1.401 mgd

0.737 mgd x [1 + (.1158 x 0.15)] = 0.750 mgd

Deduct 2% of 1.401 mgd = (-) 0.028 mgd

NET TOTAL = 2.123 mgd

Maximum monthly ADF = 3.88 mgd *(Increasing in proportion to average annual ADF)*

3.64 x (2.123 ÷ 1.993) = 3.88 mgd

Two hour peak for design) should be **21.34 mgd**, based on 5.5 times the design ADF.

B3 - City of Groves

TABLE B-3
POPULATION PROJECTIONS
GROVES, TEXAS (JEFFERSON COUNTY)

A	Groves Population			*E*	Adjusted Sewered Population			
YEAR	*B* SETRPC Water Quality Management Plan - 1993	Texas Water Development Board Most Likely Series		Adjustments to Sewered Population [Fairlea (+)]	*F* Total	*G* Ratio North:Total	*H* North Plant	*I* South Plant
		C TWDB Draft (1994)	*D* Revised per 10/26/94 Conversation w/TWDB					
1960	17,304							
1970	18,067							
1980	17,090	17,090	17,090					
1990	16,745	16,513	16,744					
1992	16,825	16,623	16,856	700	17556	0.3333	5851	11,705
1994	16,906	16,733	16,967	600	17567	0.3352	5888	11,679
1995	16,946	16,788	17,023	600	17623	0.3352	5907	11,716
2000	17,149	17,063	17,302	600	17902	0.3352	6001	11,901
2005	17,355	17,112	17,351	600	17951	0.3351	6015	11,936
2009	17,521	17,151	17,391	600	17991	0.3351	6029	11,962
2010	17,563	17,161	17,401	600	18001	0.3351	6032	11,969
2014		17,296	17,538	600	18138	0.3351	6078	12,060
2020		17,498	17,743	600	18343	0.3351	6147	12,196
2024		17,549	17,794	600	18394	0.3351	6164	12,230
2030		17,625	17,872	600	18472	0.3351	6191	12,281

TABLE B-3 NOTES

1. The South East Texas Regional Planning Commission's Water Quality Management Plan, as updated in 1993, has for purposes of this report been superseded by the TWDB Most Likely Series (recently revised, in draft form, 1994). The revised projections are based on an increased inward migration rate for Southeast Texas as a result of recent employment growth. (See Note 3.)

2. Column B: Projections were provided for every five years through 2010, with other years interpolated.

3. Column C: Projections were provided for every ten years through 2050. These projections were furnished to the Engineer's staff in October 1994 as revised drafts pending approval. These projections represent increases from the projections furnished by the TWDB in July of 1994. The revisions reflect higher inward migration because of improved economic conditions and trends in the Southeast Texas area. *However, both the recent and earlier projections were based on a 1990 census figure of 16,513 for Groves. The U. S. Census Bureau in 1992 corrected the 1990 Groves population count with an slight increase, but the TWDB disregarded the revision in order to expedite the process of updating its projections.*

4. Column D: The corrected 1990 population for Groves is 16,744, a slight increase from the originally reported 16,513. In a conversation between the Engineer's staff and Jim Hull of the TWDB on October 26, 1994, Mr. Hull concurred that the projections should be increased in some manner to reflect the corrected 1990 census figure. The selected method of adjustment was to increase all projections across the board by a ratio of 16,744:16,513.

5. Columns E and F: Adjustments have been made to the City population to derive the total sewered population of the two plants. The adjustments reflect the fact that Groves receives wastewater flows from a portion of Port Arthur.. A negative growth factor *(from 1992 through 1994)* is used for future projections for Fairlea (a Port Arthur subdivision served by Groves), since the neighborhood is the subject of a partial buyout by the adjacent Fina refinery.

6. Columns G through I: The adjusted sewered population is divided between the North and South plants according to a ratio of approximately 1:2 (adjusted slightly through the study period according to disparate growth patterns within the two service areas).

   The SETRPC projections showed approximate sewered populations in 1992 of 5449 and 10,899 for the Groves North and South plants respectively. The resulting total sewered population came out slightly less than any version of the total City population shown in this table for 1990 or 1992. The sewered population for the Groves South Plant includes the Fairlea subdivision in Port Arthur. (The sewered populations were noted as approximate.)

   For these projections, the growth in City population within the two service areas is assumed to be distributed at a ratio of 1:2, with the South Plant further affected by declines in the Fairlea population.

FLOW PROJECTIONS
CITY OF GROVES - NORTH PLANT

Wastewater flows come from the northwestern *(compass western)* portion of the City. Little if any flows come from outside the City, since Fairlea *(in Port Arthur)* is served by the South Plant. Calculations are based mainly on population projections for the North Plant service area (Appendix B-3). It is assumed that all return flows will increase in proportion to the population (with slight adjustments for water conservation).

A. <u>Baseline Conditions (1994)</u>

City sewered population 17,567 including 5888 in North Plant service area.

Annual average ADF (based on 24 months from September 1992 through August 1994) = 0.713 mgd.

The ADF does not include any bypasses; only two months during this period have bypasses reported by the City, and they would have a negligible impact on the long-term ADF (less than 0.04 mgd for the highest month, or less than 0.004 mgd for a 24 month average).

Maximum monthly ADF (February 1992) = 1.166 mgd.

Design ADF = (Maximum monthly ADF)(111%) *(Allowance for TNRCC 75/90 rule)*

Flow modelling of the incoming interceptor indicates that the maximum flow could be as high as 3.805 mgd. Assuming that excessive I/I will be transported to the WWTF in the future, the maximum monthly ADF will be increased. Assuming an average daily flow of 3.805 mgd on the day of a rainfall event, the average monthly ADF for the three months with highest reported ADF (January 1992, February 1992, and January 1993) would be 1.755 mgd.

Design ADF = 1.95 mgd *(1.755 mgd x 111%)*

Two hour peak (based on flow recorder charts showing peak flows over a recent 12 month period; highest 2 hr. peak occurred 3-9-94) = 2.3 mgd

Two hour peak (for design) should be **5.85 mgd**, based on 3 times the design ADF per TNRCC requirements.

B. <u>15 Year Projections (2009)</u>

City sewered population projected at 17,791, including 6029 in North Plant service area.

Annual average ADF = 0.716 mgd *(Based on following methodology:*

37% of annual wastewater flows are comprised of storm water as reported by City on TWDB water conservation forms. Increase both residential and nonresidential return flows in proportion to population.

Increase storm flows at 15% the rate of residential growth, since any future sewer extensions will be relatively watertight.)

Deduct 2% from future return flows because of water conservation measures.)

Existing 0.713 mgd = 63% return flows = 0.449 mgd

37% storm flows = 0.264 mgd

6029 ÷ 5688 = 1.0239 (2.39% increase)

0.449 mgd x 1.0239 = 0.460 mgd

0.264 mgd x [1 + (.0239 x 0.15)] = 0.265 mgd

Deduct 2% of 0.449 mgd = (-) 0.009 mgd

NET TOTAL = 0.716 mgd

Maximum monthly ADF = 1.96 mgd *(Increasing in proportion to average annual ADF)*

1.95 mgd x (.716 ÷ .713) = 1.96 mgd

Two hour peak (for design) should be **5.88 mgd**, based on 3 times the maximum ADF per TNRCC requirements.

C. 30 Year Projections (2024)

City population projected at 18,194, including 6164 in North Plant service area.

Annual average ADF = 0.728 mgd *(Similar to 15 year projections)*

6164 ÷ 5888 = 1.0469 (4.69% increase)

0.449 mgd x 1.0469 = 0.470 mgd

0.264 mgd x [1 + (.0469 x 0.15)] = 0.266 mgd

Deduct 2% of 0.470 mgd = (-) 0.009 mgd

NET TOTAL = 0.727 mgd

Maximum monthly ADF = 1.99 mgd *(Increasing in proportion to average annual ADF)*

1.95 x (.727 ÷ .713) = 1.99 mgd

Two hour peak (for design) should be **5.97 mgd**, based on 3 times max. ADF per TNRCC requirements.

C4 - City of Groves South WWTF

FLOW PROJECTIONS
CITY OF GROVES - SOUTH PLANT

Wastewater flows come from the City exclusive of the northwestern *(compass western)* portion of the City. Some flows also come from Fairlea *(in Port Arthur)*. Calculations are based mainly on population projections for the South Plant service area (Appendix B-3). It is assumed that all return flows will increase in proportion to the population (with slight adjustments for water conservation).

A. Baseline Conditions (1994)

City sewered population 17,567 including 11,679 in South Plant service area.

Annual average ADF (based on 24 months from September 1992 through August 1994) = 1.285 mgd

The ADF does not include any bypasses; none were reported by the City for the South Plant for this period.

Maximum monthly ADF (January 1993) = 2.333 mgd.

Flow modelling of the interceptors served by the Taft Avenue lift station indicates that the maximum influent flow could be as high as 18.7 mgd. Based on information contained in the 1981 rehabilitation plans for the Taft Avenue lift station, it appears that the lift station has a firm capacity of 5500 gpm (7.92 mgd). Assuming that excessive I/I will be transported to the WWTF in the future, the maximum monthly ADF will be increased. Assuming an average daily flow of 7.92 mgd on the day of a rainfall event, the average monthly ADF for the three months with highest reported ADF (February 1992, January 1993, and May 1994) would be 2.936 mgd.

Design ADF = (Maximum monthly ADF)(111%) *(Allowance for TNRCC 75/90 rule)*

Design ADF = 3.26 mgd *(2.936 mgd x 111%)*

Two hour peak (based on flow recorder charts showing peak flows over a recent 12 month period; highest 2 hr. peak occurred 9-1-94) = 7.5 mgd

Two hour peak (for design) should be **18.7 mgd**, based on flow modelling of incoming interceptors.

B. 15 Year Projections (2009)

City sewered population projected at 17,991, including 11,962 in South Plant service area.

Annual average ADF = 1.29 mgd *(Based on following methodology:*

37% of annual wastewater flows are comprised of storm water as reported by City on TWDB water conservation forms. Increase both residential and nonresidential return flows in proportion to population.

Increase storm flows at 25% the rate of residential growth, since any future sewer extensions will be relatively watertight.)

Deduct 2% from future return flows because of water conservation measures.)

Existing 1.285 mgd = 63% return flows = 0.810 mgd

37% storm flows = 0.475 mgd

11,962 ÷ 11,679 = 1.0242 (2.42% increase)

0.81 mgd x 1.0242 = 0.830 mgd

0.475 mgd x [1 + (.0242 x 0.25)] = 0.478 mgd

Deduct 2% of 0.83 mgd = (-) 0.017 mgd

NET TOTAL = 1.29 1mgd

Maximum monthly ADF = 3.28 mgd *(Increasing in proportion to average annual ADF)*

3.26 mgd x (1.291 ÷ 1.285) = 3.28 mgd

Two hour peak (for design) should be **18.7** mgd

C. 30 Year Projections (2024)

City population projected at 18,394, including 12,230 in South Plant service area.

Annual average ADF = 1.311 mgd *(Similar to 15 year projections)*

12,230 ÷ 11,679 = 1.0472 (4.72% increase)

.81 mgd x 1.0472 = .848 mgd

0.475 mgd x [1 + (.0472 x 0.15)] = 0.481 mgd

Deduct 2% of 0.848 mgd = (-) 0.017 mgd

NET TOTAL = 1.312 mgd

Maximum monthly ADF = 3.33 mgd *(Increasing in proportion to average annual ADF)*

3.26 x (1.312 ÷ 1.285) = 3.33 mgd

DERIVATION OF DESIGN FLOWS.

The Engineers feel that the methods which we have used in Appendix B are adequate for the scope of a regional planning study as distinguished from a detailed engineering plan. In the case of this study, they also represent the most realistic approach given the prevailing physical conditions in the collection systems and the available data.

Industrial flows are inapplicable to all three of the cities. Although at least two of the cities (Nederland and Port Neches) provide wastewater service to nearby industries, this service is for domestic flows only, with any process wastewater treated by the industries or by others.

The flow calculations show a segregation of infiltration/inflow from return flows. The City of Groves reported that approximately 37% of its total annual plant flows were composed of I/I. This figure was apparently based on past engineering studies such as those performed for the Construction Grants Program in the late 1970's. This figure looks reasonable in comparison with similar figures for other communities in the area with significant I/I problems.

Since all of the cities are primarily residential in nature, it is reasonable to assume that return flows will increase in proportion to population. Infiltration/inflow, however, can be expected to show a lesser rate of increase, since future growth will be served either by existing collection lines or by new extensions which will be relatively watertight.

For deriving peak flow rates, I/I flows could be addressed in a number of ways. A typical method used by Schaumburg & Polk, Inc. in Construction Grants projects was to estimate the total potential flow from the individual leaking segments *(without regard to lack of transportation capacity)*. A program of selected rehabilitation, based on cost effective considerations, was then developed and the amount of residual I/I flows estimated.. This process required an extensive sewer survey, including manhole inspection, smoke testing, possible television inspection, and quantification, followed by a thorough analysis. All three cities went through that process not many years back in the Construction Grants Program. Like many communities, they carried out the recommended program of system rehabilitation, only to find it much less successful than predicted.

The experience from the Construction Grants Program indicates that in many cases, the collection system is subject to so many sources of I/I flows that it cannot transport them to the plant. This is especially true in flat coastal areas where the gravity lines are laid at a minimum slope with limited conveyance, and at the same time are subject to continual shifting of the expansive soils in which they are laid. To a large extent, elimination of the major leaks can simply make room for I/I flows from other points throughout the system. It appears that this is what happened to all three cities.

In the absence of a sewer survey which would be far beyond the scope of a regional planning study, the quantity of peak I/I cannot be readily estimated. An attempt was made in most cases to estimate total peak flows on the basis of plant flow records. However, this method would tend to underestimate potential flows because of deficient transportation capacity in the system. All three cities have serious problems with surcharging and system overflows, but no reliable data is available to quantify this problem.

In the experience of the Engineer, flows from systems overloaded with I/I problems are likely to be underestimated. There is a serious danger in expanding a plant and trunk lines to handle estimated flows, then finding that the facilities are still overloaded. In the absence of extensive flow monitoring, particularly under prevailing local conditions, it is not unreasonable to expect the collection systems to be loaded to capacity during peak storm conditions.

The use of a two year storm event (5.5") for I/I calculations, even if the flows could be readily determined, would result in serious underdesign in light of the periodic storm events which exceed that amount by a factor of two or more.

In the course of the Construction Grants Program, it was learned that communities can expect to achieve only a limited quantity of I/I reduction, and this quantity is difficult to predict. Experience also indicates that I/I is a recurring problem, and that even a continuing maintenance program will leave a substantial amount of I/I. Considering the extensive rehabilitative efforts which have already been made, and the limited success of these efforts, it is unreasonable to expect the problem to be reduced substantially through additional work short of total system replacement.

The only collection system replacement or rehabilitation specifically recommended in the report is in certain sections of Nederland. This work is for the purpose of eliminating overflow conditions and may not significantly reduce flows to the plant.

The flow projections in Appendix B include a reduction in per capita return flows from water conservation measures. However, it is unrealistic to expect any significant reduction in water usage under present circumstances in Southeast Texas.

<u>In summary, we feel that the methodology used in the report for flow calculations is appropriate.</u>

APPENDIX C - Wastewater Treatment Alternatives

N1 - City of Nederland:	Upgrade Existing WWTF Activated Sludge 5/5
N2 - City of Nederland	Upgrade Existing WWTF, Divert Discharge to the Neches River Activated Sludge 10/15
N3 - City of Nederland:	Operate Existing WWTF, Add Constructed Wetland, Divert Discharge to Rhodair Gully (or Johns Gully)
N4 - City of Nederland:	Utilize Existing Treatment Plant, Construct Wetland w/Star Enterprise
N5 - City of Nederland:	Abandon Existing WWTF, Construct Lagoon/Wetland Treatment System, Discharge into Rhodair Gully or Johns Gully
N6 - City of Nederland:	Upgrade Existing WWTF, 4.76 mgd. Divert Discharge to the Neches River

General Discussion

N1 - City of Nederland: **Upgrade Existing WWTF Activated Sludge 5/5**

Upgrade the existing the existing WWTF for continued discharge into the existing receiving stream at the following effluent limits.

ADF	=	5.22 mgd
2-Hour Peak Flow	=	18,125 gpm (26.10 mgd)
BOD_5	=	5 mg/l
TSS	=	5 mg/l
NH_3	=	2 mg/l
D.O.	=	6 mg/l

A. Preliminary Treatment (Before splitting into tracks)

1. Screening

Existing: One mechanical bar screen, 7 ft. 5 in. ± length, 5 ft. total width, 30 bars, 3/8 in. width on 1 3/8" centers, with mechanical cleaning mechanism; design liquid depth 6 ft. maximum.

Required: *Some form of screening; bar openings minimum ½" for mechanical screens; velocities @ design flow minimum 2 ft./sec through channel, < 3 ft./sec. through screen.*

Analysis: Bar openings = 1"

Design Flow = 5.22 mgd = 8.08 cfs

Channel Velocity = 8.08 cfs / (5 ft. x 6 ft.)
= 0.27 fps

Screen Velocity = 8.08 cfs / (30 x 1/12 ft. x 6 ft.)
= 0.54 fps

Improvements: **NONE**

2. Influent Lift Station.

Existing: Four pumps, submersible type, installed in dry pit, each 2900 gpm capacity for firm capacity of 8700 gpm. (Two of the pumps are two speed with a slower speed of 900 gpm, with pump speed automatically adjusted as a function of wet well level. The 2900 gpm rated capacities are based on an average pumping head between high and low wet well levels.)

Required: *Firm pumping capacity (largest pump out of service) must be adequate to pump peak flow to destination; three or more pumps (or duplex pumps with automatic variable speed control) required.*

Analysis: Existing firm capacity = 8700 gpm = 12.528 mgd peak flow. Proposed firm capacity = 18,125 gpm = 26.10 mgd peak flow.

Improvements: Upgrade pumping firm capacity of lift station to 18,125 gpm. Replace three of the existing pumps with 7613 gpm pumps.

3. Aerated Grit Chamber

Existing: 20 ft. x 20 ft. chamber, 13 ft. water depth (less 5 ft. x 12 ft. x 4.5 ft. splitter box for effluent), plus hopper bottom with 1:1 slope (reported basin volume of 6240 ft^3); two air diffusers (112 cfm total) with 30" draft tube; concentrated grit/liquid mixture sent to degritter for final grit separation.

Required: *Grit removal recommended; if removal units are provided, must have method of removing grit from unit, and any unit with single chamber must have bypass.*

Analysis: Grit removal by grit pump below; piping allows flow to bypass grit chamber if needed. This unit also provides preaeration.

Improvements: NONE

4. Grit Pump

Existing: One vortex type pump, 250 gpm (pumps grit/liquid mixture from aerated grit chamber to degritter).

Improvements: NONE

5. Degritter

Existing: Hydrocyclone (10.5 ft. long) and grit classifier/washer (L shaped, approx. 5 ft. x 25 ft. plus 4 ft. x 3 ft. (dewaters grit from aerated grit chamber).

Improvements: NONE

B. Activated Sludge Process. Construct new activated sludge aeration basins.

Required: *Total volume shall be 1000 ft^3 per 35 lb. BOD_5/day. Diffused aeration shall be designed for 3200 SCF per lb. BOD_5. The diffuser system must be capable of providing 150% of design requirements.*

Analysis: lbs BOD_5/day = (5.22 MGD)(8.345)(200 mg/l) = 8712 lbs BOD_5/day

MAX BOD_5 LOAD	= 30 lb. BOD_5/day/1000 ft^3 (Conservative loading based on Engineer's experience)
BOD_5 Loading	= (8,712 lb BOD_5/day)/(30 lb BOD_5/day/1000 ft^3 = 290,400 ft^3
Each Unit	= (290,400 ft^3)/2 = 145,200 ft^3 = (145,200 ft^3)/(22 ft SWD) = 6600 ft^2 = 81 ft x 81ft
Air Requirements	= (8,712 lb. BOD_5/day)(3200 SCFM/lb. BOD_5) = 27,878,400 SCFM/day = 19,360 cfm
150% of Air Req.	= 19,360 cfm (1.5) = 29,040 cfm

Improvements: **Construct two deep tank type aerators (81 ft x 81 ft x 22 ft SWD) and provide 29,040 cfm aeration equipment capacity.**

C. Final Clarifiers. Construct new final clarifiers.

Existing: Two (2) at 60 ft. diam. x 10 ft. side water depth, to be converted to aeration units.

Required: *Maximum surface loading at Peak flow of 1200 gal./day/ft^2, and at Design flow of 600 gal./day/ft^2. Side water depth must be at least 10 ft. for surface areas of 1250 ft^2 or more. Effective detention times (based on liquid volume above a 3 ft. sludge blanket) must be 1.5 hr. @ Peak flow and 3.0 hr. @ Design flow.*

Analysis: Required Area based on surface area:

@ Peak Flow	= 26,100,000 gpd/1200 gal/day/ft^2 = 21,750 ft^2
@ Design Flow	= 5,220,000 gpd/600 gal/day/ft^2 = 8,700 ft^2

Required Area based on detention time:
Detention time is based on side water depth of 14 ft less 3 ft. sludge blanket.

@ Peak flow = (26,100,000 gpd)(1.5 hrs.)/[(24 hrs./day)(7.48 gal/ft^3)(11 ft)] = 19,825 ft^2

@ Design flow = 5,220,000 gpd)(3.0 hr.)/[(24 hrs./day)(7.48 gal/ft^3)(11 ft)] = 7,930 ft^2

New clarifier(s) required based on surface area (and feedwell area):

Surface Area of 14′ DIA Feedwell = 154 ft^2

Surface Area Required = Surface Area @ Peak + Surface Area of Feedwell

Surface Area = 21,750 ft^2 + 154 ft^2 = 21,904 ft^2

Improvements: **Construct two (2) new 118 ft. diameter clarifiers with 14 ft. side water depth required (based on minimum required surface area). Provide flow splitting/collection structures/ piping, and sludge collection/pumping.**

D. Filtration. Construct a tertiary filter to reduce effluent TSS to required 5 mg/l.

Existing: NONE

Required: *Filtration must be employed as a unit operation to supplement suspended solids removal for those treatment facilities with tertiary effluent limits. Design filtration rates shall not exceed 3 gpm/ft^2 for single media filters, 4 gpm/ft^2 for dual media filters, and 5 gpm/ft^2 for mixed media filters. There shall be a minimum of two units and the required filter area shall be calculated with one unit out of service.*

Analysis: Assuming dual media filters = 18,125 gpm / 5 gpm/ft^2
= 3,625 ft^2

Assuming filtration is provided by three (3) 35 ft. x 35 ft. units with a fourth unit out of service. 3 (35' x 35') = 3,675 ft^2

Improvements: **Construct four (4) 35 ft. x 35 ft. mixed media tertiary filters.**

E. Effluent Works.

1. Chlorine Contact Chamber.

Existing: Inside dimensions 98 ft. 6 in. x 38 ft. 2 in. including partitions and baffles; minimum liquid depth 6.25 ft. (6 ft. in final compartment); hopper bottoms in two 18.5 ft. x 18.75 ft. portions of chamber; fine bubble diffusers for mixing.

Required: *Detention time of 20 minutes @ peak flow.*

Analysis: Existing volume approximately 23,000 ft^3
23,000 ft^3/20 min. = 1150 cfm = 19.2 cfs = 12.39 mgd

Additional volume required = 13.71 mgd @ 20 minutes
= 25,455 ft^3

Improvements: Construct a second, parallel chlorine contact chamber with an effective volume of 25,455 ft^3.

2. Chlorine Feed Equipment.

Existing: Two systems, each 500 lb./day feed capacity (vacuum operated) including one standby; flow proportioned; chlorine gas from one ton size containers.

Required: *Feed equipment must be able to provide more than the highest dosage to be required at any time. Dosage must be adequate to maintain a chlorine residual of at least 1 mg/l after 20 minutes detention, prior to dechlorination.*

Analysis: In standard practice, feed equipment is designed to feed 10 ppm of Cl_2 in order to assure a 1 mg/l residual.

(500 lb./day)/(10 ppm)(8.345 lb./gal.) = 5.99 mgd
If both feeders can be used simultaneously during peak flows, they would have a theoretical capacity for 11.98 mgd peak.

Improvements: **Provide additional chlorine feed equipment as necessary to provide for chlorination of 26.10 mgd.**

3. Dechlorination.

Existing: Liquid ammonium bisulfate, 3000 gal. storage tank, one metering pump with 96 gal./day capacity; injection and reaction occur in a transitional area between chlorination and flow measurement. This dechlorination area is structurally an extension of the chlorine contact chamber, 8 ft. x 10 ft. 8 in. rectangle plus an adjacent trapezoidal area, 5 ft. long, width transitional from 8 ft. to 3 ft. 11 in.

Required: *The effluent, after chlorination and 20 minutes detention time, must be dechlorinated to less than 0.1 mg/l. For most dechlorination agents, 1 minute detention is generally considered adequate.*

Analysis: 26,10 MGD = 40.38 cfs = 2,423 cfm
2,423 cfm (1 min.) = 2,423 ft^3

Improvements: **Construct a new 2,423 ft^3 dechlorination chamber downstream of the existing and proposed chlorine contact chambers.**

4. Flow Measurement.

Existing: 24 inch parshall flume; continuously indicating, recording, and totalizing flow meter calibrated to read up to 15 mgd.

Required: *Continuous effluent measurement required, with capacity for maximum expected peak flow.*

Analysis: Existing effluent measurement is not adequate for peak flows up to 26.10 mgd.

Improvements: **Construct a new parshall flume with continuous flow recorder capable of measuring up to 30 mgd.**

5. Postaeration.

Existing: Postaeration is accomplished by a cascading effect as the effluent drops from the flow measurement device to the effluent line.

Improvements: **Enhance existing passive aeration and/or provide mechanical postaeration as necessary to achieve required 6.0 mg/l effluent dissolved oxygen.**

F. Sludge Processing.

1. Sludge Thickener.

Existing: 38 ft. diam. x 14 ft. side water depth, bottom slopes to center @ 4:1; mechanical sludge collection with pickets; supernatant to Trickling

Filter No. 1.

Required: *Aerobic digesters should be provided with sludge thickening.*

Improvements: **Supernatant from thickener shall be diverted back to head of the plant.**

2. Aerobic Digesters. Convert the existing aeration units (contact and stabilization), the existing aerobic digester, and the clarifiers within the two (2) existing contract stabilization plants into aerobic digesters.

Requirement: *Minimum solids retention time of 15 days (may be calculated as 20 ft^3 for each lb. influent BOD_5 per day). Diffused air requirement is 30 cfm per 1000 ft^3 of volume.*

Analysis: Digester Volume Required = 20 ft^3 / lb.BOD_5 / day
lb. BOD_5/day = (5.22 mgd)(8.345)(200 mg/l) = 8712 lb. BOD_5/day

Required Digester Volume = (8712 lb. BOD_5/day)(20 ft^3/lb. BOD/day) = 174,244 ft^3

Existing Units: $[\pi(100\text{ ft.})^2/4](15\text{ ft.}) = 117{,}809\text{ ft}^3$ each
Total Volume $= 2 \times 117{,}809\text{ ft}^3 = 235{,}618\text{ ft}^3 > 174{,}244\text{ ft}^3$

Required Aeration = (30 cfm/1000 ft^3)(235,618 ft^3) = 7,069 cfm

Improvements: **Convert two (2) existing contact stabilization plants into aerobic digesters and provide 7,069 cfm aeration equipment capacity.**

3. Centrifuge Facility.

Existing: One sludge grinder; two sludge metering pumps, progressive cavity, 60 gpm; one polymer feed pump (for 6% solution); two 200 gallon polymer mixers; one polymer metering pump; one horizontal centrifuge, 60 gpm with 20 hp motor and mixing tank to introduce polymer into sludge.

Improvements: **Provide additional sludge dewatering facilities.**

4. Drying Beds.

Existing: Two sets of open sand beds, 76 ft. x 220 ft. and 50 x 100 ft.; used for standby only.

G. Blowers.

Existing: Four blowers, 1500 cfm each, existing firm capacity = 4500 cfm.

Required: Blowers must be able to meet maximum aeration requirements with largest unit out of service.

Improvements: Provide additional blowers as necessary.

H. Opinion of Probable Cost

1.	Upgrade influent lift station	$ 210,000
2.	Construct activated sludge aeration basins	$ 1,411,000
3.	Construct two additional final clarifiers	$ 1,662,000
4.	Construct tertiary filters	$ 2,192,000
5.	Additional chlorine contact/dechlorination facilities	$ 317,000
6.	Flow measurement/post-aeration	$ 75,000
7.	Convert existing contact stabilization units into aerobic sludge digesters	$ 125,000
8.	Additional sludge dewatering facilities	$ 440,000
9.	Additional aeration blower equipment	$ 740,000
10.	Yard piping improvements	$ 500,000
11.	Miscellaneous site work	$ 200,000
12.	Laboratory/Office	$ 75,000
13.	Electrical and instrumentation	$ 750,000
	Subtotal	$ 8,697,000
	Contingency (15%)	$ 1,305,000
	TOTAL OPINION OF PROBABLE CONSTRUCTION COST	**$ 10,002,000**

N2 - City of Nederland: Upgrade Existing WWTF, Divert Discharge to the Neches River Activated Sludge 10/15

Upgrade the existing the existing WWTF and divert the discharge to the Neches River at the following effluent limits.

ADF	=	5.22 mgd
2-Hour Peak Flow	=	18,125 gpm (26.10 mgd)
BOD_5	=	20 mg/l
TSS	=	20 mg/l
NH_3	=	no limit
D.O.	=	4 mg/l

A. Preliminary Treatment (Before splitting into tracks)

1. Screening

Existing: One mechanical bar screen, 7 ft. 5 in. ± length, 5 ft. total width, 30 bars, 3/8 in. width on 1 3/8" centers, with mechanical cleaning mechanism; design liquid depth 6 ft. maximum.

Required: Some form of screening; bar openings minimum ½" for mechanical screens; velocities @ design flow minimum 2 ft./sec through channel, < 3 ft./sec. through screen.

Analysis: Bar openings = 1"

Design Flow = 5.22 mgd = 8.08 cfs

Channel Velocity = 8.08 cfs / (5 ft. x 6 ft.)
= 0.27 fps

Screen Velocity = 8.08 cfs / (30 x 1/12 ft. x 6 ft.)
= 0.54 fps

Improvements: NONE

2. Influent Lift Station.

Existing: Four pumps, submersible type, installed in dry pit, each 2900 gpm capacity for firm capacity of 8700 gpm. (Two of the pumps are two speed with a slower speed of 900 gpm, with pump speed automatically adjusted as a function of wet well level. The 2900 gpm rated capacities are based on an average pumping head between high and low wet well levels.)

Required: *Firm pumping capacity (largest pump out of service) must be adequate to pump peak flow to destination; three or more pumps (or duplex pumps with automatic variable speed control) required.*

Analysis: Firm capacity = 8700 gpm = 12.528 mgd peak flow.

Improvements: **Upgrade pumping firm capacity of lift station to 18,125 gpm. Replace three of the existing pumps with 7613 gpm pumps.**

3. Aerated Grit Chamber

Existing: 20 ft. x 20 ft. chamber, 13 ft. water depth (less 5 ft. x 12 ft. x 4.5 ft. splitter box for effluent), plus hopper bottom with 1:1 slope (reported basin volume of 6240 ft^3); two air diffusers (112 cfm total) with 30" draft tube; concentrated grit/liquid mixture sent to degritter for final grit separation.

Required: *Grit removal recommended; if removal units are provided, must have method of removing grit from unit, and any unit with single chamber must have bypass.*

Analysis: Grit removal by grit pump below; piping allows flow to bypass grit chamber if needed. This unit also provides preaeration.

Improvements: **NONE**

4. Grit Pump

Existing: One vortex type pump, 250 gpm (pumps grit/liquid mixture from aerated grit chamber to degritter).

Improvements: **NONE**

5. Degritter

Existing: Hydrocyclone (10.5 ft. long) and grit classifier/washer (L shaped, approx. 5 ft. x 25 ft. plus 4 ft. x 3 ft. (dewaters grit from aerated grit chamber).

Improvements: **NONE**

B. Activated Sludge Process. Construct new activated sludge aeration units.

Required: *Total volume shall be 1000 ft^3 per 35 lb. BOD_5/day. Diffused aeration shall be designed for 3200 SCF per lb. BOD_5. The diffuser system must be capable of providing 150% of design requirements.*

Analysis: lbs. BOD_5/day = (5.22 mgd)(8.345)(200 mg/l) = 8712 lbs. BOD_5/day

Max BOD_5 Load	= 30 lb BOD_5/day/1000 ft^3 (Conservative loading)
BOD_5 Loading	= (8712 lb BOD_5/day)/(30 lb. BOD_5/day/1000 ft^3)
	= 290,400 ft^3
Each Unit	= 290,400 ft^3 / 2 = 145,200 ft^3
	= 145,200 ft^3/(22 ft SWD) = 6,600 ft^3 = 81 ft x 81 ft
Air Requirements	= (8,712 lb. BOD_5/day)(3,200 SCFM/lb. BOD_5)
	= 27,878,400 SCFM/day
	= 19,360 cfm
150% Air Req.	= (19,360 cfm)(1.5)
	= 29,040 cfm

Improvements: **Construct two deep tank type aerators (81 ft x 81 ft x 22 ft SWD) and provide 29,040 cfm aeration equipment capacity**

C. Final Clarifiers. Construct new final clarifiers.

Existing: Two (2) at 60 ft. diam. x 10 ft. side water depth, to be converted to digesters.

Required: *Maximum surface loading at Peak flow of 1200 gal./day/ft^2, and at Design flow of 600 gal./day/ft^2. Side water depth must be at least 10 ft. for surface areas of 1250 ft^2 or more. Effective detention times (based on liquid volume above a 3 ft. sludge blanket) must be 1.5 hr. @ Peak flow and 3.0 hr. @ Design flow.*

Analysis: Required Area based on surface area:

@ Peak Flow	= 26,100,000 gpd/1200 gal/day/ft^2 = 21,750 ft^2
@ Design Flow	= 5,220,000 gpd/600 gal/day/ft^2 = 8,700 ft^2

Required Area based on detention time:

@ Peak Flow = (26,100,000 gpd)(1.5 hrs.)/[(24 hrs./day)(7.48 gal/ft^3)(11 ft)]
= 19,825 ft^2

@ Design Flow = (5,200,000)(3.0 hrs)/[(24 hrs./day)(7.48 gal/ft^3)(11 ft)]
= 7,930 ft^2

New clarifier(s) required based on surface area (and feedwell area):

Surface Area of 14′ feedwell = 154 ft^2

Surface Area Required = Surface Area @ Peak + Surface Area of Feedwell

Surface Area Required = 21,750 + 154 = 21,904 ft^2

Improvements: **Construct two (2) new 118 ft. diameter clarifiers with 14 ft. side water depth required (based on minimum required surface area). Provide flow splitting/collection structures/piping, and sludge collection/pumping.**

D. Effluent Works.

1. Chlorine Contact Chamber.

Existing: Inside dimensions 98 ft. 6 in. x 38 ft. 2 in. including partitions and baffles; minimum liquid depth 6.25 ft. (6 ft. in final compartment); hopper bottoms in two 18.5 ft. x 18.75 ft. portions of chamber; fine bubble diffusers for mixing.

Required: *Detention time of 20 minutes @ peak flow.*

Analysis: Existing volume approximately 23,000 ft^3
23,000 ft^3/20 min. = 1150 cfm = 19.2 cfs = 12.39 mgd

Additional volume required = 13.71 mgd @ 20 minutes
= 25,455 ft^3

Improvements: **Construct a second, parallel chlorine contact chamber with an effective volume of 25,455 ft^3.**

2. Chlorine Feed Equipment.

Existing: Two systems, each 500 lb./day feed capacity (vacuum operated) including one standby; flow proportioned; chlorine gas from one ton size containers.

Required: *Feed equipment must be able to provide more than the highest* *dosage to be required at any time. Dosage must be adequate to maintain a chlorine residual of at least 1 mg/l after 20 minutes detention, prior to dechlorination.*

Analysis: In standard practice, feed equipment is designed to feed 10 ppm of Cl_2 in order to assure a 1 mg/l residual.

(500 lb./day)/(10 ppm)(8.345 lb./gal.) = 5.99 mgd
If both feeders can be used simultaneously during peak flows, they would have a theoretical capacity for 11.98 mgd peak.

Improvements: **Provide additional chlorine feed equipment as necessary to provide for chlorination of 26.10 mgd.**

3. Dechlorination.

Existing: Liquid ammonium bisulfate, 3000 gal. storage tank, one metering pump with 96 gal./day capacity; injection and reaction occur in a transitional area between chlorination and flow measurement. This dechlorination area is structurally an extension of the chlorine contact chamber, 8 ft. x 10 ft. 8 in. rectangle plus an adjacent trapezoidal area, 5 ft. long, width transitional from 8 ft. to 3 ft. 11 in.

Required: *The effluent, after chlorination and 20 minutes detention time, must be dechlorinated to less than 0.1 mg/l. For most dechlorination agents, 1 minute detention is generally considered adequate.*

Analysis: 26.10 mgd = 40.38 cfs = 2,423 cfm
2,423 cfm (1 min) = 2,423 ft^3

Improvements: **Construct a new 2,423 ft^3 dechlorination chamber downstream of the existing and proposed chlorine contact chambers.**

4. Flow Measurement.

Existing: 24 inch parshall flume; continuously indicating, recording, and totalizing flow meter calibrated to read up to 15 mgd.

Required: *Continuous effluent measurement required, with capacity for maximum expected peak flow.*

Analysis: Existing effluent measurement is not adequate for peak flows up to 26.10 mgd.

Improvements: **Construct a new parshall flume with a continuous flow recorder capable of measuring up to 30 mgd.**

5. Postaeration.

Existing: Postaeration is accomplished by a cascading effect as the effluent drops from the flow measurement device to the effluent line.

Improvements: **Enhance existing passive aeration and/or provide mechanical postaeration as necessary to achieve required 4.0 mg/l effluent dissolved oxygen.**

E. Effluent Lift Station. Construct an effluent lift station to pump the effluent flows from the existing WWTF to the Neches River.

Required: *Firm pumping capacity (largest pump out of service) must be adequate to pump peak flow to destination; three or more pumps (or duplex pumps with automatic variable speed control) required.*

Analysis: Firm capacity = 18,125 gpm

Five (5) pumps with firm capacity of 18,125 gpm with largest pump out of service.

Effluent force mains sized for a minimum of 2 fps at low flow and a maximum 10 fps at peak flow.

Install two (2) force mains, one for ADF and one as a Peak Flow force main. Proposed ADF force main = 18″ diameter and proposed peak flow force main = 20″ diameter.

Improvements: **Construct an effluent lift station with five (5) pumps (firm capacity of 18,125 gpm) and a dual 18″/30" diameter force main to the Neches River.**

F. Sludge Processing.

1. Sludge Thickener.

Existing: 38 ft. diam. x 14 ft. side water depth, bottom slopes to center @ 4:1; mechanical sludge collection with pickets; supernatant to Trickling Filter No. 1.

Required: *Aerobic digesters should be provided with sludge thickening.*

Improvements: **Supernatant from thickener shall be diverted back to head of the plant.**

2. Aerobic Digesters. (See N1)

Requirement: *Minimum solids retention time of 15 days (may be calculated as 20 ft^3 for each lb. influent BOD_5 per day). Diffused air requirement is 30 cfm per 1000 ft^3 of volume.*

Analysis: Digester Volume Required = 20 ft^3 / lb. BOD_5 per day
lb. BOD_5/day = (5.22 mgd)(8.345)(200 mg/l) = 8712 lb. BOD_5/day

Required Digester Volume = (8712 lb. BOD/day)(20 ft^3/lb. BOD/day) = 174,244 ft^3

Existing Units: = $[\pi(100 \text{ ft.})^2/4]$ (15 ft.) = 17,809 ft^3 each
Total Volume: = 2 x 117,809 ft^3 = 235,618 ft^3 > 174,244 ft^3

Required Aeration = (30 cfm/1000 ft^3)(235,618 ft^3) = 7069 cfm

Improvements: **Convert two (2) existing contact stabilization plants into aerobic digesters and provide 7,069 aeration equipment capacity.**

3. Centrifuge Facility.

Existing: One sludge grinder; two sludge metering pumps, progressive cavity, 60 gpm; one polymer feed pump (for 6% solution); two 200 gallon polymer mixers; one polymer metering pump; one horizontal centrifuge, 60 gpm with 20 hp motor and mixing tank to introduce polymer into sludge.

Improvements: **Provide additional sludge dewatering facilities.**

4. Drying Beds.

Existing: Two sets of open sand beds, 76 ft. x 220 ft. and 50 x 100 ft.; used for standby only.

G. Blowers.

Existing: Four blowers, 1500 cfm each, existing firm capacity = 4500 cfm.

Required: *Blowers must be able to meet maximum aeration requirements with largest unit out of service.*

Improvements: **Provide additional blowers as necessary.**

H. Opinion of Probable Cost

1.	Upgrade influent lift station	$	210,000
2.	Construct activated sludge aeration basins	$	1,411,000
3.	Construct two additional final clarifiers	$	1,662,000
4.	Additional chlorine contact/dechlorination facilities	$	317,000
5.	Flow measurement/post-aeration	$	75,000
6.	Construct effluent lift station/force mains	$	4,037,000
7.	Convert existing contact stabilization units into aerobic sludge digesters	$	125,000
8.	Additional sludge dewatering facilities	$	440,000
9.	Additional aeration blower equipment	$	740,000
10.	Yard piping improvements	$	500,000
11.	Miscellaneous site work	$	200,000
12.	Laboratory/Office	$	75,000
13.	Electrical and instrumentation	$	750,000
	Subtotal	$	10,102,000
	Contingency (15%)	$	1,515,000
	TOTAL OPINION OF PROBABLE CONSTRUCTION COST	$	11,617,000

In analyzing Alternate N2 it was assumed that the TNRCC would require that all flows be diverted to the Neches River. During wet weather the existing receiving stream may have adequate flow to receive a discharge at less stringent quality standards. Therefore, if the TNRCC would allow all flows up to the permitted ADF from each treatment facility to be diverted to the Neches River and allow peak flow in excess of the permitted ADF to continue to be discharged to the existing receiving stream at current 20/20 limits, then the cost of this alternative could be significantly reduced.

Opinion of Probable Cost

1.	Upgrade influent lift station	$	210,000
2.	Construct activated sludge aeration basins	$	1,411,000
3.	Construct two additional final clarifiers	$	1,662,000
4.	Additional chlorine contact/dechlorination facilities	$	317,000
5.	Flow measurement/post-aeration	$	75,000
6.	Construct effluent lift station/force mains	$	1,659,000
7.	Convert existing contact stabilization units into aerobic sludge digesters	$	125,000
8.	Additional sludge dewatering facilities	$	440,000
9.	Additional aeration blower equipment	$	740,000
10.	Yard piping improvements	$	500,000
11.	Miscellaneous site work	$	200,000
12.	Laboratory/Office	$	75,000
13.	Electrical and instrumentation	$	750,000
	Subtotal	$	7,724,000
	Contingency (15%)	$	1,159,000
	TOTAL OPINION OF PROBABLE CONSTRUCTION COST	$	**8,883,000**

N3 - City of Nederland: **Operate Existing WWTF, Add Constructed Wetland, Divert Discharge to Rhodair Gully**

Continue to operate existing WWTF, construct a transfer lift station to pump the WWTF effluent to a constructed surface flow wetland to polish the effluent from the existing WWTF and then discharge into Rhodair Gully (or Johns Gully) at the following effluent limits.

ADF	=	5.22 mgd
2-Hour Peak Flow	=	18,125 gpm (26.10 mgd)
BOD_5	=	10 mg/l
TSS	=	15 mg/l
NH_3	=	3 mg/l
D.O.	=	6 mg/l

A. Existing WWTF. Internal plant piping and hydraulic through existing WWTF will need to be analyzed and upgraded as necessary to provide for 26.10 mgd 2-Hour peak flow. Also, will require TNRCC approval to re-rate existing WWTF for a reduced efficiency at the higher flows. Assume existing WWTF will consistently produce a 60 mg/l BOD_5, 60 mg/l TSS and 20 mg/l NH_3 effluent at 5.22 mgd ADF.

Improvements: **Upgrade existing influent lift station as necessary to provide for 26.10 mgd peak flow (see Alternate N1).**

Upgrade existing chlorination/dechlorination system to provide for disinfection of 26.10 mgd (see Alternate N1).

B. Transfer Lift Station. Construct a lift station to transfer the effluent flows from the existing WWTF to the proposed constructed wetland.

Required: *Firm pumping capacity (largest pump out of service) must be adequate to pump peak flow to destination; three or more pumps (or duplex pumps with automatic variable speed control) required.*

Analysis: Firm capacity = 18,125 gpm

Five (5) pumps with firm capacity of 18,125 gpm with largest pump out of service.

Transfer force main sized for a minimum of 2 fps at low flow and a maximum 10 fps at peak flow.

Install two (2) force mains, one for ADF and one as a Peak Flow force main. Proposed ADF force main = 18″ diameter and proposed peak flow force main = 20″ diameter.

Improvements: **Construct a transfer lift station with five (5) pumps (firm capacity of 18,125 gpm) and a dual 18"/30" diameter transfer force main to the constructed wetland facility.**

C. Constructed Wetland Facility. Construct a surface flow wetland for polishing the effluent from the existing WWTF and discharge into Rhodair Gully (or Johns Gully).

Required: *Detention time required for a fraction BOD_5 remaining after secondary treatment of 0.30 (i.e. 60 mg/l / 200 mg/l) and a permitted BOD_5 of 10 mg/l is 11 days. In-situ or constructed clay liner or synthetic liner required. Wetland must be protected from a 100-year flood. Berms shall have 3H:1V sideslopes. Multiple cells required, multiple inlets/outlets required. Refer to section 317.15. Appendix G of the TNRCC Design Criteria for Sewerage Systems for additional requirements.*

Analysis: For NH_3 reduction a Marsh-Pond-Marsh configuration will be utilized. Marsh sections will encompass approximately 66% of total area at an average water depth of 8", and Pond section will have an average water depth of 36". Mean water depth across entire wetland will be approximately 17". Assume a porosity for wetland vegetation of 86%.

Per Jefferson County Soil Survey, Series 1960, No. 21, average monthly rainfall and evaporation rates are listed below.

	Average Rainfall	Average Evaporation	Net Contribution
January	4.34"	2.00"	2.34"
February	3.98"	2.32"	1.66"
March	3.25"	3.40"	- 0.15"
April	3.68"	4.27"	- 0.59"
May	4.47"	5.16"	- 0.69"
June	4.44"	5.49"	-1.05"
July	6.56"	5.48"	1.08"
August	5.32"	5.39"	- 0.07"
September	4.73"	4.41"	0.32"
October	3.19"	3.79"	- 0.60"
November	3.61"	2.65"	0.96"
December	5.11"	2.08"	3.03"

$$\frac{(5{,}220{,}000 \text{ gpd})(11 \text{ days})}{(7.48 \text{ gals./ft}^3)(17''/12\text{inch/ft.})(0.86)} = 6{,}300{,}797 \text{ ft}^2$$

$$= 145 \text{ acres}$$

Proposed wetland = 155 acres

Rainfall contribution $= \frac{(3.03'')(155 \text{ ac.})(43560 \text{ ft}^2/\text{ac.})(7.48 \text{ gal/ft}^3)}{(12 \text{ inches/ft.})(30 \text{ days/month})}$

= 425,071 gpd

Average flow through wetland = (Influent + Effluent) / 2
= (5,220,000 + 5,645,071) / 2
= 5,432,535 gpd

$$\frac{(155 \text{ ac.})(43560 \text{ ft}^2/\text{ac.})(7.48 \text{ gals./ft}^3)(17''/12 \text{ inch/ft.})(0.86)}{5{,}432{,}535 \text{ gpd}} = 11.3 \text{ days}$$

Determine loading rates based on 155 acres

BOD_5 = (5.22 mgd)(8.345)(60 mg/l) = $\frac{2614 \text{ lbs./day}}{155 \text{ acres}}$

= 16.9 lb./acre-day

TSS = 16.9 lb./acre-day

NH_3 = (5.22 mgd)(8.345)(20 mg/l) = $\frac{871 \text{ lbs./day}}{155 \text{ acres}}$

= 5.6 lb./acre-day

Improvements: **Construct a surface flow wetland with a total treatment area of 155 acres. Total area required for constructed wetlands, including perimeter easements, is approximately 200 acres.**

D. Effluent Works.

1. Post-aeration.

Improvements: **Construct a passive, cascade type aeration structure on the discharge from the constructed wetland capable of producing a 6 mg/l dissolved oxygen effluent.**

2. Flow Measurement.

Required: *Continuous effluent measurement required, with capacity for maximum expected peak flow.*

Analysis: Peak flow = 26.10 mgd + maximum rainfall.

Per 1960 Jefferson County Soils Survey report the wettest month is listed as May, 1946 with a total rainfall of 20.01 inches.

$$\text{Rainfall} = \frac{(20.01")(155\ \text{ac.})(43560\ \text{ft}^2/\text{ac.})(7.48\ \text{gal/ft}^3)}{(12\ \text{inches/ft.})(30\ \text{days/month})}$$

$$= 2{,}807{,}151\ \text{gpd}$$

$$\text{Peak flow} = 26.10\ \text{mgd} + 2.81\ \text{mgd}$$
$$= 28.91\ \text{mgd}$$

Improvements: **Construct a parshall flume flow measurement structure on the discharge from the constructed wetland capable of measuring flows up to 35 mgd.**

E. Opinion of Probable Cost

Proposed Wetland Site 'A' - Located between Highway 69,96,287 and West Port Arthur Road (SPUR 93), adjacent to and on the north side of Rhodair Gully. Availability questionable.

1.	Upgrade influent lift station	$	210,000
2.	Additional chlorine contact/dechlorination facilities	$	317,000
3.	Additional sludge dewatering facilities	$	220,000
4.	Yard piping improvements	$	300,000
5.	Electrical and instrumentation	$	235,000
6.	Laboratory/Office	$	75,000
7.	Construct transfer lift station/force main	$	3,297,000
8.	Constructed wetland system	$	3,000,000
9.	Effluent flow measurement/post-aeration	$	75,000
10.	Land Acquisition	$	400,000
	Subtotal	$	8,129,000
	Contingency (15%)	$	1,219,000
	TOTAL OPINION OF PROBABLE CONSTRUCTION COST	$	**9,348,000**

Proposed Wetland Site 'B' - Located west of West Port Arthur Road (SPUR 93), south of the proposed Federal Prison site, adjacent to Johns Gully. Assumed effluent limits of 10/15/3 for Johns Gully. Availability likely.

1.	Upgrade influent lift station	$	210,000
2.	Additional chlorine contact/dechlorination facilities	$	317,000
3.	Additional sludge dewatering facilities	$	220,000
4.	Yard piping improvements	$	300,000
5.	Electrical and instrumentation	$	235,000
6.	Laboratory/Office	$	75,000
7.	Construct transfer lift station/force main	$	5,617,000
8.	Constructed wetland system	$	3,000,000
9.	Effluent flow measurement/post-aeration	$	75,000
10.	Land Acquisition	$	200,000
	Subtotal	$	10,249,000
	Contingency (15%)	$	11,786,000
	TOTAL OPINION OF PROBABLE CONSTRUCTION COST	$	**11,786,000**

N4 - City of Nederland: **Utilize Existing Treatment Plant, Construct Wetland w/Star Enterprise**

Continue to operated existing WWTF, construct a transfer lift station to pump the WWTF effluent to a constructed surface flow wetland to polish the effluent from the existing WWTF and then discharge to STAR Enterprise for reuse at the following effluent limits.

	Nederland	STAR Enterprise
ADF	5.22 mgd	10.0 mgd
2-Hour Peak Flow	26.10 mgd	N/A

BOD_5	=	10 mg/l
TSS	=	15 mg/l
NH_3	=	3 mg/l
D.O.	=	6 mg/l

A. Existing WWTF. Internal plant piping and hydraulic through existing WWTF will need to be analyzed and upgraded as necessary to provide for 26.10 mgd 2-Hour peak flow. Also, will require TNRCC approval to re-rate existing WWTF for a reduced efficiency at the higher flows. Assume existing WWTF will consistently produce a 60 mg/l BOD_5, 60 mg/l TSS and 20 mg/l NH_3 effluent at 5.22 mgd ADF.

Improvements: **Upgrade existing influent lift station as necessary to provide for 26.10 mgd peak flow (see Alternate N1).**

Upgrade existing chlorination/dechlorination system to provide for disinfection of 26.10 mgd (see Alternate N1).

B. Transfer Lift Station. Construct a lift station to transfer the effluent flows from the existing WWTF to the proposed constructed wetland.

Required: *Firm pumping capacity (largest pump out of service) must be adequate to pump peak flow to destination; three or more pumps (or duplex pumps with automatic variable speed control) required.*

Analysis: Firm capacity = 18,125 gpm

Five (5) pumps with firm capacity of 18,125 gpm with largest pump out of service.

Transfer force main sized for a minimum of 2 fps at low flow and a maximum 10 fps at peak flow.

Install two (2) force mains, one for ADF and one as a Peak Flow force main.

Proposed ADF force main = 18″ diameter and proposed peak flow force main = 30″ diameter.

Improvements: **Construct a transfer lift station with five (5) pumps (firm capacity of 18,125 gpm) and a dual 18"/30″ diameter transfer force main to the constructed wetland facility.**

C. Constructed Wetland Facility. Construct a surface flow wetland for polishing the effluent from the existing WWTF and discharge into STAR Enterprise/LNVA Canal for industrial reuse at STAR Enterprise.

Required: *Detention time required for a fraction BOD_5 remaining after secondary treatment of 0.30 (i.e. 60 mg/l / 200 mg/l) and a permitted BOD_5 of 10 mg/l is 11 days. In-situ or constructed clay liner or synthetic liner required. Wetland must be protected from a 100-year flood. Berms shall have 3H:1V sideslopes. Multiple cells required, multiple inlets/outlets required. Refer to section 317.15. Appendix G of the TNRCC Design Criteria for Sewerage Systems for additional requirements.*

Analysis: For NH_3 reduction a Marsh-Pond-Marsh configuration will be utilized. Marsh sections will encompass approximately 66% of total area at an average water depth of 8", and Pond section will have an average water depth of 36". Mean water depth across entire wetland will be approximately 17". Assume a porosity for wetland vegetation of 86%.

Per Jefferson County Soil Survey, Series 1960, No. 21, average monthly rainfall and evaporation rates are listed below.

	Average Rainfall	Average Evaporation	Net Contribution
January	4.34"	2.00"	2.34"
February	3.98"	2.32"	1.66"
March	3.25"	3.40"	- 0.15"
April	3.68"	4.27"	- 0.59"
May	4.47"	5.16"	- 0.69"
June	4.44"	5.49"	-1.05"
July	6.56"	5.48"	1.08"
August	5.32"	5.39"	- 0.07"
September	4.73"	4.41"	0.32"
October	3.19"	3.79"	- 0.60"
November	3.61"	2.65"	0.96"
December	5.11"	2.08"	3.03"

$$\frac{(15{,}220{,}000 \text{ gpd})(11 \text{ days})}{(7.48 \text{ gals./ft}^3)(17''/12\text{inch/ft.})(0.86)} = 18{,}371{,}288 \text{ ft}^2$$

$$= 422 \text{ acres}$$

Proposed wetland = 450 acres

$$\text{Rainfall contribution} = \frac{(3.03'')(450 \text{ ac.})(43560 \text{ ft}^2/\text{ac.})(7.48 \text{ gal/ft}^3)}{(12 \text{ inches/ft.})(30 \text{ days/month})}$$

$$= 1{,}234{,}077 \text{ gpd}$$

Average flow through wetland = (Influent + Effluent) / 2
= (15,220,000 + 16,454,077) / 2
= 15,837,038 gpd

$$\frac{(450 \text{ ac.})(43560 \text{ ft}^2/\text{ac.})(7.48 \text{ gals./ft}^3)(17''/12 \text{ inch/ft.})(0.86)}{15{,}837{,}038 \text{ gpd}} = 11.3 \text{ days}$$

Determine loading rates based on 450 acres

$$BOD_5 = (15.22 \text{ mgd})(8.345)(60 \text{ mg/l}) = \frac{7621 \text{ lbs./day}}{450 \text{ acres}}$$

$$= 16.9 \text{ lb./acre-day}$$

TSS = 16.9 lb./acre-day

$$NH_3 = (15.22 \text{ mgd})(8.345)(20 \text{ mg/l}) = \frac{2540 \text{ lbs./day}}{450 \text{ acres}}$$

$$= 5.6 \text{ lb./acre-day}$$

Improvements: **Construct a surface flow wetland with a total treatment area of 450 acres. Total area required for constructed wetlands, including perimeter easements, is approximately 600 acres.**

D. Effluent Works.

1. Post-aeration.

Improvements: **Construct a passive, cascade type aeration structure on the discharge from the constructed wetland capable of producing a 6 mg/l dissolved oxygen effluent.**

2. Flow Measurement.

Required: *Continuous effluent measurement required, with capacity for maximum expected peak flow.*

Analysis: Peak flow = 26.10 mgd + maximum rainfall + $\text{Peak}_{\text{STAR}}$.

Per 1960 Jefferson County Soils Survey report the wettest month is listed as May, 1946 with a total rainfall of 20.01 inches.

$$\text{Rainfall} = \frac{(20.01")(450\text{ ac.})(43560\text{ ft}^2/\text{ac.})(7.48\text{ gal/ft}^3)}{(12\text{ inches/ft.})(30\text{ days/month})}$$

$$= 8{,}149{,}793\text{ gpd}$$

$$\text{Peak flow} = 26.10\text{ mgd} + 8.15\text{ mgd} + \text{Peak}_{\text{STAR}}$$
$$= 34.25\text{ mgd} + \text{Peak}_{\text{STAR}}$$

Improvements: **Construct a parshall flume flow measurement structure on the discharge from the constructed wetland capable of measuring flows up to 125% of total peak flow.**

3. Effluent Lift Station. Construct an effluent lift station to pump effluent flows to STAR Enterprise for reuse. Size lift station to pump 150% ADF flows. Force Main shall be as required by STAR Enterprise.

Required: *Firm pumping capacity (largest pump out of service) must be adequate to pump peak flow to destination; three or more pumps (or duplex pumps with automatic variable speed control) required.*

Analysis: Firm capacity = 15,854 gpm

Five (5) pumps with firm capacity of 15,854 gpm with largest pump out of service.

Transfer force main sized for a minimum of 2 fps at low flow and a maximum 10 fps at peak flow.

Improvements: **Construct an effluent lift station with five (5) pumps (firm capacity of 15,854 gpm) and a transfer force main as required.**

Calculate the efficiency of the trickling filters based on the NRC formula.

$$E_2 = 1 / [1 + (m / 1 - E_1)(i)^n]$$

Where:

n	=	0.5
m	=	0.0085
i	=	W_2/VF
W_2	=	lb. BOD to second-stage filter
V	=	ac-ft of trickling filter media
F	=	recirculation factor

$$F = [1 + R/I] / [1 + (1 - f)R/I]^2$$

Where:

R	=	rate of recirculation *(assume 2.43 per J&N Report)*
I	=	rate of raw influent
f	=	weighing factor, generally taken as 0.9 for domestic sewage

$$F = \frac{1 + (2.43)}{[1 + (1 - 0.9)(2.43)]^2}$$

$$= 2.22$$

$$E_1 = \frac{1}{1 + (0.0085 / 1 - 0.663) \{692 / [(0.68)(2.22)]\}^{0.5}}$$

$$= 0.649 \text{ (or } 64.9\ \%)$$

Effluent BOD_5 = (692 lb./day)(1 - 0.649)
= 242.9 lb./day

3. <u>Final Clarifiers</u>. Two, 60 ft. diam. x 10 ft. side water depth, total surface area of 5655 ft^2, bottoms slope to center.

Required: *Maximum surface loading at Peak flow of 1600 gal./day/ft^2, and at Design flow of 800 gal./day/ft^2. Side water depth must be at least 10 ft. for surface areas of 1250 ft^2 or more.*

Analysis: Surface area of each clarifier = $\pi(30^2) = 2827\ ft^2$
Total surface area of clarifiers = $2 \times \pi(30^2) = 5655\ ft^2$

Allowable Peak flow = $(5655)(1600)/10^6 = 9.05$ mgd

Allowable Design flow = $(5655)(800)/10^6 = 4.52$ mgd
Side water depth is adequate.

D. Stormwater Clarifiers. The stormwater clarifiers are operated as second stage final clarifiers during flows of 9 mgd or less, and will receive direct stormwater when flows exceed 9 mgd during storm events.

1. Stormwater Clarifiers. Two, 80 ft. x 12 ft. side water depth, total surface area of 10,053 ft^2, bottoms slope to center.

Required: *Maximum surface loading at Peak flow of 1600 gal./day/ft^2, and at Design flow of 800 gal./day/ft^2. Side water depth must be at least 10 ft. for surface areas of 1250 ft^2 or more.*

Analysis: Surface area of each clarifier = $\pi(40^2) = 5027$ ft^2
Total surface area of clarifiers = $2 \times \pi(40^2) = 10{,}053$ ft^2

Allowable Peak flow = $(10{,}053)(1600)/10^6 = 16.08$ mgd
Allowable Design flow = $(10{,}053)(800)/10^6 = 8.04$ mgd

Side water depth is adequate.

E. Effluent Works.

1. Chlorine Contract Chamber. Two chambers, total tank volume of 12,533 ft^3, average water depth of 6.3 ft.

Required: *Detention time of 20 minutes @ peak flow.*

Analysis: Existing volume 12,533 ft^3*
12,533 ft^3/20 min. = 627 cfm = 10.4 cfs = 6.75 mgd*
* Recently enlarged per permit requirements for 9 mgd.

2. Dechlorination.

Required: *The effluent, after chlorination and 20 minutes detention time, must be dechlorinated to less than 0.1 mg/l. For most dechlorination agents, 1 minute detention is generally considered adequate.*

3. Post-Aeration. Diffused aeration in a portion of chlorine chamber structure.

4. Flow Measurement.

Required: *Continuous effluent measurement required, with capacity for*

maximum expected peak flow.

F. Sludge Processing.

1. First Stage Digester. 36,320 ft^3, heating and mixing equipment.

Required: *Minimum solids retention time of 15 days required for unheated anaerobic digesters. 19.0 ft^3/lb BOD_5/day required.*

Analysis: Volume = 36,320 ft^3

Allowable BOD_5 = 36,320 ft^3 / 19.0 ft^3/lb BOD_5/day
= 1912 lb. BOD_5/day.

Per Texas Water Commission letter, dated September 28, 1989, the Port Neches WWTF is designed to treat 3157 lb. of BOD_5 per day. Therefore, at a design flow of 4.98 mgd this equals 76 mg/l BOD_5.

At 76 mg/l BOD_5, the first stage digester would be rated for a flow of 3.01 mgd.

2. Second Stage Digester. 33,120 ft^3.

Requirement: *Minimum solids retention time of 30 days required for unheated anaerobic digesters. 26.5 ft^3/lb BOD_5/day required.*

Analysis: Volume = 33,120 ft^3

Allowable BOD_5 = 33,120 ft^3 / 26.5 ft^3/lb BOD_5/day
= 1250 lb. BOD_5/day.

At 76 mg/l BOD_5, the second stage digester would be rated for a flow of 1.97 mgd.

3. Belt Filter Press.

G. Plant Capacity

		ADF	PEAK
1.	Primary Clarifier	5.66 mgd	10.18 mgd
2.	Final Clarifier	4.52 mgd	9.05 mgd
3.	Stormwater Clarifiers	8.04 mgd	16.08 mgd
4.	Chlorine Contact Chamber		6.75 mgd
5.	Anaerobic Digesters	4.98 mgd	

TEXAS NATURAL RESOURCE CONSERVATION COMMISSION

IN THE MATTER OF THE APPLICATION OF THE CITY OF PORT NECHES FOR RENEWAL Of PERMIT NO. 10477-004	§ § § §	BEFORE THE TEXAS NATURAL RESOURCE CONSERVATION COMMISSION

AN ORDER FOR A TEMPORARY VARIANCE FROM TEXAS SURFACE WATER QUALITY STANDARDS

On this the 11th day of May, 1994, the Texas Natural Resource Conservation Commission ("Commission" or "TNRCC"), at a hearing pursuant to notice properly and timely given, considered the application of the City of Port Neches, ("Applicant" or "Port Neches"), for an temporary variance pursuant to 30 Texas Administrative Code ("TAC") §307.2(d) (4).

Having heard the argument of the parties, the Commission is satisfied that the applicant has satisfied the requirements of 30 TAC §307.2(d)(4), therefore, the Commission finds that the temporary variance should be approved.

FINDINGS OF FACT

1. The TNRCC Water Quality Standards Team has determined that the criteria for the perennial Jefferson County Drainage District Canals in Segment No. 0702 of the Neches-Trinity Coastal Basin should be lowered to intermediate quality aquatic life uses. This change in criteria will require a revision to the Water Quality Standards and approval from EPA during the 1994 triennial revision of the standards.

2. The City of Port Neches's plant is an existing permitted discharge facility.

3. The City of Port Neches applied for a temporary variance during the permit renewal application process.

4. Notice of the temporary variance request was included in the public notice of the permit application.

5. The variance shall not exceed a time period of two years.

CONCLUSIONS OF LAW

1. The above facts are conditions sufficient to issue this order pursuant to 30 TAC §307.2(d)(4).

2. Issuance of this order will effectuate the purposes of Chapter 26 of the Texas Water Code.

NOW, THEREFORE, BE IT ORDERED BY THE TEXAS NATURAL RESOURCE CONSERVATION COMMISSION THAT:

1. The City of Port Neches is granted a temporary variance to existing water quality standards of the perennial Jefferson County Drainage District Canals in Segment No. 0702 of the Neches-Trinity coastal Basin.

2. The City of Port Neches will evaluate several options that would result in compliance with new effluent limitations. These options include 1) upgrading the treatment system to advanced levels, 2) rerouting the effluent to the Lower Neches River Tidal Segment 0601 and 3) joining a regional wastewater treatment system in the area.

3. If the Commission adopts the site specific standards for the perennial Jefferson County Drainage District Canals in Segment No. 0702, the City of Port Neches shall apply for a permit amendment to meet revised water quality standards.

4. If the Commission does not approve the site specific standard prior to the expiration of the variance period, then final effluent limits based on existing water quality standards shall remain in effect.

5. This temporary variance shall expire two years from the date of issuance of this Order.

6. The Chief Clerk of the Commission is directed to forward a copy of this Order to the Applicant and all other parties and to issue the Order and cause it to be recorded in the files of the Commission.

Issued this date: MAY 13 1994

TEXAS NATURAL RESOURCE CONSERVATION COMMISSION

John Hall

John Hall, Chairman

ATTEST:

Gloria A. Vasquez

Gloria Vasquez, Chief Clerk

C: LEON
Rudy

PERMIT NO. 10477-004
(corresponds to
NPDES PERMIT NO. TX0022926)

This is a renewal of Permit No. 10477-004, approved December 13, 1988.

TEXAS NATURAL RESOURCE CONSERVATION COMMISSION
Stephen F. Austin State Office Building
1700 N. Congress Ave.
Austin, Texas 78711

PERMIT TO DISPOSE OF WASTES
under provisions of Chapter 26
of the Texas Water Code

City of Port Neches

whose mailing address is

P.O. Box 758
Port Neches, Texas 77651

is authorized to treat and dispose of wastes from the Main Plant Wastewater Treatment Facilities

located approximately 1 mile northwest of the intersection of State Highway 347 and State Highway 73 in the 6100 block of Georgia Street in Jefferson County, Texas

to a concrete lined Jefferson County Drainage District No. 7 (DD7) drainage canal; thence to DD7 Canal A; thence to Alligator Bayou; thence to Taylor Bayou; thence to DD7 Main Outfall Canal; thence to the Intracoastal Waterway in Segment No. 0702 of the Neches-Trinity Coastal Basin

only in accordance with effluent limitations, monitoring requirements and other conditions set forth herein, as well as the rules of the Texas Natural Resource Conservation Commission ("Commission"), the laws of the State of Texas, and other orders of the Commission. The issuance of this permit does not grant to the permittee the right to use private or public property for conveyance of wastewater along the herein described discharge route. This includes property belonging to but not limited to any individual, partnership, corporation or other entity. Neither does this permit authorize any invasion of personal rights nor any violation of federal, state, or local laws or regulations. It is the responsibility of the permittee to acquire property rights as may be necessary to use the herein described discharge route.

This permit and the authorization contained herein shall expire at midnight, five years after the date of Commission approval.

ISSUED DATE: **MAY 13 1994**

ATTEST: Gloria A. Vasquez

John Hall
For the Commission

RECEIVED 6/8/94

INTERIM EFFLUENT LIMITATIONS AND MONITORING REQUIREMENTS

Outfall Number 001

1. During the period beginning upon the date of issuance and lasting through the April 30, 1997*, the permittee is authorized to discharge subject to the following effluent limitations:

   The daily average flow of effluent shall not exceed 4.98 million gallons per day (MGD); nor shall the average discharge during any two-hour period (2-hour peak) exceed 6,250 gallons per minute (gpm).

Effluent Characteristic	Discharge Limitations				Minimum Self-Monitoring Requirements	
	Daily Avg mg/l(lbs/day)	7-day Avg mg/l	Daily Max mg/l	Single Grab mg/l	Report Daily Avg. & Daily Max. Measurement Frequency	Sample Type
Flow, MGD	Report	N/A	Report	N/A	Continuous	Totalizing meter
Biochemical Oxygen Demand (5-day)	20(832)	30	45	65	One/day	Composite
Total Suspended Solids	20(832)	30	45	65	One/day	Composite

2. The effluent shall contain a chlorine residual of at least 1.0 mg/l after a detention time of at least 20 minutes (based on peak flow), and shall be monitored daily by grab sample. The permittee shall dechlorinate the chlorinated effluent to less than 0.1 mg/l chlorine residual and shall monitor daily by grab sample after the dechlorination process. An equivalent method of disinfection may be substituted only with prior approval of the Commission.

3. The pH shall not be less than 6.0 standard units nor greater than 9.0 standard units and shall be monitored once per week by grab sample.

4. There shall be no discharge of floating solids or visible foam in other than trace amounts and no discharge of visible oil.

5. Effluent monitoring samples shall be taken at the following location(s): Following the final treatment unit.

6. The effluent shall contain a minimum dissolved oxygen of 5.0 mg/l and shall be monitored once per day by grab sample.

* See Other Requirement No. 1, Page 9.

OTHER REQUIREMENTS

1. FINAL PHASE - During the period beginning upon May 1, 1997 and lasting through the date of expiration, no discharge of pollutants into waters in the State is authorized and the following provisions apply:

Conditions of the permit: No discharge of pollutants to surface water in the State is authorized.

Character: Treated Domestic Sewage Effluent

Volume: 30-day Average - 4.98 MGD from the treatment system

Quality: The following degree of treatment shall be required:

A.

Parameter	30-day Average	Effluent Concentrations (Not to Exceed) Single Grab
BOD_5, mg/l	20	65
TSS, mg/l	20	65

The pH shall not be less than 6.0 standard units nor greater than 9.0 standard units.

The effluent shall be chlorinated in a chlorine contact chamber to a residual of 1.0 mg/l with a minimum detention time of 20 minutes.

B. Monitoring Requirements:

Parameter	Monitoring Frequency	Sample Type
Flow, MGD	Five/week	Instantaneous
BOD_5, mg/l	One/week	Grab
pH	One/week	Grab
Chlorine, mg/l	Five/week	Grab

The monitoring shall be done after the final treatment unit. These records shall be maintained on a monthly basis and be available at the plant site for inspection by authorized representatives of the Commission for at least three years.

2. This Category B facility shall be operated and maintained by a chief operator or operator in responsible charge holding a valid Class B certificate of competency or higher issued pursuant to 30 TAC Chapter 325. All shift supervisors and other plant operators shall be certified in accordance with the provisions of the Chapter therein. Note, Class D certificates are not renewable at any activated sludge facility, regardless of size, or any trickling filter or RBC facility with a permitted flow greater than 100,000 gallons per day.

3. Prior to May 1, 1997, the permittee shall submit an amendment request detailing how the permittee will meet the requirements of Page 9.

4. The permittee shall obtain approval from the Watershed Management Division, Plans and Specs Review Unit of an engineering report and/or plans and specifications that clearly show how the treatment system will meet the 1.0 mg/l chlorine residual and 20 minutes detention time required in the final permitted effluent limitations required on Page 9 of the permit prior to construction or January 1, 1995, whichever occurs first.

5. The permittee shall notify the Austin Office, Watershed Management Division, Enforcement Support Unit and the Region Office of the Texas Natural Resource Conservation Commission in writing at least forty-five (45) days prior to the completion of the chlorination facilities.

6. By April 1, 1995, the permittee shall submit to the Texas Natural Resource Conservation Commission, Municipal Permits Section, Watershed Management Division and the Region Office of the Texas Natural Resource Conservation Commission a study that investigates the possibility of substituting reclaimed water for potable water and/or freshwater where such substitution would be both appropriate and cost effective pursuant to Chapter 30 TAC Section 305.126(b). At a minimum, the study shall include:

   A. a water supply and demand assessment for the area served;
   B. an inventory of potential areas where reclaimed water may be appropriately substituted for potable water and/or freshwater;
   C. an inventory of potential uses of reclaimed water;
   D. an analysis of the market for reclaimed water and the conditions necessary to serve that market (eg. quantity, quality, selling price, distribution system); and
   E. a preliminary cost-benefit analysis for the treatment and use of reclaimed water compared with the continued use of potable water and/or freshwater, water supply augmentation, water conservation, and/or cost of treatment and disposal of treated wastewater.

   Forty-five (45) days prior to implementation of an approved Use of Reclaimed Water program, the permittee shall provide written notice to the Austin Office, Watershed Management Division, Enforcement Support Unit and Region Office of the Commission. The sampling and monitoring required under Chapter 30 TAC Section 310.10 to 310.13 shall be submitted by the 25th of each month.

7. The permittee shall operate the parallel peak flow treatment system in accordance with the following provisions:

   A. Influent to the wastewater treatment facility will be diverted to the peak flow clarifiers only when wet weather cause the influent flowrate to the treatment plant to exceed 6,250 gallons per minute (9 MGD).

B. The average discharge during any two-hour (2-hour peak) from the peak flow clarifiers shall not exceed 11,806 gpm (17 MGD). Subsequently, the total two-hour flow (2-hour peak) from the peak flow clarifiers and the wastewater treatment system shall not exceed 18,056 gpm (26 MGD).

C. When the peak flow clarifiers are treating influent due to wet weather, the combined effluent concentration shall meet all limitations on page 2 of the permit.

D. If the peak flow clarifiers are removed from service, these units shall be drained and the supernatant and sludge returned to the head of the treatment plant.

E. Provisions shall be made to allow for influent testing by grab or composite sampling at the head of the treatment plant for BOD_5 and TSS at the same frequency listed on page 2 of this permit.

F. A flow measurement device shall be installed and maintained for both the peak flow clarifier and wastewater treatment systems.

G. When raw influent is diverted directly to the peak flow clarifiers, the permittee shall monitor both the peak flow system effluent and the total combined effluent for BOD_5 and TSS by a 24-hour composite sample. The composite sample shall begin with one sample taken within 1/2 hour after starting to divert raw effluent to the peak flow clarifiers and end with one sample taken 1/2 hour before ceasing direct diversion to the peak flow clarifiers.

H. The peak flow clarifiers may be used as final clarifiers for the wastewater treatment system under the following conditions:

i. The peak flow clarifiers are preceded by the two existing 60 foot diameter final clarifiers.

ii. The sludge blanket in the peak flow clarifiers is maintained at a level of one (1) foot or less.

iii. Raw influent is not being diverted to the peak flow clarifiers.

I. Each time raw influent is diverted directly to the peak flow clarifiers, the permittee shall keep records which include the following information:

i. Date(s) of operation and length of time of diversion.

ii. Flow data during operation and total volume treated by both the peak flow and wastewater treatment systems.

iii. Composite or grab sample analysis results for BOD_5 and TSS for both peak flow system effluent and total combined effluent.

iv. Date and time when the peak flow clarifier is totally drained, as applicable.

v. The requirements found in Item 2 of page 2 of this permit are met for flows from the peak flow clarifiers and wastewater treatment system.

The above records shall be maintained on a monthly basis and be available at the plant site for inspection by authorized representative of the Commission for at least three years.

J. The existing final clarifiers shall be operated only as final clarifiers. Any change in the operational mode shall require prior approval by the Executive Director.

8. The permittee shall comply with the following sludge requirements:

A. The permittee is authorized to dispose of sludge at a co-disposal landfill or land application site permitted or registered by the Texas Natural Resource Conservation Commission.

B. The permittee shall use only those sewage sludge disposal practices that comply with the federal regulations for landfills and solid waste disposal established in 40 CFR Part 257 and 258 and in accordance with all the applicable rules of the Texas Natural Resource Conservation Commission.

C. The permittee shall handle and dispose of sewage sludge in accordance with all applicable state and federal regulations to protect public health and the environment from any reasonable anticipated adverse effects due to any toxic pollutants which may be present.

D. If an applicable "acceptable management practice" or numerical limitation for pollutants in sewage sludge promulgated under Section 405(d)(2) of the Clean Water Act is more stringent than the sludge pollutant limit or acceptable management practice in this permit, or controls a pollutant not listed in this permit, this permit may be modified or revoked and reissued to conform to the requirements promulgated under Section 405(d)(2). In accordance with 40 CFR 122.41, one year following promulgation of the technical sludge regulations (40 CFR 503), the facility must be in compliance with all requirements regardless of whether the permit is modified to incorporate these standards.

A3 - CITY OF GROVES NORTH WWTF

A3 - CITY OF GROVES NORTH WWTF

A. General

The existing treatment units are analyzed according to current TNRCC design criteria for secondary treatment to determine the ratable plant sizing. The analysis is based on assumed secondary treatment requirements.

The plant is presently permitted for 0.83 mgd design flow (maximum monthly average) at secondary standards (20 mg/l BOD_5 and TSS) with a DO requirement of 5 mg/l. The permitted two hour peak flow is 2000 gpm (equivalent to 2.88 mgd).

The Groves North treatment facility consist of a comminutor, bar screen, influent lift station, primary clarifier, trickling filter, final clarifier, chlorine contact, sludge digester, and sludge drying beds. Treatment units are as described in the following sections.

B. Preliminary Treatment.

1. Comminutor.

   Required: *Some form of screening; where shredders are used, a backup unit or manually cleaned bar screen shall be provided.*

2. Influent Lift Station. Three self-priming pumps, each rated at 575 gpm at 35 ft. TDH and 20 ft. suction lift, for a firm capacity of 1150 gpm. One sludge pump rated at 75 gpm at 50 ft. TDH.

   Required: *Firm pumping capacity (largest pump out of service) must be adequate to pump peak flow to destination; three or more pumps (or duplex pumps with automatic variable speed control) required.*

   Analysis: Firm capacity = 1150 gpm = 1.656 mgd peak flow.

3. Primary Clarifier. 40 ft. diam. x 9 ft. side water depth, bottom slopes to center, 6 ft. diam. stilling well, mechanical sludge collection.

   Required: *Maximum surface loading at Peak flow of 1800 gal./day/ft^2, and at Design flow of 1000 gal./day/ft^2. Side water depth must be at least 7 ft.*

   Analysis: Allowable flow based on surface area:
   Effective surface area of clarifier = $\pi(20^2 - 3^2) = 1228$ ft^2

   Allowable Peak flow = $(1228)(1800)/10^6 = 2.21$ mgd

Allowable Design flow = (1228)(1000)/10^6 = 1.23 mgd

Side water depth is adequate.

Primary clarifier is considered to remove 35% of raw influent BOD_5.

C. Secondary Treatment.

1. Trickling Filter. 100 ft. diam. x 6 ft. media depth, four 8" distributor arms, rock media, 7854 ft^2 surface area, recirculation pumps.

Required: *Typical design loadings for high rate rock media are 230-900 gpd/ft^2 hydraulic loading and 25-300 lb BOD/day/1000 ft^3 organic loading, and a BOD removal of 65-85%. The National Research Council formula may be used for calculation the efficiency of rock filters.*

Analysis: Gross media area = $\pi(50^2)$ = 7854 ft^2 {0.1803 acres}

Media volume = 7854 x 6 = 47,124 ft^3 {1.082 Acre-ft.}

Calculate loading rates based on 0.83 mgd permitted ADF:

Hydraulic Loading = 830,000 gpd / 7854 ft^2
= 106 gpd/ft^2

Organic Loading $= \dfrac{65\%(0.83 \text{ mgd} \times 8.345 \times 200 \text{ mg/l})}{47{,}124 \text{ ft}^3}$
= 19.1 lb BOD/day/1000 ft^3

Calculate the efficiency of the trickling filters based on the NRC formula.

$$E_1 = 1 / [1 \pm m(i)^n]$$

Where:

n	=	0.5
m	=	0.0085
i	=	W/VF
W	=	lb. BOD to first stage of filter
V	=	ac-ft of trickling filter media
F	=	recirculation factor

$F = [1 \pm R/I] / [1 \pm (1 - f)R/I]^2$

Where: R = rate of recirculation
I = rate of raw influent
f = weighing factor, generally taken as 0.9 for domestic sewage

$$F = \frac{1 + (2.16/0.83)}{[1 \pm (1 - 0.9)(2.16 / 0.83)]^2}$$

$$= 2.86$$

$$E_1 = \frac{1}{1 \pm 0.0085 \{900 / [(1.082)(2.859)]\}^{0.5}}$$

$$= 0.873 \text{ (or 87.3\%)}$$

Per the City of Groves 1981 Design Information, the BOD_5 Removal Efficiency of the trickling filter was listed as 84.65%. Since the removal efficiency calculated by the NRC formula exceeds the allowable, assume 84.65% efficiency is correct.

2. Final Clarifier. 40 ft. diam. x 6 ft. side water depth, bottom slopes to center.

Required: *Maximum surface loading at Peak flow of 1600 gal./day/ft², and at Design flow of 800 gal./day/ft². Side water depth must be at least 10 ft. for surface areas of 1250 ft² or more. Effective detention times (based on liquid volume above a 3 ft. sludge blanket) must be 1.1 hr. @ Peak flow and 2.2 hr. @ Design flow.*

Analysis: Allowable flow based on surface area:
Effective surface area of clarifier = $\pi(20^2) = 1257$ ft²

Allowable Peak flow = $(1257)(1600)/10^6 = 2.01$ mgd
Allowable Design flow = $(1257)(800)/10^6 = 1.01$ mgd

Side water depth is not adequate.

Allowable flow based on detention time:
Detention time is based on effective surface area and side water depth less three ft. (1257)(6 - 3) = 3771 ft³ x 7.48 gal./ft³ = 28,207 gal.

Allowable Peak flow = 28,207 gal./(1.1 hr.)(1 day/24 hr.)
= 0.62 mgd

Allowable Design flow = 28,207 gal./(2.2 hr.)(day/24 hr.)
= 0.31 mgd

The flows based on detention govern, since they are less than the flows based on surface area.

D. Effluent Works.

1. Chlorine Contract Chamber. 112 ft. long x 5 ft. bottom width/14 ft. top width x 4.5 ft. deep, divided into two chambers by a center wall running the length of the chamber, total tank volume of 34,600 gallons (4626 ft^3). Chlorination equipment designed for 0 to 500 pounds per day of chlorine.

Required: *Detention time of 20 minutes @ peak flow.*

Analysis: Existing volume approximately 4,626 ft^3
4,626 ft^3/20 min. = 231 cfm = 3.9 cfs = 2.49 mgd

2. Flow Measurement. 90° V-notch weir.

E. Sludge Processing.

1. Digester. 60 ft. diam. x 12 ft. side water depth, 37,670 ft^3 of volume per 1981 plans.

Requirement: *Minimum solids retention time of 30 days required for unheated anaerobic digesters. 26.5 ft^3/lb BOD_5/day required.*

Analysis: Volume = 37,670 ft^3

Allowable BOD_5 = 37,670 ft^3 / 26.5 ft^3/lb BOD_5/day
= 1421 lb. BOD_5/day.

At an influent strength of 200 mg/l, this volume is sufficient for 0.85 mgd design flow.

2. Drying Beds. Total area of 14,304 ft^2.

F. Plant Capacity

		ADF	PEAK
1.	Influent Lift Station		1.656 mgd
2.	Primary Clarifier	1.23 mgd	2.21 mgd
3.	Final Clarifier	0.31 mgd	0.62 mgd
4.	Chlorine Contact Chamber		2.49 mgd
5.	Anaerobic Digesters	0.85 mgd	

OCT 1990
RECEIVED
CITY
OF
GROVES

PERMIT NO. 10094-02
(corresponds to
NPDES PERMIT NO. TX0024651)

This is a renewal of Permit No. 10094-02, approved September 24, 1985.

TEXAS WATER COMMISSION
Stephen F. Austin State Office Building
1700 N. Congress Ave.
Austin, Texas 78711

PERMIT TO DISPOSE OF WASTES
under provisions of Chapter 26
of the Texas Water Code

City of Groves

whose mailing address is

P.O. Box 846
Groves, Texas 77619

is authorized to treat and dispose of wastes from the North Wastewater Treatment Plant

located at the western corner of Georgia Avenue and Mockingbird Lane, approximately 1/2 mile northeast of the intersection of State Highway 347 and State Highway Spur 136 in Jefferson County, Texas

to Jefferson County Drainage District No. 7 Main A-3 Canal; thence to Main Canal; thence to the Main A Canal, the Alligator Bayou; thence to Taylor Bayou; thence to the Drainage District No. 7 Main Outfall Canal; thence into the Intracoastal Waterway in Segment No. 0703 of the Neches - Trinity Coastal Basin

only in accordance with effluent limitations, monitoring requirements and other conditions set forth herein, as well as the rules of the Texas Water Commission ("Commission"), the laws of the State of Texas, and other orders of the Commission. The issuance of this permit does not grant to the permittee the right to use private or public property for conveyance of wastewater along the herein described discharge route. This includes property belonging to but not limited to any individual, partnership, corporation or other entity. Neither does this permit authorize any invasion of personal rights nor any violation of federal, state, or local laws or regulations. It is the responsibility of the permittee to acquire property rights as may be necessary to use the herein described discharge route.

This permit and the authorization contained herein shall expire at midnight, five years after the date of Commission approval.

APPROVED, ISSUED AND EFFECTIVE this 22nd day of October 19 90.

ATTEST: Brenda W. Foster

For the Commission

10-22-95

FINAL EFFLUENT LIMITATIONS AND MONITORING REQUIREMENTS

Outfall Number 001

1. During the period beginning upon the date of issuance and lasting through the date of expiration, the permittee is authorized to discharge subject to the following effluent limitations:

   The daily average flow of effluent shall not exceed 0.83 million gallons per day (MGD); nor shall the average discharge during any two-hour period (2-hour peak) exceed 2,000 gallons per minute (gpm).

Effluent Characteristic	Discharge Limitations				Minimum Self-Monitoring Requirements	
	Daily Avg mg/l(lbs/day)	7-day Avg mg/l	Daily Max mg/l	Single Grab mg/l	Report Daily Avg. & Daily Max. Measurement Frequency	Sample Type
Flow, MGD	Report	N/A	Report	N/A	Continuous	Totalizing meter
Biochemical Oxygen Demand (5-day)	20(138)	30	45	65	One/week	Composite
Total Suspended Solids	20(138)	30	45	65	One/week	Composite

2. The effluent shall contain a chlorine residual of at least 1.0 mg/l and shall not exceed a chlorine residual of 4.0 mg/l after a detention time of at least 20 minutes (based on peak flow), and shall be monitored daily by grab sample. An equivalent method of disinfection may be substituted only with prior approval of the Commission.

3. The pH shall not be less than 6.0 standard units nor greater than 9.0 standard units and shall be monitored once per week by grab sample.

4. There shall be no discharge of floating solids or visible foam in other than trace amounts and no discharge of visible oil.

5. Effluent monitoring samples shall be taken at the following location(s): Following the final treatment unit.

6. The effluent shall contain a minimum dissolved oxygen of 5.0 mg/l and shall be monitored once per week by grab sample.

John Hall, *Chairman*
Pam Reed, *Commissioner*
R. B. "Ralph" Marquez, *Commissioner*
Dan Pearson, *Executive Director*

TEXAS NATURAL RESOURCE CONSERVATION COMMISSION

Protecting Texas by Reducing and Preventing Pollution

June 14, 1995

The Honorable Sylvester Moore, Mayor
City of Groves
P.O. Box 846
Groves, Texas 77619

Re: City of Groves - Renewal of Permit No. 10094-002

Dear Mayor Moore:

Attached for your review and comment is a copy of a draft proposed permit for the above-referenced operation. This draft is subject to further staff review and modification; however, we believe it generally includes the terms and conditions that are appropriate to your discharge. Please read the entire draft carefully because the following changes have been proposed since the permit was last issued:

1. Please note, that according to the analysis using the QUAL-TX model, an effluent set of 5 mg/l $CBOD_5$, 12 mg/l TSS, 3 mg/l NH_3-N and 6 mg/l DO will not meet the dissolved oxygen criterion established by the TNRCC Standards Team. Therefore, no discharge of pollutants into waters in the State is authorized in the final phase of the draft permit. (The series of perennial canals has been classified according to TNRCC implementation procedures for the Texas Surface Water Quality Standards and 30 TAC Chapter 307.4(H) and (K) with presumed high aquatic life use with 5.0 mg/l dissolved oxygen);
2. Regarding the proposed effluent limitations the City may request a standards revision for the discharge stream. Information regarding the discharge stream classification may be submitted to Mr. Charles Bayer of the Research of Environmental Assessment Section of the Water Planning and Assessment Division;
3. The sludge language in the draft permit has been modified since the last permit issuance; and
4. The expiration date on Page 1 of the draft permit is in accordance with the newly adopted rules of basin schedules in 30 TAC Chapter 305.71.

If you have any comments or questions, please contact me at (512) 239-4545 within two weeks from the date of this letter.

Sincerely,

Zdenek Matl

Zdenek Matl
Municipal Team, Permitting Section (MC 148)
Watershed Management Division

ZM:sp

Attachment
cc: TNRCC Region 10

PERMIT NO. 10094-002
(corresponds to
NPDES PERMIT NO. TX0024651)

This is a renewal of Permit No. 10094-002, approved October 22, 1990.

TEXAS NATURAL RESOURCE CONSERVATION COMMISSION
P. O. Box 13087
Austin, Texas 78711-3087

PERMIT TO DISPOSE OF WASTES
under provisions of Chapter 26
of the Texas Water Code

DRAFT
SUBJECT TO REVISION

City of Groves

whose mailing address is

P.O. Box 846
Groves, Texas 77619

is authorized to treat and dispose of wastes from the North Wastewater Treatment Facilities

located at the western corner of Georgia Avenue and Mockingbird Lane, approximately 0.5 mile northeast of the intersection of State Highway 347 and State Highway Spur 136 in Jefferson County, Texas

to ditch A-3A; thence into a series of perennial canals; thence into the Intracoastal Waterway in Segment No. 0702 of the Neches-Trinity Coastal Basin

only in accordance with effluent limitations, monitoring requirements and other conditions set forth herein, as well as the rules of the Texas Natural Resource Conservation Commission ("Commission"), the laws of the State of Texas, and other orders of the Commission. The issuance of this permit does not grant to the permittee the right to use private or public property for conveyance of wastewater along the herein described discharge route. This includes property belonging to but not limited to any individual, partnership, corporation or other entity. Neither does this permit authorize any invasion of personal rights nor any violation of federal, state, or local laws or regulations. It is the responsibility of the permittee to acquire property rights as may be necessary to use the herein described discharge route.

This permit and the authorization contained herein shall expire at midnight, October 1, 1998.

ISSUED DATE:

ATTEST:______________________________ ______________________________
For the Commission

INTERIM EFFLUENT LIMITATIONS AND MONITORING REQUIREMENTS — Outfall Number 001

1. During the period beginning upon the date of issuance and lasting through September 30, 1998*, the permittee is authorized to discharge subject to the following effluent limitations:

   The daily average flow of effluent shall not exceed 0.83 million gallons per day (MGD); nor shall the average discharge during any two-hour period (2-hour peak) exceed 2,000 gallons per minute (gpm).

Effluent Characteristic	Discharge Limitations				Minimum Self-Monitoring Requirements	
	Daily Avg mg/l(lbs/day)	7-day Avg mg/l	Daily Max mg/l	Single Grab mg/l	Report Daily Avg. & Daily Max. Measurement Frequency	Sample Type
Flow, MGD	Report	N/A	Report	N/A	Five/week	Totalizing meter
Biochemical Oxygen Demand (5-day)	20(138)	30	45	65	One/week	Composite
Total Suspended Solids	20(138)	30	45	65	One/week	Composite

2. The effluent shall contain a chlorine residual of at least 1.0 mg/l and shall not exceed a chlorine residual of 4.0 mg/l after a detention time of at least 20 minutes (based on peak flow), and shall be monitored daily by grab sample. An equivalent method of disinfection may be substituted only with prior approval of the Commission.

3. The pH shall not be less than 6.0 standard units nor greater than 9.0 standard units and shall be monitored twice per month by grab sample.

4. There shall be no discharge of floating solids or visible foam in other than trace amounts and no discharge of visible oil.

5. Effluent monitoring samples shall be taken at the following location(s): Following the final treatment unit.

6. The effluent shall contain a minimum dissolved oxygen of 5.0 mg/l and shall be monitored once per week by grab sample.

* **See Other Requirement No. 1**

OTHER REQUIREMENTS

1. FINAL PHASE - During the period beginning upon October 1, 1998 and lasting through the date of expiration, no discharge of pollutants into waters in the State is authorized and the following provisions apply:

   Conditions of the permit: No discharge of pollutants to surface water in the State is authorized.

   Character: Treated Domestic Sewage Effluent

   Volume: 30-day Average - 0.83 MGD from the treatment system

   Quality: The following degree of treatment shall be required:

   A.

Parameter	30-day Average	Effluent Concentrations (Not to Exceed) Single Grab
BOD_5, mg/l	20	65

   The pH shall not be less than 6.0 standard units nor greater than 9.0 standard units.

   The effluent shall be chlorinated in a chlorine contact chamber to a residual of 1.0 mg/l with a minimum detention time of 20 minutes.

   B. Monitoring Requirements:

Parameter	Monitoring Frequency	Sample Type
Flow, MGD	Five/week	Instantaneous
BOD_5, mg/l	One/week	Composite
pH	Two/month	Grab
Chlorine, mg/l	Daily	Grab

   The monitoring shall be done after the final treatment unit. These records shall be maintained on a monthly basis and be available at the plant site for inspection by authorized representatives of the Commission for at least three years.

2. This Category C facility shall be operated and maintained by a chief operator or operator in responsible charge holding a valid Class C certificate of competency or higher issued pursuant to 30 TAC Chapter 325. All shift supervisors and other plant operators shall be certified in accordance with the applicable provisions of Chapter 325. Note, Class D certificates are not renewable at any activated sludge facility, regardless of size, or any trickling filter or RBC facility with a permitted flow greater than 100,000 gallons per day.

3. The permittee shall submit within two years from the date of permit issuance an amendment application providing information about the no discharge facility to the Texas Natural Resource Conservation Commission, Permitting Section (MC 148), Watershed Management Division.

4. The permittee shall obtain approval from the Watershed Management Division, Permitting Section (MC 148)of an engineering report and/or plans and specifications that clearly show how the treatment system will meet the final permitted no discharge requirements required on Page 20 of the permit prior to construction or October 1, 1998, whichever occurs first.

A4 - CITY OF GROVES SOUTH WWTF

A4 - CITY OF GROVES SOUTH WWTF

A. General

The existing treatment units are analyzed according to current TNRCC design criteria for secondary treatment to determine the ratable plant sizing. The analysis is based on assumed secondary treatment requirements.

The plant is presently permitted for 2.29 mgd design flow (maximum monthly average) at secondary standards (20 mg/l BOD_5 and TSS) with a DO requirement of 5 mg/l. The permitted two hour peak flow is 4771 gpm (equivalent to 6.87 mgd).

The Groves South treatment facility consist of bar screens, preaeration units, primary clarifier, trickling filters, final clarifier, chlorination, dechlorination, anaerobic sludge digesters, and sludge drying beds. Treatment units are as described in the following sections.

B. Preliminary Treatment.

1. Bar Screens. Two fixed bar screens, 3 ft. side channels and screens, 1/2" bars with 1" openings between bars, screens at 45°, 3 ft. deep channels.

   Required: *Some form of screening; bar openings minimum ½" for mechanical screens; velocities @ design flow minimum 2 ft./sec through channel, < 3 ft./sec. through screen.*

2. Preaeration. Two basins, 22 ft. x 22 ft. x 13.5 ft. SWD, hopper bottoms.

3. Primary Clarifier. 60 ft. diam. x 10 ft. side water depth, bottom slopes to center, 9 ft. diam. stilling well, mechanical sludge collection.

   Required: *Maximum surface loading at Peak flow of 1800 gal./day/ft^2, and at Design flow of 1000 gal./day/ft^2. Side water depth must be at least 7 ft.*

   Analysis: Allowable flow based on surface area:
   Effective surface area of clarifier = $\pi(30^2 - 4.5^2) = 2764$ ft^2

   Allowable Peak flow = $(2764)(1800)/10^6 = 4.98$ mgd
   Allowable Design flow = $(2764)(1000)/10^6 = 2.76$ mgd

   Side water depth is adequate.

   Primary clarifier is considered to remove 35% of raw influent BOD_5.

C. Secondary Treatment.

1. Trickling Filters. 3 @ 60 ft. diam. x 5.5 ft. media depth.

Required: *Typical design loadings for high rate rock media are 230-900 gpd/ft^2 hydraulic loading and 25-300 lb BOD/day/1000 ft^3 organic loading, and a BOD removal of 65-85%. The National Research Council formula may be used for calculating the efficiency of rock filters.*

Analysis: Gross media area = $(3)(\pi)(30^2) = 8482$ ft^2 {0.195 acres}

Media volume = 8482 x 5.5 = 46,651 ft^3 {1.071 Acre-ft.}

Calculate loading rates based on 2.29 mgd permitted ADF:

Hydraulic Loading = 2,290,000 gpd / 8482 ft^2
= 270 gpd/ft^2

Organic Loading $= \dfrac{65\%(2.29 \text{ mgd} \times 8.345 \times 200 \text{ mg/l})}{46{,}651 \text{ ft}^3}$
= 53.3 lb BOD/day/1000 ft^3

Calculate the efficiency of the trickling filters based on the NRC formula.

$$E_1 = 1 / [1 \pm m(i)^n]$$

Where:
- n = 0.5
- m = 0.0085
- i = W/VF
- W = lb. BOD to first stage of filter
- V = ac-ft of trickling filter media
- F = recirculation factor

$$F = [1 \pm R/I] / [1 \pm (1 - f)R/I]^2$$

Where:
- R = rate of recirculation
- I = rate of raw influent
- f = weighing factor, generally taken as 0.9 for domestic sewage

$$F = \frac{1 + (2.0/2.29)}{[1 \pm (1 - 0.9)(2.0 / 2.29)]^2}$$

$$= \quad 1.58$$

$$E_1 = \frac{1}{1 \pm 0.0085\ \{2494 / [(1.071)(1.58)]\}^{0.5}}$$

$$= \quad 0.754 \text{ (or 75.4\%)} \qquad \textit{85\% Required}$$

2. <u>Final Clarifier</u>. 60 ft. diam. x 8 ft. side water depth, bottom slopes to center, 9 ft. diam. stilling well, mechanical sludge collection.

Required: *Maximum surface loading at Peak flow of 1600 gal./day/ft², and at Design flow of 800 gal./day/ft². Side water depth must be at least 10 ft. for surface areas of 1250 ft² or more. Effective detention times (based on liquid volume above a 3 ft. sludge blanket) must be 1.1 hr. @ Peak flow and 2.2 hr. @ Design flow.*

Analysis: <u>Allowable flow based on surface area</u>:
Effective surface area of clarifier = $\pi(30^2) = 2827\ ft^2$

Allowable Peak flow = $(2827)(1600)/10^6 = 4.52$ mgd
Allowable Design flow = $(2827)(800)/10^6 = 2.26$ mgd

Side water depth is <u>not</u> adequate.

<u>Allowable flow based on detention time</u>:
Detention time is based on effective surface area and side water depth less three ft. (2827)(8 - 3) = 14,135 ft^3 x 7.48 gal./ft^3 = 105,730 gal.

Allowable Peak flow = 105,730 gal./(1.1 hr.)(1 day/24 hr.)
= 2.31 mgd

Allowable Design flow = 105,730 gal./(2.2 hr.)(day/24 hr.)
= 1.15 mgd

The flows based on detention govern, since they are less than the flows based on surface area.

D. <u>Effluent Works</u>.

1. <u>Chlorine Contract Chamber</u>. 1,150 linear feet x 36" diameter effluent pipe @ 0.06% slope.

Required: *Detention time of 20 minutes @ peak flow.*

Analysis: Existing volume approximately 8,129 ft^3
8,129 ft^3/20 min. = 406 cfm = 6.8 cfs = 4.39 mgd

E. Sludge Processing.

1. Digester. Two 45 ft. diameter anaerobic digesters. Primary = 21.5 ft. SWD, Secondary = 19.5 ft. SWD.

Requirement: Minimum solids retention time of 30 days required for unheated anaerobic digesters. 26.5 ft^3/lb BOD_5/day required.

Analysis: Volume = 65,208 ft^3

Allowable BOD_5 = 65,208 ft^3 / 26.5 ft^3/lb BOD_5/day
= 2461 lb. BOD_5/day.

At an influent strength of 200 mg/l, this volume is sufficient for 1.47 mgd design flow.

2. Drying Beds. Total area of 26,400 ft^2.

F. Plant Capacity

		ADF	PEAK
1.	Primary Clarifier	2.76 mgd	4.98 mgd
2.	Final Clarifier	1.15 mgd	2.31 mgd
3.	Chlorine Contact Pipe		4.39 mgd
5.	Anaerobic Digesters	1.47 mgd	

PERMIT NO. 10094-001
(corresponds to
NPDES PERMIT NO. TX0024643)

This amendment supersedes and replaces Permit No. 10094-001 approved January 31, 1990.

TEXAS WATER COMMISSION
Stephen F. Austin State Office Building
1700 N. Congress Ave.
Austin, Texas 78711

PERMIT TO DISPOSE OF WASTES
under provisions of Chapter 26
of the Texas Water Code

City of Groves

whose mailing address is

P.O. Box 846
Groves, Texas 77619

is authorized to treat and dispose of wastes from the South Wastewater Treatment Facilities

located on Taft Avenue approximately 1 mile southeast of the intersection of Taft Avenue and State Highway 73 in Jefferson County, Texas

to the Sabine-Neches Canal in Segment No. 0703 of the Neches-Trinity Coastal Basin

only in accordance with effluent limitations, monitoring requirements and other conditions set forth herein, as well as the rules of the Texas Water Commission ("Commission"), the laws of the State of Texas, and other orders of the Commission. The issuance of this permit does not grant to the permittee the right to use private or public property for conveyance of wastewater along the herein described discharge route. This includes property belonging to but not limited to any individual, partnership, corporation or other entity. Neither does this permit authorize any invasion of personal rights nor any violation of federal, state, or local laws or regulations. It is the responsibility of the permittee to acquire property rights as may be necessary to use the herein described discharge route.

This permit and the authorization contained herein shall expire at midnight, January 31, 1995.

APPROVED, ISSUED AND EFFECTIVE this 9th day of March 19 92 .

ATTEST: Gloria A. Vasquez John Hall
For the Commission

S STP TWC permit 10094-01 expires: 1-31-95

INTERIM EFFLUENT LIMITATIONS AND MONITORING REQUIREMENTS

Outfall Number 001

1. During the period beginning upon the date of issuance and lasting through December 31, 1992, the permittee is authorized to discharge subject to the following effluent limitations:

The daily average flow of effluent shall not exceed 2.29 million gallons per day (MGD); nor shall the average discharge during any two-hour period (2-hour peak) exceed 4771 gallons per minute (gpm).

Effluent Characteristic	Discharge Limitations				Minimum Self-Monitoring Requirements	
	Daily Avg mg/l(lbs/day)	7-day Avg mg/l	Daily Max mg/l	Single Grab mg/l	Report Daily Avg. & Daily Max. Measurement Frequency	Sample Type
Flow, MGD	Report	N/A	Report	N/A	Continuous	Totalizing meter
Biochemical Oxygen Demand (5-day)	20 (382)	30	45	65	Two/week	Composite
Total Suspended Solids	20 (382)	30	45	65	Two/week	Composite
Copper	Report(Report)	N/A	Report	N/A	Two/month	Composite

2. The effluent shall contain a chlorine residual of at least 1.0 mg/l after a detention time of at least 20 minutes (based on peak flow), and shall be monitored daily by grab sample. The permittee shall dechlorinate the chlorinated effluent to less than 0.1 mg/l chlorine residual and shall monitor daily by grab sample after the dechlorination process. An equivalent method of disinfection may be substituted only with prior approval of the Commission.

3. The pH shall not be less than 6.0 standard units nor greater than 9.0 standard units and shall be monitored once per week by grab sample.

4. There shall be no discharge of floating solids or visible foam in other than trace amounts and no discharge of visible oil.

5. Effluent monitoring samples shall be taken at the following location(s): Following the final treatment unit.

6. The effluent shall contain a minimum dissolved oxygen concentration of 2 mg/l and shall be monitored twice per week by grab sample.

FINAL EFFLUENT LIMITATIONS AND MONITORING REQUIREMENTS

Outfall Number 001

1. During the period beginning upon January 1, 1993 and lasting through the date of expiration, the permittee is authorized to discharge subject to the following effluent limitations:

The daily average flow of effluent shall not exceed 2.29 million gallons per day (MGD), nor shall the average discharge during any two-hour period (2-hour peak) exceed 4771 gallons per minute (gpm).

Effluent Characteristic	Discharge Limitations				Minimum Self-Monitoring Requirements Report Daily Avg. & Daily Max.	
	Daily Avg mg/l(lbs/day)	7-day Avg mg/l	Daily Max mg/l	Single Grab mg/l	Measurement Frequency	Sample Type
Flow	N/A	N/A	N/A	N/A	Continuous	Totalizing meter
Biochemical Oxygen Demand (5-day)	20 (382)	30	45	65	Two/week	Composite
Total Suspended Solids	20 (382)	30	45	65	Two/week	Composite
Copper	0.029(0.55)	N/A	0.061	N/A	Two/month	Composite

2. The effluent shall contain a chlorine residual of at least 1.0 mg/l after a detention time of at least 20 minutes (based on peak flow), and shall be monitored daily by grab sample. The permittee shall dechlorinate the chlorinated effluent to less than 0.1 mg/l chlorine residual and shall monitor daily by grab sample after the dechlorination process. An equivalent method of disinfection may be substituted only with prior approval of the Commission.

3. The pH shall not be less than 6.0 standard units nor greater than 9.0 standard units and shall be monitored once per week by grab sample.

4. There shall be no discharge of floating solids or visible foam in other than trace amounts and no discharge of visible oil.

5. Effluent monitoring samples shall be taken at the following location(s): following the final treatment unit.

6. The effluent shall contain a minimum dissolved oxygen concentration of 2 mg/l and shall be monitored twice per week by grab sample.

Shall report daily average and daily maximum limitation ← see Endorcement dated 1-7-93 attached to the back of this permit

TEXAS NATURAL RESOURCE CONSERVATION COMMISSION

Protecting Texas by Reducing and Preventing Pollution

June 8, 1995

The Honorable Sylvester Moore, Mayor
City of Groves
P.O. Box 846
Groves, Texas 77619-6048

Re: City of Groves - Amendment of Permit No. 10094-001

Dear Mayor Groves:

Attached for your review and comment is a copy of a draft proposed permit for the above-referenced operation. This draft is subject to further staff review and modification; however, we believe it generally includes the terms and conditions that are appropriate to your discharge. Please read the entire draft carefully because the following changes have been proposed since the permit was last issued:

1. Biomonitoring is required in the draft permit;
2. The sludge language in the draft permit has been modified since the last permit issuance;
3. According to the submitted information and a TNRCC evaluation, the limit of Copper has been deleted from the existing permit;
4. The expiration date on Page 1 of the draft permit is in accordance with the newly adopted rules of basin schedules in 30 TAC Chapter 305.71; and
5. This Category B facility shall be operated and maintained by a chief operator or operator in responsible charge holding a valid Class B certificate of competency or higher issued pursuant to 30 TAC Chapter 325.

If you have any comments or questions, please contact me at (512) 239-4545 within two weeks from the date of this letter.

Sincerely,

Zdenek Matl

Zdenek Matl
Municipal Team
Permitting Section (MC 148)
Watershed Management Division

ZM:sp

Attachment
cc: TNRCC Region 10

PERMIT NO. 10094-001
(corresponds to
NPDES PERMIT NO. TX0024643)

This permit supersedes and replaces Permit No. 10094-001, approved March 9, 1992.

TEXAS NATURAL RESOURCE CONSERVATION COMMISSION
P. O. Box 13087
Austin, Texas 78711-3087

PERMIT TO DISPOSE OF WASTES
under provisions of Chapter 26
of the Texas Water Code

DRAFT
SUBJECT TO REVISION

City of Groves

whose mailing address is

P.O. Box 846
Groves, Texas 77619-6048

is authorized to treat and dispose of wastes from the South Wastewater Treatment Facilities

located on Taft Avenue approximately 1 mile southeast of the intersection of Taft Avenue and State Highway 73 in Jefferson County, Texas

through a 36" pipe; thence under the Hurricane Protection Levee; thence to the Sabine-Neches Canal in Segment No. 0703 of the Neches-Trinity Coastal Basin

only in accordance with effluent limitations, monitoring requirements and other conditions set forth herein, as well as the rules of the Texas Natural Resource Conservation Commission ("Commission"), the laws of the State of Texas, and other orders of the Commission. The issuance of this permit does not grant to the permittee the right to use private or public property for conveyance of wastewater along the herein described discharge route. This includes property belonging to but not limited to any individual, partnership, corporation or other entity. Neither does this permit authorize any invasion of personal rights nor any violation of federal, state, or local laws or regulations. It is the responsibility of the permittee to acquire property rights as may be necessary to use the herein described discharge route.

This permit and the authorization contained herein shall expire at midnight, July 1, 1998.

ISSUED DATE:

ATTEST: ______________________

____
For the Commission

FINAL EFFLUENT LIMITATIONS AND MONITORING REQUIREMENTS

Outfall Number 001

1. During the period beginning upon the date of issuance and lasting through the date of expiration, the permittee is authorized to discharge subject to the following effluent limitations:

   The daily average flow of effluent shall not exceed 2.29 million gallons per day (MGD); nor shall the average discharge during any two-hour period (2-hour peak) exceed 4,771 gallons per minute (gpm).

Effluent Characteristic	Discharge Limitations				Minimum Self-Monitoring Requirements	
	Daily Avg mg/l(lbs/day)	7-day Avg mg/l	Daily Max mg/l	Single Grab mg/l	Report Daily Avg. & Daily Max. Measurement Frequency	Sample Type
Flow, MGD	Report	N/A	Report	N/A	Continuous	Totalizing meter
Biochemical Oxygen Demand (5-day)	20(382)	30	45	65	Two/week	Composite
Total Suspended Solids	20(382)	30	45	65	Two/week	Composite

2. The effluent shall contain a chlorine residual of at least 1.0 mg/l after a detention time of at least 20 minutes (based on peak flow), and shall be monitored daily by grab sample. The permittee shall dechlorinate the chlorinated effluent to less than 0.1 mg/l chlorine residual and shall monitor daily by grab sample after the dechlorination process. An equivalent method of disinfection may be substituted only with prior approval of the Commission.

3. The pH shall not be less than 6.0 standard units nor greater than 9.0 standard units and shall be monitored once per week by grab sample.

4. There shall be no discharge of floating solids or visible foam in other than trace amounts and no discharge of visible oil.

5. Effluent monitoring samples shall be taken at the following location(s): Following the final treatment unit.

6. The effluent shall contain a minimum dissolved oxygen of 2.0 mg/l and shall be monitored twice per week by grab sample.

B1 - City of Nederland

TABLE B-1
POPULATION PROJECTIONS
NEDERLAND, TEXAS (JEFFERSON COUNTY)

A	SETRPC Water Quality Management Plan - 1993				Texas Water Development Board Most Likely Series				Population					
YEAR	*B* Nederland	*C* City % Increase	*D* District % Increase	*E* Jefferson County WCID # 10	*F* Nederland	*G* City % Increase	*H* County % Increase	*I* Jefferson County	*J* City of Nederland Per TWDB	*K* District % Increase	*L* Jefferson County WCID No. 1	*M* Proposed Annexation Outside District	*N* Total City Population	*O* Total City Sewered Population
1950	3805							195,083						
1960	12,036							245,659						
1970	16,810							246,347						
1980	16,855				16,855			250,938						
1990	16,192				16,192			239,397						
1992	16,312			5000	16,370			243,251	16,370		5000			
1994	16,432	0.74	3.34	5167	16,549	1.09	1.58	247,104	16,549	4.92	5246	1406 (incl. 1101 sewered)	16,549	17,650
1995	16,492	0.37	1.61	5250	16,638	0.54	0.78	249,031	16,638	2.35	5369	1417 (incl. 1110 sewered)	16,638	17,748
2000	16,822	2.00	2.38	5375	17,084	2.68	3.87	258,665	17,084	3.19	5540	1472	24,096	18,556
2005	17,157	1.99	2.33	5500	17,123	0.23	1.81	263,340	17,123	0.27	5555	1499	24,177	18,622
2009	17,433	1.61	1.82	5600	17,154	0.18	1.42	267,079	17,154	0.20	5566	1520	24,240	18,674
2010	17,502	0.40	0.45	5625	17,162	0.05	0.35	268,014	17,162	0.06	5569	1525	24,256	18,687
2014					17,293	0.76	1.40	271,756	17,293		5611	1546	24,450	18,839
2020					17,489	1.13	2.07	277,368	17,489		5674	1578	24,741	19,067
2024					17,536	0.27	0.83	279,671	17,536		5689	1591	24,816	19,127
2030					17,606	0.40	1.24	283,125	17,606		5712	1611	24,929	19,217

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POPULATION PROJECTIONS
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A	SETRPC Water Quality Management Plan - 1993				Texas Water Development Board Most Likely Series				Population					
YEAR	B Nederland	C City % Increase	D District % Increase	E Jefferson County WCID # 10	F Nederland	G City % Increase	H County % Increase	I Jefferson County	J City of Nederland Per TWDB	K District % Increase	L Jefferson County WCID No. 1	M Proposed Annexation Outside District	N Total City Population	O Total City Sewered Population
1950	3805							195,083						
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TABLE B-1 NOTES

1. The South East Texas Regional Planning Commission's Water Quality Management Plan, as updated in 1993, has for purposes of this report been superseded by the TWDB Most Likely Series (revised, in draft form, 1994). The revised projections are based on an increased inward migration rate for Southeast Texas as a result of recent employment growth.

2. Column B: Projections were provided for every five years through 2010, with other years interpolated.

3. Column E: Population was shown as (estimated) 5000 for 1992, with corresponding flow of 0.4 mgd. Populations for 1995, 2000, 2005, and 2010 were not shown directly in Plan, but were prorated on basis of flow projections in Plan.

4. Columns F and I: Projections were provided for every ten years through 2050. *(County population before 1980 is based on historic census figures.)* It is assumed for this report that the City projections do not reflect anticipated future annexations of Jefferson County WCID No. 10 and surrounding unincorporated areas.

5. Columns C, D, G, H, and K: Percent increase shown for each year is based on increase from the year on the row above.

6. Column L: Population for 1992 is taken at 5000 from SETRPC Plan. For subsequent years through 2010, population reflects a rate of increase at each stage equal to (Column D)(Column K)/(Column C). After that date, the rate of increase is taken to be equal to rate of City increase (Column G).

7. Column M: Total population for 1994 is based on 1994 house count.* Subsequent total population is taken as proportional to county population. For 1994 and 1995, sewered population is based on best available information on sewered areas. For 2000 and later, it is assumed that City sewer service will be extended to all houses in this area.

8. Columns N and O: Total City population is based on anticipated future annexation of District and surrounding areas. However, the District population is excluded from City sewered population since the City anticipates leaving the District's wastewater collection and treatment system intact. (For purposes of this table, annexation is assumed to occur between 1995 and 2000. For 1994 and 1995, only the sewered portion of the area outside District is included in sewered population. For 2000 and later, all annexed areas outside District are assumed to receive City sewer service.)

*See summary next page.

Table B-1 Notes (cont.)

Parkway Village (mobile home park):	187 active connections, management reports estimated 167 school children and 750 total population:		750
Other residential areas:	Total 230 customers, estimated 95% residential, assume 3 persons/residence:		656
	TOTAL	1406	

Available information indicates that the following portions of the area outside the District are presently (1994) receiving sewer service (wholesale or retail) from the City:

Parkway Village (through mobile park owners):		750
Ridgecrest and Crestview subdivisions (area bounded by U. S. 69, Canal Avenue, 27th, and LNVA Canal; included in area outside District discussed above; 123 connections, assume 95% residential @ 3 persons/residence)		351
	TOTAL	1101

FLOW PROJECTIONS
CITY OF NEDERLAND

Wastewater flows come primarily from the existing City, but a significant amount of flows come from two large areas outside the City. One of these areas is designated as the ETJ service area. This area is an area outside the City *(and also outside Jefferson County WCID No. 10)* which the City has indicated that it is likely to annex in the future. The area includes large amounts of vacant land, with most existing and projected development being residential. Portions of the ETJ service area already contribute waste flows to the City.

The other area is the Jefferson County Airport, which has no population.

Note that Jefferson County WCID No. 10 is excluded from City flow projections. Although the District may be annexed in the future along with the ETJ service area, the City anticipates leaving the existing District sewer system in service, with no effect on City flows.

Calculations are based mainly on population projections for the service area (Appendix B-1). As discussed above, this area includes the City and the ETJ service area. Approximately 70% of residents within the ETJ service area already receive City sewer service. It is assumed that the City will in the future annex the area and also extend sewer service to the remaining portions of the area.

A. Baseline Conditions (1994)

City sewered population 17,650 *(Including ETJ service area.)*

Annual average ADF (based on 24 month period ending August 1994) = 2.917 mgd *(Includes bypasses, based on quantities estimated and reported by City.)*

Maximum monthly ADF (May 1993) = 4.5 mgd *(Neglecting bypasses, since no reported bypasses occurred that month)*

Design ADF = (Maximum monthly ADF)(111%) *(Allowance for TNRCC 75/90 rule)*

Design ADF = 5.0 mgd *(4.5 mgd x 111%)*

Flow modelling of the interceptors leading to the treatment facility indicates that the maximum peak flow which could be presently transported to the treatment facility is 22.77 mgd. Therefore, the treatment facility should be designed with a two hour peaking factor of 5:1 (22.77 mgd:4.5 mgd).

Two hour peak (for design) should be **25.0 mgd**, based on 5 times the design ADF.

B. <u>15 Year Projections (2009)</u>

City sewered population projected at 18,674. *(Including effects of extending service to the ETJ service area)*

Annual average ADF 2.995 mgd *(Based on following methodology:*

Assume that 37% of annual wastewater flows are comprised of storm water, as reported by City of Groves. Increase both residential and nonresidential return flows in proportion to population.

Increase storm flows at 15% the rate of residential growth, since future sewer extensions will be relatively watertight.)

Deduct 2% from future return flows because of water conservation measures.)

Existing 2.917 mgd = 63% return flows = 1.838 mgd

37% storm flows = 1.079 mgd

18,674 ÷ 17,650 = 1.0580 (5.8% increase)

1.838 mgd x 1.058 = 1.945 mgd

1.079 mgd x [1 + (.0580 x 0.15)] = 1.089 mgd

Deduct 2% of 1.945 mgd = (-) 0.039 mgd

<u>NET TOTAL = 2.995 mgd</u>

Maximum monthly ADF = 5.13 mgd *(Increasing in proportion to average annual ADF)*

5.0 mgd x (2.995 ÷ 2.917) = 5.13 mgd

Two hour peak (for design) should be **25.67 mgd**, based on 5 times the design ADF.

C. <u>30 Year Projections (2024)</u>

City sewered population projected at 19,127. *(Including effects of extending service to the ETJ service area)*

Annual average ADF = 3.045 mgd *(Similar to 15 year projections)*

19,127 ÷ 17,650 = 1.0837 (8.37% increase)

1.838 mgd x 1.0837 = 1.992 mgd

1.079 mgd x [1 + (.0837 x 0.15)] = 1.093 mgd

Deduct 2% of 1.992 mgd = (-) 0.040 mgd

NET TOTAL = 3.045 mgd

Maximum monthly ADF = 5.22 mgd *(Increasing in proportion to average annual ADF)*

5.0 mgd x (3.045 ÷ 2.917) = 5.22 mgd

Two hour peak (for design) should be **26.10 mgd**, based on 5 times the design ADF.

B2 - City of Port Neches

TABLE B-2
POPULATION PROJECTIONS
PORT NECHES, TEXAS (JEFFERSON COUNTY)

A YEAR	*B* SETRPC Water Quality Management Plan - 1993	*C* Texas Water Development Board Most Likely Series	*D* Selected Population
1950	5488		5488
1960	8696		8696
1970	10,894		10,894
1980	13,944	13,944	13,944
1990	12,974	12,974	12,974
1992	13,114	13,227	13,227
1994	13,254	13,479	13,479
1995	13,324	13,606	13,606
2000	13,724	14,237	14,237
2005	14,124	14,392	14,392
2009	14,464	14,517	14,517
2010	14,549	14,548	14,548
2014		14,710	14,710
2020		14,953	14,953
2024		15,040	15,040
2030		15,171	15,171

TABLE B-2 NOTES

1. The South East Texas Regional Planning Commission's Water Quality Management Plan, as updated in 1993, has for purposes of this report been superseded by the TWDB Most Likely Series (recently adopted by Board action, January 1995). The revised projections are based on an increased inward migration rate for Southeast Texas as a result of recent employment growth.

2. Column B: Projections were provided for every five years through 2010, with other years interpolated.

3. Column C: Projections were provided for every ten years through 2050, with other years interpolated. It is assumed for this report that the City projections reflect no future annexations, and that there will in fact be no such annexations.

Table B-2 Notes (cont.)

4. Column D: Historical census figures (as quoted in SETRPC Plan) are used through 1990, then the TWDB projections (actual or interpolated) are used for all subsequent years. For purposes of this report, the City is assumed to serve all City residents and no residents outside the City.

FLOW PROJECTIONS
CITY OF PORT NECHES

Wastewater flows come from throughout the City. The sewered population is taken as equal to City population.

Calculations are based mainly on population projections for the City (Appendix B-2). It is assumed that all return flows will increase in proportion to the population (with slight adjustments for water conservation).

A. <u>Baseline Conditions (1994)</u>

City sewered population 13,479 *(Same as City population)*

Annual average ADF (based on 24 month period ending July 1994) *(Does not include any overflows, since the available monthly effluent reports do not show any overflows.)*

= 1.889 mgd main units

0.104 mgd storm water clarifiers

1.993 mgd TOTAL

Maximum monthly ADF = 3.03 mgd for main units (January 1993)

0.338 mgd for storm water clarifier (May 1994)

3.277 mgd for all units combined (June 1993)

Design ADF = (Maximum monthly ADF)(111%) *(Allowance for TNRCC 75/90 rule)*

Design ADF = 3.64 mgd *(3.277 mgd x 111%)*

Two hour peak: *An examination of monthly reports, including storm water clarifier usage, from September 1991 through July 1994, shows that the governing factor for peak 24 hr. flow is storm water clarifier usage. For each month that the storm water clarifiers were used as such, a summary report for that month was attached to the monthly reports showing various information for each date of usage. The flow-related data consisted of duration of usage, volume of storm water for that event, and total combined (24 hour) volume. A comparison with corresponding monthly operating reports indicates that the combined volume is the sum of (a) the 24 hr. flow through the main units, as shown on the monthly operating report for the following day, and (b) the flow diverted through the storm water clarifiers and not reflected on the monthly reporting forms.*

For calculation purposes, the storm flow is assumed to occur evenly throughout its duration as listed on the report, with the remainder of the combined flow passing through the main units at a constant rate throughout the day. The two hour peak would thus consist of the total of the two flow rates (storm flow plus other flow). The highest reliable value thus derived for the two hour peak occurred on June 13, 1994, as follows:

Duration of storm flow:	14 hr. 15 min.
Total storm flow:	5,615,000 gallons
Total daily combined flow:	11,871,000 gallons

5.615 mgd x 24/14.25 =	9.457 mgd
(11.571 - 5.615) mgd =	6.256 mgd
TOTAL	15.713 mgd two hour peak

To this peak historic plant flow should be added an amount for collection system overflows, which are known to be a serious problem in the Lee-Block neighborhood and which occur concurrently with the flows which activate the storm water clarifiers. The best available estimate of these flows is a previous engineering study which implied an overflow magnitude of 2.3 mgd.

Combined plant flow:	15.713 mgd
Manhole overflow:	2.3 mgd
TOTAL	18.013 mgd two hour peak[a]

[a] A higher value of 24.845 mgd was calculated similarly for June 20, 1992, but was considered unreliable because of limited transportation capacity as discussed below.

Peak reported flows may be unreliable because of two factors:

- *Observations by the City since completion of storm water clarifiers and related collection system work suggest that no more than 19 mgd can get to the plant because of limited gravity interceptor capacity upstream from the Park Lift Station. Overflows occur upstream from the gravity line, apparently because of inadequate line depth.*

- *The effluent meter for the main units (from which the reported effluent flows from main units are derived) reportedly functions inaccurately when the receiving stream level is high. The backwater problem results from operating practices at the downstream pump station operated by Jefferson County Drainage District No. 7. The problem reportedly needs pump station upgrading to correct the problem, and the District has been seeking funding.*

The design two hour peaking factor should be 5.5:1 (18.013 mgd:3.277 mgd).

Two hour peak (for design) should be **20.02 mgd**, based on 5.5 times the design ADF.

B. <u>15 Year Projections (2009)</u>

City sewered population projected at 14,710 *(Same as City population)*

Annual average ADF = 2.091 mgd *(Based on following methodology:*

Assume that 37% of annual wastewater flows are comprised of storm water, as reported by City of Groves. Increase both residential and nonresidential return flows in proportion to population.

Increase storm flows at 15% the rate of residential growth, since future sewer extensions will be relatively watertight.)

Deduct 2% from future return flows because of water conservation measures.)

Existing 1.993 mgd = 63% return flows = 1.256 mgd
37% storm flows = 0.737 mgd

14,710 ÷ 13,479 = 1.0913 (9.13% increase)

1.256 mgd x 1.0913 = 1.371 mgd

0.737 mgd x [1 + (.0913 x 0.15)] = 0.747 mgd

Deduct 2% of 1.371 mgd = (-) 0.027 mgd

<u>NET TOTAL = 2.091 mgd</u>

Maximum monthly ADF = 3.82 mgd *(Increasing in proportion to average annual ADF)*

3.64 mgd x (2.091 ÷ 1.993) = 3.82 mgd

Two hour peak (for design) = **21.01 mgd**, based on 5.5 times the design ADF.

C. <u>30 Year Projections (2024)</u>

City sewered population projected at 15,040 *(Same as City population)*

Annual average ADF = 2.123 mgd *(Similar to 15 year projections)*

15,040 ÷ 13,479 = 1.1158 (11.58% increase)

1.256 mgd x 1.1158 = 1.401 mgd

0.737 mgd x [1 + (.1158 x 0.15)] = 0.750 mgd

Deduct 2% of 1.401 mgd = (-) 0.028 mgd

<u>NET TOTAL = 2.123 mgd</u>

Maximum monthly ADF = 3.88 mgd *(Increasing in proportion to average annual ADF)*

3.64 x (2.123 ÷ 1.993) = 3.88 mgd

Two hour peak for design) should be **21.34 mgd**, based on 5.5 times the design ADF.

B3 - City of Groves

TABLE B-3
POPULATION PROJECTIONS
GROVES, TEXAS (JEFFERSON COUNTY)

A	Groves Population			*E*	Adjusted Sewered Population			
YEAR	*B* SETRPC Water Quality Management Plan - 1993	Texas Water Development Board Most Likely Series		Adjustments to Sewered Population [Fairlea (+)]	*F* Total	*G* Ratio North:Total	*H* North Plant	*I* South Plant
		C TWDB Draft (1994)	*D* Revised per 10/26/94 Conversation w/TWDB					
1960	17,304							
1970	18,067							
1980	17,090	17,090	17,090					
1990	16,745	16,513	16,744					
1992	16,825	16,623	16,856	700	17556	0.3333	5851	11,705
1994	16,906	16,733	16,967	600	17567	0.3352	5888	11,679
1995	16,946	16,788	17,023	600	17623	0.3352	5907	11,716
2000	17,149	17,063	17,302	600	17902	0.3352	6001	11,901
2005	17,355	17,112	17,351	600	17951	0.3351	6015	11,936
2009	17,521	17,151	17,391	600	17991	0.3351	6029	11,962
2010	17,563	17,161	17,401	600	18001	0.3351	6032	11,969
2014		17,296	17,538	600	18138	0.3351	6078	12,060
2020		17,498	17,743	600	18343	0.3351	6147	12,196
2024		17,549	17,794	600	18394	0.3351	6164	12,230
2030		17,625	17,872	600	18472	0.3351	6191	12,281

TABLE B-3 NOTES

1. The South East Texas Regional Planning Commission's Water Quality Management Plan, as updated in 1993, has for purposes of this report been superseded by the TWDB Most Likely Series (recently revised, in draft form, 1994). The revised projections are based on an increased inward migration rate for Southeast Texas as a result of recent employment growth. (See Note 3.)

2. Column B: Projections were provided for every five years through 2010, with other years interpolated.

3. Column C: Projections were provided for every ten years through 2050. These projections were furnished to the Engineer's staff in October 1994 as revised drafts pending approval. These projections represent increases from the projections furnished by the TWDB in July of 1994. The revisions reflect higher inward migration because of improved economic conditions and trends in the Southeast Texas area. *However, both the recent and earlier projections were based on a 1990 census figure of 16,513 for Groves. The U. S. Census Bureau in 1992 corrected the 1990 Groves population count with an slight increase, but the TWDB disregarded the revision in order to expedite the process of updating its projections.*

4. Column D: The corrected 1990 population for Groves is 16,744, a slight increase from the originally reported 16,513. In a conversation between the Engineer's staff and Jim Hull of the TWDB on October 26, 1994, Mr. Hull concurred that the projections should be increased in some manner to reflect the corrected 1990 census figure. The selected method of adjustment was to increase all projections across the board by a ratio of 16,744:16,513.

5. Columns E and F: Adjustments have been made to the City population to derive the total sewered population of the two plants. The adjustments reflect the fact that Groves receives wastewater flows from a portion of Port Arthur.. A negative growth factor *(from 1992 through 1994)* is used for future projections for Fairlea (a Port Arthur subdivision served by Groves), since the neighborhood is the subject of a partial buyout by the adjacent Fina refinery.

6. Columns G through I: The adjusted sewered population is divided between the North and South plants according to a ratio of approximately 1:2 (adjusted slightly through the study period according to disparate growth patterns within the two service areas).

   The SETRPC projections showed approximate sewered populations in 1992 of 5449 and 10,899 for the Groves North and South plants respectively. The resulting total sewered population came out slightly less than any version of the total City population shown in this table for 1990 or 1992. The sewered population for the Groves South Plant includes the Fairlea subdivision in Port Arthur. (The sewered populations were noted as approximate.)

   For these projections, the growth in City population within the two service areas is assumed to be distributed at a ratio of 1:2, with the South Plant further affected by declines in the Fairlea population.

FLOW PROJECTIONS
CITY OF GROVES - NORTH PLANT

Wastewater flows come from the northwestern *(compass western)* portion of the City. Little if any flows come from outside the City, since Fairlea *(in Port Arthur)* is served by the South Plant. Calculations are based mainly on population projections for the North Plant service area (Appendix B-3). It is assumed that all return flows will increase in proportion to the population (with slight adjustments for water conservation).

A. <u>Baseline Conditions (1994)</u>

City sewered population 17,567 including 5888 in North Plant service area.

Annual average ADF (based on 24 months from September 1992 through August 1994) = 0.713 mgd.

The ADF does not include any bypasses; only two months during this period have bypasses reported by the City, and they would have a negligible impact on the long-term ADF (less than 0.04 mgd for the highest month, or less than 0.004 mgd for a 24 month average).

Maximum monthly ADF (February 1992) = 1.166 mgd.

Design ADF = (Maximum monthly ADF)(111%) *(Allowance for TNRCC 75/90 rule)*

Flow modelling of the incoming interceptor indicates that the maximum flow could be as high as 3.805 mgd. Assuming that excessive I/I will be transported to the WWTF in the future, the maximum monthly ADF will be increased. Assuming an average daily flow of 3.805 mgd on the day of a rainfall event, the average monthly ADF for the three months with highest reported ADF (January 1992, February 1992, and January 1993) would be 1.755 mgd.

Design ADF = 1.95 mgd *(1.755 mgd x 111%)*

Two hour peak (based on flow recorder charts showing peak flows over a recent 12 month period; highest 2 hr. peak occurred 3-9-94) = 2.3 mgd

Two hour peak (for design) should be **5.85 mgd**, based on 3 times the design ADF per TNRCC requirements.

B. <u>15 Year Projections (2009)</u>

City sewered population projected at 17,791, including 6029 in North Plant service area.

Annual average ADF = 0.716 mgd *(Based on following methodology:*

37% of annual wastewater flows are comprised of storm water as reported by City on TWDB water conservation forms. Increase both residential and nonresidential return flows in proportion to population.

Increase storm flows at 15% the rate of residential growth, since any future sewer extensions will be relatively watertight.)

Deduct 2% from future return flows because of water conservation measures.)

Existing 0.713 mgd = 63% return flows = 0.449 mgd

37% storm flows = 0.264 mgd

6029 ÷ 5688 = 1.0239 (2.39% increase)

0.449 mgd x 1.0239 = 0.460 mgd

0.264 mgd x [1 + (.0239 x 0.15)] = 0.265 mgd

Deduct 2% of 0.449 mgd = (-) 0.009 mgd

NET TOTAL = 0.716 mgd

Maximum monthly ADF = 1.96 mgd *(Increasing in proportion to average annual ADF)*

1.95 mgd x (.716 ÷ .713) = 1.96 mgd

Two hour peak (for design) should be **5.88 mgd**, based on 3 times the maximum ADF per TNRCC requirements.

C. 30 Year Projections (2024)

City population projected at 18,194, including 6164 in North Plant service area.

Annual average ADF = 0.728 mgd *(Similar to 15 year projections)*

6164 ÷ 5888 = 1.0469 (4.69% increase)

0.449 mgd x 1.0469 = 0.470 mgd

0.264 mgd x [1 + (.0469 x 0.15)] = 0.266 mgd

Deduct 2% of 0.470 mgd = (-) 0.009 mgd

<u>NET TOTAL = 0.727 mgd</u>

Maximum monthly ADF = 1.99 mgd *(Increasing in proportion to average annual ADF)*

1.95 x (.727 ÷ .713) = 1.99 mgd

Two hour peak (for design) should be **5.97 mgd**, based on 3 times max. ADF per TNRCC requirements.

C4 - City of Groves South WWTF

FLOW PROJECTIONS
CITY OF GROVES - SOUTH PLANT

Wastewater flows come from the City exclusive of the northwestern *(compass western)* portion of the City. Some flows also come from Fairlea *(in Port Arthur)*. Calculations are based mainly on population projections for the South Plant service area (Appendix B-3). It is assumed that all return flows will increase in proportion to the population (with slight adjustments for water conservation).

A. Baseline Conditions (1994)

City sewered population 17,567 including 11,679 in South Plant service area.

Annual average ADF (based on 24 months from September 1992 through August 1994) = 1.285 mgd

The ADF does not include any bypasses; none were reported by the City for the South Plant for this period.

Maximum monthly ADF (January 1993) = 2.333 mgd.

Flow modelling of the interceptors served by the Taft Avenue lift station indicates that the maximum influent flow could be as high as 18.7 mgd. Based on information contained in the 1981 rehabilitation plans for the Taft Avenue lift station, it appears that the lift station has a firm capacity of 5500 gpm (7.92 mgd). Assuming that excessive I/I will be transported to the WWTF in the future, the maximum monthly ADF will be increased. Assuming an average daily flow of 7.92 mgd on the day of a rainfall event, the average monthly ADF for the three months with highest reported ADF (February 1992, January 1993, and May 1994) would be 2.936 mgd.

Design ADF = (Maximum monthly ADF)(111%) *(Allowance for TNRCC 75/90 rule)*

Design ADF = 3.26 mgd *(2.936 mgd x 111%)*

Two hour peak (based on flow recorder charts showing peak flows over a recent 12 month period; highest 2 hr. peak occurred 9-1-94) = 7.5 mgd

Two hour peak (for design) should be **18.7 mgd**, based on flow modelling of incoming interceptors.

B. 15 Year Projections (2009)

City sewered population projected at 17,991, including 11,962 in South Plant service area.

Annual average ADF = 1.29 mgd *(Based on following methodology:*

37% of annual wastewater flows are comprised of storm water as reported by City on TWDB water conservation forms. Increase both residential and nonresidential return flows in proportion to population.

Increase storm flows at 25% the rate of residential growth, since any future sewer extensions will be relatively watertight.)

Deduct 2% from future return flows because of water conservation measures.)

Existing 1.285 mgd = 63% return flows = 0.810 mgd

37% storm flows = 0.475 mgd

11,962 ÷ 11,679 = 1.0242 (2.42% increase)

0.81 mgd x 1.0242 = 0.830 mgd

0.475 mgd x [1 + (.0242 x 0.25)] = 0.478 mgd

Deduct 2% of 0.83 mgd = (-) 0.017 mgd

NET TOTAL = 1.29 1mgd

Maximum monthly ADF = 3.28 mgd *(Increasing in proportion to average annual ADF)*

3.26 mgd x (1.291 ÷ 1.285) = 3.28 mgd

Two hour peak (for design) should be **18.7** mgd

C. 30 Year Projections (2024)

City population projected at 18,394, including 12,230 in South Plant service area.

Annual average ADF = 1.311 mgd *(Similar to 15 year projections)*

12,230 ÷ 11,679 = 1.0472 (4.72% increase)

.81 mgd x 1.0472 = .848 mgd

0.475 mgd x [1 + (.0472 x 0.15)] = 0.481 mgd

Deduct 2% of 0.848 mgd = (-) 0.017 mgd

NET TOTAL = 1.312 mgd

Maximum monthly ADF = 3.33 mgd *(Increasing in proportion to average annual ADF)*

3.26 x (1.312 ÷ 1.285) = 3.33 mgd

DERIVATION OF DESIGN FLOWS.

The Engineers feel that the methods which we have used in Appendix B are adequate for the scope of a regional planning study as distinguished from a detailed engineering plan. In the case of this study, they also represent the most realistic approach given the prevailing physical conditions in the collection systems and the available data.

Industrial flows are inapplicable to all three of the cities. Although at least two of the cities (Nederland and Port Neches) provide wastewater service to nearby industries, this service is for domestic flows only, with any process wastewater treated by the industries or by others.

The flow calculations show a segregation of infiltration/inflow from return flows. The City of Groves reported that approximately 37% of its total annual plant flows were composed of I/I. This figure was apparently based on past engineering studies such as those performed for the Construction Grants Program in the late 1970's. This figure looks reasonable in comparison with similar figures for other communities in the area with significant I/I problems.

Since all of the cities are primarily residential in nature, it is reasonable to assume that return flows will increase in proportion to population. Infiltration/inflow, however, can be expected to show a lesser rate of increase, since future growth will be served either by existing collection lines or by new extensions which will be relatively watertight.

For deriving peak flow rates, I/I flows could be addressed in a number of ways. A typical method used by Schaumburg & Polk, Inc. in Construction Grants projects was to estimate the total potential flow from the individual leaking segments *(without regard to lack of transportation capacity)*. A program of selected rehabilitation, based on cost effective considerations, was then developed and the amount of residual I/I flows estimated.. This process required an extensive sewer survey, including manhole inspection, smoke testing, possible television inspection, and quantification, followed by a thorough analysis. All three cities went through that process not many years back in the Construction Grants Program. Like many communities, they carried out the recommended program of system rehabilitation, only to find it much less successful than predicted.

The experience from the Construction Grants Program indicates that in many cases, the collection system is subject to so many sources of I/I flows that it cannot transport them to the plant. This is especially true in flat coastal areas where the gravity lines are laid at a minimum slope with limited conveyance, and at the same time are subject to continual shifting of the expansive soils in which they are laid. To a large extent, elimination of the major leaks can simply make room for I/I flows from other points throughout the system. It appears that this is what happened to all three cities.

In the absence of a sewer survey which would be far beyond the scope of a regional planning study, the quantity of peak I/I cannot be readily estimated. An attempt was made in most cases to estimate total peak flows on the basis of plant flow records. However, this method would tend to underestimate potential flows because of deficient transportation capacity in the system. All three cities have serious problems with surcharging and system overflows, but no reliable data is available to quantify this problem.

In the experience of the Engineer, flows from systems overloaded with I/I problems are likely to be underestimated. There is a serious danger in expanding a plant and trunk lines to handle estimated flows, then finding that the facilities are still overloaded. In the absence of extensive flow monitoring, particularly under prevailing local conditions, it is not unreasonable to expect the collection systems to be loaded to capacity during peak storm conditions.

The use of a two year storm event (5.5") for I/I calculations, even if the flows could be readily determined, would result in serious underdesign in light of the periodic storm events which exceed that amount by a factor of two or more.

In the course of the Construction Grants Program, it was learned that communities can expect to achieve only a limited quantity of I/I reduction, and this quantity is difficult to predict. Experience also indicates that I/I is a recurring problem, and that even a continuing maintenance program will leave a substantial amount of I/I. Considering the extensive rehabilitative efforts which have already been made, and the limited success of these efforts, it is unreasonable to expect the problem to be reduced substantially through additional work short of total system replacement.

The only collection system replacement or rehabilitation specifically recommended in the report is in certain sections of Nederland. This work is for the purpose of eliminating overflow conditions and may not significantly reduce flows to the plant.

The flow projections in Appendix B include a reduction in per capita return flows from water conservation measures. However, it is unrealistic to expect any significant reduction in water usage under present circumstances in Southeast Texas.

<u>In summary, we feel that the methodology used in the report for flow calculations is appropriate.</u>

APPENDIX C - Wastewater Treatment Alternatives

N1 - City of Nederland:	Upgrade Existing WWTF Activated Sludge 5/5
N2 - City of Nederland	Upgrade Existing WWTF, Divert Discharge to the Neches River Activated Sludge 10/15
N3 - City of Nederland:	Operate Existing WWTF, Add Constructed Wetland, Divert Discharge to Rhodair Gully (or Johns Gully)
N4 - City of Nederland:	Utilize Existing Treatment Plant, Construct Wetland w/Star Enterprise
N5 - City of Nederland:	Abandon Existing WWTF, Construct Lagoon/Wetland Treatment System, Discharge into Rhodair Gully or Johns Gully
N6 - City of Nederland:	Upgrade Existing WWTF, 4.76 mgd. Divert Discharge to the Neches River

General Discussion

N1 - City of Nederland: Upgrade Existing WWTF Activated Sludge 5/5

Upgrade the existing the existing WWTF for continued discharge into the existing receiving stream at the following effluent limits.

ADF	=	5.22 mgd
2-Hour Peak Flow	=	18,125 gpm (26.10 mgd)
BOD_5	=	5 mg/l
TSS	=	5 mg/l
NH_3	=	2 mg/l
D.O.	=	6 mg/l

A. <u>Preliminary Treatment</u> (Before splitting into tracks)

1. <u>Screening</u>

Existing: One mechanical bar screen, 7 ft. 5 in. ± length, 5 ft. total width, 30 bars, 3/8 in. width on 1 3/8" centers, with mechanical cleaning mechanism; design liquid depth 6 ft. maximum.

Required: Some form of screening; bar openings minimum ½" for mechanical screens; velocities @ design flow minimum 2 ft./sec through channel, < 3 ft./sec. through screen.

Analysis: Bar openings = 1"

Design Flow = 5.22 mgd = 8.08 cfs

Channel Velocity = 8.08 cfs / (5 ft. x 6 ft.)
= 0.27 fps

Screen Velocity = 8.08 cfs / (30 x 1/12 ft. x 6 ft.)
= 0.54 fps

Improvements: NONE

2. <u>Influent Lift Station</u>.

Existing: Four pumps, submersible type, installed in dry pit, each 2900 gpm capacity for firm capacity of 8700 gpm. (Two of the pumps are two speed with a slower speed of 900 gpm, with pump speed automatically adjusted as a function of wet well level. The 2900 gpm rated capacities are based on an average pumping head between high and low wet well levels.)

Required: *Firm pumping capacity (largest pump out of service) must be adequate to pump peak flow to destination; three or more pumps (or duplex pumps with automatic variable speed control) required.*

Analysis: Existing firm capacity = 8700 gpm = 12.528 mgd peak flow.
Proposed firm capacity = 18,125 gpm = 26.10 mgd peak flow.

Improvements: Upgrade pumping firm capacity of lift station to 18,125 gpm. Replace three of the existing pumps with 7613 gpm pumps.

3. Aerated Grit Chamber

Existing: 20 ft. x 20 ft. chamber, 13 ft. water depth (less 5 ft. x 12 ft. x 4.5 ft. splitter box for effluent), plus hopper bottom with 1:1 slope (reported basin volume of 6240 ft^3); two air diffusers (112 cfm total) with 30" draft tube; concentrated grit/liquid mixture sent to degritter for final grit separation.

Required: *Grit removal recommended; if removal units are provided, must have method of removing grit from unit, and any unit with single chamber must have bypass.*

Analysis: Grit removal by grit pump below; piping allows flow to bypass grit chamber if needed. This unit also provides preaeration.

Improvements: NONE

4. Grit Pump

Existing: One vortex type pump, 250 gpm (pumps grit/liquid mixture from aerated grit chamber to degritter).

Improvements: NONE

5. Degritter

Existing: Hydrocyclone (10.5 ft. long) and grit classifier/washer (L shaped, approx. 5 ft. x 25 ft. plus 4 ft. x 3 ft. (dewaters grit from aerated grit chamber).

Improvements: NONE

B. Activated Sludge Process. Construct new activated sludge aeration basins.

Required: *Total volume shall be 1000 ft^3 per 35 lb. BOD_5/day. Diffused aeration shall be designed for 3200 SCF per lb. BOD_5. The diffuser system must be capable of providing 150% of design requirements.*

Analysis: lbs BOD_5/day = (5.22 MGD)(8.345)(200 mg/l) = 8712 lbs BOD_5/day

MAX BOD_5 LOAD	= 30 lb. BOD_5/day/1000 ft^3 (Conservative loading based on Engineer's experience)
BOD_5 Loading	= (8,712 lb BOD_5/day)/(30 lb BOD_5/day/1000 ft^3 = 290,400 ft^3
Each Unit	= (290,400 ft^3)/2 = 145,200 ft^3 = (145,200 ft^3)/(22 ft SWD) = 6600 ft^2 = 81 ft x 81ft
Air Requirements	= (8,712 lb. BOD_5/day)(3200 SCFM/lb. BOD_5) = 27,878,400 SCFM/day = 19,360 cfm
150% of Air Req.	= 19,360 cfm (1.5) = 29,040 cfm

Improvements: **Construct two deep tank type aerators (81 ft x 81 ft x 22 ft SWD) and provide 29,040 cfm aeration equipment capacity.**

C. Final Clarifiers. Construct new final clarifiers.

Existing: Two (2) at 60 ft. diam. x 10 ft. side water depth, to be converted to aeration units.

Required: *Maximum surface loading at Peak flow of 1200 gal./day/ft^2, and at Design flow of 600 gal./day/ft^2. Side water depth must be at least 10 ft. for surface areas of 1250 ft^2 or more. Effective detention times (based on liquid volume above a 3 ft. sludge blanket) must be 1.5 hr. @ Peak flow and 3.0 hr. @ Design flow.*

Analysis: Required Area based on surface area:

@ Peak Flow	= 26,100,000 gpd/1200 gal/day/ft^2 = 21,750 ft^2
@ Design Flow	= 5,220,000 gpd/600 gal/day/ft^2 = 8,700 ft^2

Required Area based on detention time:
Detention time is based on side water depth of 14 ft less 3 ft. sludge blanket.

@ Peak flow = (26,100,000 gpd)(1.5 hrs.)/[(24 hrs./day)(7.48 gal/ft^3)(11 ft)] = 19,825 ft^2

@ Design flow = 5,220,000 gpd)(3.0 hr.)/[(24 hrs./day)(7.48 gal/ft^3)(11 ft)] = 7,930 ft^2

New clarifier(s) required based on surface area (and feedwell area):

Surface Area of 14' DIA Feedwell = 154 ft^2

Surface Area Required = Surface Area @ Peak + Surface Area of Feedwell

Surface Area = 21,750 ft^2 + 154 ft^2 = 21,904 ft^2

Improvements: **Construct two (2) new 118 ft. diameter clarifiers with 14 ft. side water depth required (based on minimum required surface area). Provide flow splitting/collection structures/ piping, and sludge collection/pumping.**

D. Filtration. Construct a tertiary filter to reduce effluent TSS to required 5 mg/l.

Existing: NONE

Required: *Filtration must be employed as a unit operation to supplement suspended solids removal for those treatment facilities with tertiary effluent limits. Design filtration rates shall not exceed 3 gpm/ft^2 for single media filters, 4 gpm/ft^2 for dual media filters, and 5 gpm/ft^2 for mixed media filters. There shall be a minimum of two units and the required filter area shall be calculated with one unit out of service.*

Analysis: Assuming dual media filters = 18,125 gpm / 5 gpm/ft^2
= 3,625 ft^2

Assuming filtration is provided by three (3) 35 ft. x 35 ft. units with a fourth unit out of service. 3 (35' x 35') = 3,675 ft^2

Improvements: **Construct four (4) 35 ft. x 35 ft. mixed media tertiary filters.**

E. Effluent Works.

1. Chlorine Contact Chamber.

Existing: Inside dimensions 98 ft. 6 in. x 38 ft. 2 in. including partitions and baffles; minimum liquid depth 6.25 ft. (6 ft. in final compartment); hopper bottoms in two 18.5 ft. x 18.75 ft. portions of chamber; fine bubble diffusers for mixing.

Required: *Detention time of 20 minutes @ peak flow.*

Analysis: Existing volume approximately 23,000 ft^3
23,000 ft^3/20 min. = 1150 cfm = 19.2 cfs = 12.39 mgd

Additional volume required = 13.71 mgd @ 20 minutes
= 25,455 ft^3

Improvements: Construct a second, parallel chlorine contact chamber with an effective volume of 25,455 ft^3.

2. Chlorine Feed Equipment.

Existing: Two systems, each 500 lb./day feed capacity (vacuum operated) including one standby; flow proportioned; chlorine gas from one ton size containers.

Required: *Feed equipment must be able to provide more than the highest dosage to be required at any time. Dosage must be adequate to maintain a chlorine residual of at least 1 mg/l after 20 minutes detention, prior to dechlorination.*

Analysis: In standard practice, feed equipment is designed to feed 10 ppm of Cl_2 in order to assure a 1 mg/l residual.

(500 lb./day)/(10 ppm)(8.345 lb./gal.) = 5.99 mgd
If both feeders can be used simultaneously during peak flows, they would have a theoretical capacity for 11.98 mgd peak.

Improvements: **Provide additional chlorine feed equipment as necessary to provide for chlorination of 26.10 mgd.**

3. Dechlorination.

Existing: Liquid ammonium bisulfate, 3000 gal. storage tank, one metering pump with 96 gal./day capacity; injection and reaction occur in a transitional area between chlorination and flow measurement. This dechlorination area is structurally an extension of the chlorine contact chamber, 8 ft. x 10 ft. 8 in. rectangle plus an adjacent trapezoidal area, 5 ft. long, width transitional from 8 ft. to 3 ft. 11 in.

Required: *The effluent, after chlorination and 20 minutes detention time, must be dechlorinated to less than 0.1 mg/l. For most dechlorination agents, 1 minute detention is generally considered adequate.*

Analysis: 26,10 MGD = 40.38 cfs = 2,423 cfm
2,423 cfm (1 min.) = 2,423 ft^3

Improvements: **Construct a new 2,423 ft^3 dechlorination chamber downstream of the existing and proposed chlorine contact chambers.**

4. Flow Measurement.

Existing: 24 inch parshall flume; continuously indicating, recording, and totalizing flow meter calibrated to read up to 15 mgd.

Required: *Continuous effluent measurement required, with capacity for maximum expected peak flow.*

Analysis: Existing effluent measurement is not adequate for peak flows up to 26.10 mgd.

Improvements: **Construct a new parshall flume with continuous flow recorder capable of measuring up to 30 mgd.**

5. Postaeration.

Existing: Postaeration is accomplished by a cascading effect as the effluent drops from the flow measurement device to the effluent line.

Improvements: **Enhance existing passive aeration and/or provide mechanical postaeration as necessary to achieve required 6.0 mg/l effluent dissolved oxygen.**

F. Sludge Processing.

1. Sludge Thickener.

Existing: 38 ft. diam. x 14 ft. side water depth, bottom slopes to center @ 4:1; mechanical sludge collection with pickets; supernatant to Trickling

Filter No. 1.

Required: *Aerobic digesters should be provided with sludge thickening.*

Improvements: **Supernatant from thickener shall be diverted back to head of the plant.**

2. Aerobic Digesters. Convert the existing aeration units (contact and stabilization), the existing aerobic digester, and the clarifiers within the two (2) existing contract stabilization plants into aerobic digesters.

Requirement: *Minimum solids retention time of 15 days (may be calculated as 20 ft^3 for each lb. influent BOD_5 per day). Diffused air requirement is 30 cfm per 1000 ft^3 of volume.*

Analysis: Digester Volume Required = 20 ft^3 / lb.BOD_5 / day
lb. BOD_5/day = (5.22 mgd)(8.345)(200 mg/l) = 8712 lb. BOD_5/day

Required Digester Volume = (8712 lb. BOD_5/day)(20 ft^3/lb. BOD/day) = 174,244 ft^3

Existing Units: $[\pi(100 \text{ ft.})^2/4](15 \text{ ft.}) = 117{,}809 \text{ ft}^3$ each
Total Volume $= 2 \times 117{,}809 \text{ ft}^3 = 235{,}618 \text{ ft}^3 > 174{,}244 \text{ ft}^3$

Required Aeration = (30 cfm/1000 ft^3)(235,618 ft^3) = 7,069 cfm

Improvements: **Convert two (2) existing contact stabilization plants into aerobic digesters and provide 7,069 cfm aeration equipment capacity.**

3. Centrifuge Facility.

Existing: One sludge grinder; two sludge metering pumps, progressive cavity, 60 gpm; one polymer feed pump (for 6% solution); two 200 gallon polymer mixers; one polymer metering pump; one horizontal centrifuge, 60 gpm with 20 hp motor and mixing tank to introduce polymer into sludge.

Improvements: **Provide additional sludge dewatering facilities.**

4. Drying Beds.

Existing: Two sets of open sand beds, 76 ft. x 220 ft. and 50 x 100 ft.; used for standby only.

G. Blowers.

Existing: Four blowers, 1500 cfm each, existing firm capacity = 4500 cfm.

Required: Blowers must be able to meet maximum aeration requirements with largest unit out of service.

Improvements: Provide additional blowers as necessary.

H. Opinion of Probable Cost

1.	Upgrade influent lift station	$ 210,000
2.	Construct activated sludge aeration basins	$ 1,411,000
3.	Construct two additional final clarifiers	$ 1,662,000
4.	Construct tertiary filters	$ 2,192,000
5.	Additional chlorine contact/dechlorination facilities	$ 317,000
6.	Flow measurement/post-aeration	$ 75,000
7.	Convert existing contact stabilization units into aerobic sludge digesters	$ 125,000
8.	Additional sludge dewatering facilities	$ 440,000
9.	Additional aeration blower equipment	$ 740,000
10.	Yard piping improvements	$ 500,000
11.	Miscellaneous site work	$ 200,000
12.	Laboratory/Office	$ 75,000
13.	Electrical and instrumentation	$ 750,000
	Subtotal	$ 8,697,000
	Contingency (15%)	$ 1,305,000
	TOTAL OPINION OF PROBABLE CONSTRUCTION COST	**$ 10,002,000**

N2 - City of Nederland: **Upgrade Existing WWTF, Divert Discharge to the Neches River Activated Sludge 10/15**

Upgrade the existing the existing WWTF and divert the discharge to the Neches River at the following effluent limits.

ADF	=	5.22 mgd
2-Hour Peak Flow	=	18,125 gpm (26.10 mgd)
BOD_5	=	20 mg/l
TSS	=	20 mg/l
NH_3	=	no limit
D.O.	=	4 mg/l

A. Preliminary Treatment (Before splitting into tracks)

1. Screening

Existing: One mechanical bar screen, 7 ft. 5 in. ± length, 5 ft. total width, 30 bars, 3/8 in. width on 1 3/8" centers, with mechanical cleaning mechanism; design liquid depth 6 ft. maximum.

Required: Some form of screening; bar openings minimum ½" for mechanical screens; velocities @ design flow minimum 2 ft./sec through channel, < 3 ft./sec. through screen.

Analysis: Bar openings = 1"

Design Flow = 5.22 mgd = 8.08 cfs

Channel Velocity = 8.08 cfs / (5 ft. x 6 ft.)
= 0.27 fps

Screen Velocity = 8.08 cfs / (30 x 1/12 ft. x 6 ft.)
= 0.54 fps

Improvements: NONE

2. Influent Lift Station.

Existing: Four pumps, submersible type, installed in dry pit, each 2900 gpm capacity for firm capacity of 8700 gpm. (Two of the pumps are two speed with a slower speed of 900 gpm, with pump speed automatically adjusted as a function of wet well level. The 2900 gpm rated capacities are based on an average pumping head between high and low wet well levels.)

Required: *Firm pumping capacity (largest pump out of service) must be adequate to pump peak flow to destination; three or more pumps (or duplex pumps with automatic variable speed control) required.*

Analysis: Firm capacity = 8700 gpm = 12.528 mgd peak flow.

Improvements: **Upgrade pumping firm capacity of lift station to 18,125 gpm. Replace three of the existing pumps with 7613 gpm pumps.**

3. Aerated Grit Chamber

Existing: 20 ft. x 20 ft. chamber, 13 ft. water depth (less 5 ft. x 12 ft. x 4.5 ft. splitter box for effluent), plus hopper bottom with 1:1 slope (reported basin volume of 6240 ft^3); two air diffusers (112 cfm total) with 30" draft tube; concentrated grit/liquid mixture sent to degritter for final grit separation.

Required: *Grit removal recommended; if removal units are provided, must have method of removing grit from unit, and any unit with single chamber must have bypass.*

Analysis: Grit removal by grit pump below; piping allows flow to bypass grit chamber if needed. This unit also provides preaeration.

Improvements: **NONE**

4. Grit Pump

Existing: One vortex type pump, 250 gpm (pumps grit/liquid mixture from aerated grit chamber to degritter).

Improvements: **NONE**

5. Degritter

Existing: Hydrocyclone (10.5 ft. long) and grit classifier/washer (L shaped, approx. 5 ft. x 25 ft. plus 4 ft. x 3 ft. (dewaters grit from aerated grit chamber).

Improvements: **NONE**

B. <u>Activated Sludge Process</u>. Construct new activated sludge aeration units.

Required: *Total volume shall be 1000 ft^3 per 35 lb. BOD_5/day. Diffused aeration shall be designed for 3200 SCF per lb. BOD_5. The diffuser system must be capable of providing 150% of design requirements.*

Analysis: lbs. BOD_5/day = (5.22 mgd)(8.345)(200 mg/l) = 8712 lbs. BOD_5/day

Max BOD_5 Load	= 30 lb BOD_5/day/1000 ft^3 (Conservative loading)
BOD_5 Loading	= (8712 lb BOD_5/day)/(30 lb. BOD_5/day/1000 ft^3) = 290,400 ft^3
Each Unit	= 290,400 ft^3 / 2 = 145,200 ft^3 = 145,200 ft^3/(22 ft SWD) = 6,600 ft^3 = 81 ft x 81 ft
Air Requirements	= (8,712 lb. BOD_5/day)(3,200 SCFM/lb. BOD_5) = 27,878,400 SCFM/day = 19,360 cfm
150% Air Req.	= (19,360 cfm)(1.5) = 29,040 cfm

Improvements: Construct two deep tank type aerators (81 ft x 81 ft x 22 ft SWD) and provide 29,040 cfm aeration equipment capacity

C. <u>Final Clarifiers</u>. Construct new final clarifiers.

Existing: Two (2) at 60 ft. diam. x 10 ft. side water depth, to be converted to digesters.

Required: *Maximum surface loading at Peak flow of 1200 gal./day/ft^2, and at Design flow of 600 gal./day/ft^2. Side water depth must be at least 10 ft. for surface areas of 1250 ft^2 or more. Effective detention times (based on liquid volume above a 3 ft. sludge blanket) must be 1.5 hr. @ Peak flow and 3.0 hr. @ Design flow.*

Analysis: <u>Required Area based on surface area</u>:

@ Peak Flow	= 26,100,000 gpd/1200 gal/day/ft^2 = 21,750 ft^2
@ Design Flow	= 5,220,000 gpd/600 gal/day/ft^2 = 8,700 ft^2

<u>Required Area based on detention time</u>:

@ Peak Flow = (26,100,000 gpd)(1.5 hrs.)/[(24 hrs./day)(7.48 gal/ft^3)(11 ft)]
= 19,825 ft^2

@ Design Flow = (5,200,000)(3.0 hrs)/[(24 hrs./day)(7.48 gal/ft^3)(11 ft)]
= 7,930 ft^2

New clarifier(s) required based on surface area (and feedwell area):

Surface Area of 14′ feedwell = 154 ft^2

Surface Area Required = Surface Area @ Peak + Surface Area of Feedwell

Surface Area Required = 21,750 + 154 = 21,904 ft^2

Improvements: **Construct two (2) new 118 ft. diameter clarifiers with 14 ft. side water depth required (based on minimum required surface area). Provide flow splitting/collection structures/piping, and sludge collection/pumping.**

D. Effluent Works.

1. Chlorine Contact Chamber.

Existing: Inside dimensions 98 ft. 6 in. x 38 ft. 2 in. including partitions and baffles; minimum liquid depth 6.25 ft. (6 ft. in final compartment); hopper bottoms in two 18.5 ft. x 18.75 ft. portions of chamber; fine bubble diffusers for mixing.

Required: *Detention time of 20 minutes @ peak flow.*

Analysis: Existing volume approximately 23,000 ft^3
23,000 ft^3/20 min. = 1150 cfm = 19.2 cfs = 12.39 mgd

Additional volume required = 13.71 mgd @ 20 minutes
= 25,455 ft^3

Improvements: **Construct a second, parallel chlorine contact chamber with an effective volume of 25,455 ft^3.**

2. Chlorine Feed Equipment.

Existing: Two systems, each 500 lb./day feed capacity (vacuum operated) including one standby; flow proportioned; chlorine gas from one ton size containers.

Required: *Feed equipment must be able to provide more than the highest* *dosage to be required at any time. Dosage must be adequate to maintain a chlorine residual of at least 1 mg/l after 20 minutes detention, prior to dechlorination.*

Analysis: In standard practice, feed equipment is designed to feed 10 ppm of Cl_2 in order to assure a 1 mg/l residual.

(500 lb./day)/(10 ppm)(8.345 lb./gal.) = 5.99 mgd
If both feeders can be used simultaneously during peak flows, they would have a theoretical capacity for 11.98 mgd peak.

Improvements: **Provide additional chlorine feed equipment as necessary to provide for chlorination of 26.10 mgd.**

3. Dechlorination.

Existing: Liquid ammonium bisulfate, 3000 gal. storage tank, one metering pump with 96 gal./day capacity; injection and reaction occur in a transitional area between chlorination and flow measurement. This dechlorination area is structurally an extension of the chlorine contact chamber, 8 ft. x 10 ft. 8 in. rectangle plus an adjacent trapezoidal area, 5 ft. long, width transitional from 8 ft. to 3 ft. 11 in.

Required: *The effluent, after chlorination and 20 minutes detention time, must be dechlorinated to less than 0.1 mg/l. For most dechlorination agents, 1 minute detention is generally considered adequate.*

Analysis: 26.10 mgd = 40.38 cfs = 2,423 cfm
2,423 cfm (1 min) = 2,423 ft^3

Improvements: **Construct a new 2,423 ft^3 dechlorination chamber downstream of the existing and proposed chlorine contact chambers.**

4. Flow Measurement.

Existing: 24 inch parshall flume; continuously indicating, recording, and totalizing flow meter calibrated to read up to 15 mgd.

Required: *Continuous effluent measurement required, with capacity for maximum expected peak flow.*

Analysis: Existing effluent measurement is not adequate for peak flows up to 26.10 mgd.

Improvements: **Construct a new parshall flume with a continuous flow recorder capable of measuring up to 30 mgd.**

5. Postaeration.

Existing: Postaeration is accomplished by a cascading effect as the effluent drops from the flow measurement device to the effluent line.

Improvements: **Enhance existing passive aeration and/or provide mechanical postaeration as necessary to achieve required 4.0 mg/l effluent dissolved oxygen.**

E. Effluent Lift Station. Construct an effluent lift station to pump the effluent flows from the existing WWTF to the Neches River.

Required: *Firm pumping capacity (largest pump out of service) must be adequate to pump peak flow to destination; three or more pumps (or duplex pumps with automatic variable speed control) required.*

Analysis: Firm capacity = 18,125 gpm

Five (5) pumps with firm capacity of 18,125 gpm with largest pump out of service.

Effluent force mains sized for a minimum of 2 fps at low flow and a maximum 10 fps at peak flow.

Install two (2) force mains, one for ADF and one as a Peak Flow force main. Proposed ADF force main = 18″ diameter and proposed peak flow force main = 20″ diameter.

Improvements: **Construct an effluent lift station with five (5) pumps (firm capacity of 18,125 gpm) and a dual 18″/30" diameter force main to the Neches River.**

F. Sludge Processing.

1. Sludge Thickener.

Existing: 38 ft. diam. x 14 ft. side water depth, bottom slopes to center @ 4:1; mechanical sludge collection with pickets; supernatant to Trickling Filter No. 1.

Required: *Aerobic digesters should be provided with sludge thickening.*

Improvements: **Supernatant from thickener shall be diverted back to head of the plant.**

2. Aerobic Digesters. (See N1)

Requirement: *Minimum solids retention time of 15 days (may be calculated as 20 ft^3 for each lb. influent BOD_5 per day). Diffused air requirement is 30 cfm per 1000 ft^3 of volume.*

Analysis: Digester Volume Required = 20 ft^3 / lb. BOD_5 per day
lb. BOD_5/day = (5.22 mgd)(8.345)(200 mg/l) = 8712 lb. BOD_5/day

Required Digester Volume = (8712 lb. BOD/day)(20 ft^3/lb. BOD/day) = 174,244 ft^3

Existing Units: = $[\pi(100 \text{ ft.})^2/4]$ (15 ft.) = 17,809 ft^3 each
Total Volume: = 2 x 117,809 ft^3 = 235,618 ft^3 > 174,244 ft^3

Required Aeration = (30 cfm/1000 ft^3)(235,618 ft^3) = 7069 cfm

Improvements: **Convert two (2) existing contact stabilization plants into aerobic digesters and provide 7,069 aeration equipment capacity.**

3. Centrifuge Facility.

Existing: One sludge grinder; two sludge metering pumps, progressive cavity, 60 gpm; one polymer feed pump (for 6% solution); two 200 gallon polymer mixers; one polymer metering pump; one horizontal centrifuge, 60 gpm with 20 hp motor and mixing tank to introduce polymer into sludge.

Improvements: **Provide additional sludge dewatering facilities.**

4. Drying Beds.

Existing: Two sets of open sand beds, 76 ft. x 220 ft. and 50 x 100 ft.; used for standby only.

G. Blowers.

Existing: Four blowers, 1500 cfm each, existing firm capacity = 4500 cfm.

Required: *Blowers must be able to meet maximum aeration requirements with largest unit out of service.*

Improvements: **Provide additional blowers as necessary.**

H. Opinion of Probable Cost

1.	Upgrade influent lift station	$	210,000
2.	Construct activated sludge aeration basins	$	1,411,000
3.	Construct two additional final clarifiers	$	1,662,000
4.	Additional chlorine contact/dechlorination facilities	$	317,000
5.	Flow measurement/post-aeration	$	75,000
6.	Construct effluent lift station/force mains	$	4,037,000
7.	Convert existing contact stabilization units into aerobic sludge digesters	$	125,000
8.	Additional sludge dewatering facilities	$	440,000
9.	Additional aeration blower equipment	$	740,000
10.	Yard piping improvements	$	500,000
11.	Miscellaneous site work	$	200,000
12.	Laboratory/Office	$	75,000
13.	Electrical and instrumentation	$	750,000
	Subtotal	$	10,102,000
	Contingency (15%)	$	1,515,000
	TOTAL OPINION OF PROBABLE CONSTRUCTION COST	$	11,617,000

In analyzing Alternate N2 it was assumed that the TNRCC would require that all flows be diverted to the Neches River. During wet weather the existing receiving stream may have adequate flow to receive a discharge at less stringent quality standards. Therefore, if the TNRCC would allow all flows up to the permitted ADF from each treatment facility to be diverted to the Neches River and allow peak flow in excess of the permitted ADF to continue to be discharged to the existing receiving stream at current 20/20 limits, then the cost of this alternative could be significantly reduced.

Opinion of Probable Cost

1.	*Upgrade influent lift station*	*$*	*210,000*
2.	*Construct activated sludge aeration basins*	*$*	*1,411,000*
3.	*Construct two additional final clarifiers*	*$*	*1,662,000*
4.	*Additional chlorine contact/dechlorination facilities*	*$*	*317,000*
5.	*Flow measurement/post-aeration*	*$*	*75,000*
6.	*Construct effluent lift station/force mains*	*$*	*1,659,000*
7.	*Convert existing contact stabilization units into aerobic sludge digesters*	*$*	*125,000*
8.	*Additional sludge dewatering facilities*	*$*	*440,000*
9.	*Additional aeration blower equipment*	*$*	*740,000*
10.	*Yard piping improvements*	*$*	*500,000*
11.	*Miscellaneous site work*	*$*	*200,000*
12.	*Laboratory/Office*	*$*	*75,000*
13.	*Electrical and instrumentation*	*$*	*750,000*
	Subtotal	*$*	*7,724,000*
	Contingency (15%)	*$*	*1,159,000*
	TOTAL OPINION OF PROBABLE CONSTRUCTION COST	**$**	***8,883,000***

N3 - City of Nederland: Operate Existing WWTF, Add Constructed Wetland, Divert Discharge to Rhodair Gully

Continue to operate existing WWTF, construct a transfer lift station to pump the WWTF effluent to a constructed surface flow wetland to polish the effluent from the existing WWTF and then discharge into Rhodair Gully (or Johns Gully) at the following effluent limits.

ADF	=	5.22 mgd
2-Hour Peak Flow	=	18,125 gpm (26.10 mgd)
BOD_5	=	10 mg/l
TSS	=	15 mg/l
NH_3	=	3 mg/l
D.O.	=	6 mg/l

A. Existing WWTF. Internal plant piping and hydraulic through existing WWTF will need to be analyzed and upgraded as necessary to provide for 26.10 mgd 2-Hour peak flow. Also, will require TNRCC approval to re-rate existing WWTF for a reduced efficiency at the higher flows. Assume existing WWTF will consistently produce a 60 mg/l BOD_5, 60 mg/l TSS and 20 mg/l NH_3 effluent at 5.22 mgd ADF.

Improvements: **Upgrade existing influent lift station as necessary to provide for 26.10 mgd peak flow (see Alternate N1).**

Upgrade existing chlorination/dechlorination system to provide for disinfection of 26.10 mgd (see Alternate N1).

B. Transfer Lift Station. Construct a lift station to transfer the effluent flows from the existing WWTF to the proposed constructed wetland.

Required: *Firm pumping capacity (largest pump out of service) must be adequate to pump peak flow to destination; three or more pumps (or duplex pumps with automatic variable speed control) required.*

Analysis: Firm capacity = 18,125 gpm

Five (5) pumps with firm capacity of 18,125 gpm with largest pump out of service.

Transfer force main sized for a minimum of 2 fps at low flow and a maximum 10 fps at peak flow.

Install two (2) force mains, one for ADF and one as a Peak Flow force main. Proposed ADF force main = 18″ diameter and proposed peak flow force main = 20″ diameter.

Improvements: **Construct a transfer lift station with five (5) pumps (firm capacity of 18,125 gpm) and a dual 18"/30" diameter transfer force main to the constructed wetland facility.**

C. Constructed Wetland Facility. Construct a surface flow wetland for polishing the effluent from the existing WWTF and discharge into Rhodair Gully (or Johns Gully).

Required: *Detention time required for a fraction BOD_5 remaining after secondary treatment of 0.30 (i.e. 60 mg/l / 200 mg/l) and a permitted BOD_5 of 10 mg/l is 11 days. In-situ or constructed clay liner or synthetic liner required. Wetland must be protected from a 100-year flood. Berms shall have 3H:1V sideslopes. Multiple cells required, multiple inlets/outlets required. Refer to section 317.15. Appendix G of the TNRCC Design Criteria for Sewerage Systems for additional requirements.*

Analysis: For NH_3 reduction a Marsh-Pond-Marsh configuration will be utilized. Marsh sections will encompass approximately 66% of total area at an average water depth of 8", and Pond section will have an average water depth of 36". Mean water depth across entire wetland will be approximately 17". Assume a porosity for wetland vegetation of 86%.

Per Jefferson County Soil Survey, Series 1960, No. 21, average monthly rainfall and evaporation rates are listed below.

	Average Rainfall	Average Evaporation	Net Contribution
January	4.34"	2.00"	2.34"
February	3.98"	2.32"	1.66"
March	3.25"	3.40"	- 0.15"
April	3.68"	4.27"	- 0.59"
May	4.47"	5.16"	- 0.69"
June	4.44"	5.49"	-1.05"
July	6.56"	5.48"	1.08"
August	5.32"	5.39"	- 0.07"
September	4.73"	4.41"	0.32"
October	3.19"	3.79"	- 0.60"
November	3.61"	2.65"	0.96"
December	5.11"	2.08"	3.03"

$$\frac{(5{,}220{,}000 \text{ gpd})(11 \text{ days})}{(7.48 \text{ gals./ft}^3)(17"/12\text{inch/ft.})(0.86)} = 6{,}300{,}797 \text{ ft}^2$$

$$= 145 \text{ acres}$$

Proposed wetland = 155 acres

Rainfall contribution $= \frac{(3.03")(155 \text{ ac.})(43560 \text{ ft}^2/\text{ac.})(7.48 \text{ gal/ft}^3)}{(12 \text{ inches/ft.})(30 \text{ days/month})}$

= 425,071 gpd

Average flow through wetland = (Influent + Effluent) / 2
= (5,220,000 + 5,645,071) / 2
= 5,432,535 gpd

$$\frac{(155 \text{ ac.})(43560 \text{ ft}^2/\text{ac.})(7.48 \text{ gals./ft}^3)(17"/12 \text{ inch/ft.})(0.86)}{5{,}432{,}535 \text{ gpd}} = 11.3 \text{ days}$$

Determine loading rates based on 155 acres

BOD_5 = (5.22 mgd)(8.345)(60 mg/l) = $\frac{2614 \text{ lbs./day}}{155 \text{ acres}}$

= 16.9 lb./acre-day

TSS = 16.9 lb./acre-day

NH_3 = (5.22 mgd)(8.345)(20 mg/l) = $\frac{871 \text{ lbs./day}}{155 \text{ acres}}$

= 5.6 lb./acre-day

Improvements: **Construct a surface flow wetland with a total treatment area of 155 acres. Total area required for constructed wetlands, including perimeter easements, is approximately 200 acres.**

D. Effluent Works.

1. Post-aeration.

Improvements: **Construct a passive, cascade type aeration structure on the discharge from the constructed wetland capable of producing a 6 mg/l dissolved oxygen effluent.**

2. Flow Measurement.

Required: *Continuous effluent measurement required, with capacity for maximum expected peak flow.*

Analysis: Peak flow = 26.10 mgd + maximum rainfall.

Per 1960 Jefferson County Soils Survey report the wettest month is listed as May, 1946 with a total rainfall of 20.01 inches.

$$\text{Rainfall} = \frac{(20.01")(155 \text{ ac.})(43560 \text{ ft}^2/\text{ac.})(7.48 \text{ gal/ft}^3)}{(12 \text{ inches/ft.})(30 \text{ days/month})}$$

$$= 2{,}807{,}151 \text{ gpd}$$

$$\text{Peak flow} = 26.10 \text{ mgd} + 2.81 \text{ mgd}$$
$$= 28.91 \text{ mgd}$$

Improvements: **Construct a parshall flume flow measurement structure on the discharge from the constructed wetland capable of measuring flows up to 35 mgd.**

E. Opinion of Probable Cost

Proposed Wetland Site 'A' - Located between Highway 69,96,287 and West Port Arthur Road (SPUR 93), adjacent to and on the north side of Rhodair Gully. Availability questionable.

1.	Upgrade influent lift station	$	210,000
2.	Additional chlorine contact/dechlorination facilities	$	317,000
3.	Additional sludge dewatering facilities	$	220,000
4.	Yard piping improvements	$	300,000
5.	Electrical and instrumentation	$	235,000
6.	Laboratory/Office	$	75,000
7.	Construct transfer lift station/force main	$	3,297,000
8.	Constructed wetland system	$	3,000,000
9.	Effluent flow measurement/post-aeration	$	75,000
10.	Land Acquisition	$	400,000
	Subtotal	$	8,129,000
	Contingency (15%)	$	1,219,000
	TOTAL OPINION OF PROBABLE CONSTRUCTION COST	$	**9,348,000**

Proposed Wetland Site 'B' - Located west of West Port Arthur Road (SPUR 93), south of the proposed Federal Prison site, adjacent to Johns Gully. Assumed effluent limits of 10/15/3 for Johns Gully. Availability likely.

1.	Upgrade influent lift station	$	210,000
2.	Additional chlorine contact/dechlorination facilities	$	317,000
3.	Additional sludge dewatering facilities	$	220,000
4.	Yard piping improvements	$	300,000
5.	Electrical and instrumentation	$	235,000
6.	Laboratory/Office	$	75,000
7.	Construct transfer lift station/force main	$	5,617,000
8.	Constructed wetland system	$	3,000,000
9.	Effluent flow measurement/post-aeration	$	75,000
10.	Land Acquisition	$	200,000
	Subtotal	$	10,249,000
	Contingency (15%)	$	11,786,000
	TOTAL OPINION OF PROBABLE CONSTRUCTION COST	$	**11,786,000**

N4 - City of Nederland: Utilize Existing Treatment Plant, Construct Wetland w/Star Enterprise

Continue to operated existing WWTF, construct a transfer lift station to pump the WWTF effluent to a constructed surface flow wetland to polish the effluent from the existing WWTF and then discharge to STAR Enterprise for reuse at the following effluent limits.

	Nederland	STAR Enterprise
ADF	5.22 mgd	10.0 mgd
2-Hour Peak Flow	26.10 mgd	N/A

BOD_5	=	10 mg/l
TSS	=	15 mg/l
NH_3	=	3 mg/l
D.O.	=	6 mg/l

A. Existing WWTF. Internal plant piping and hydraulic through existing WWTF will need to be analyzed and upgraded as necessary to provide for 26.10 mgd 2-Hour peak flow. Also, will require TNRCC approval to re-rate existing WWTF for a reduced efficiency at the higher flows. Assume existing WWTF will consistently produce a 60 mg/l BOD_5, 60 mg/l TSS and 20 mg/l NH_3 effluent at 5.22 mgd ADF.

Improvements: **Upgrade existing influent lift station as necessary to provide for 26.10 mgd peak flow (see Alternate N1).**

Upgrade existing chlorination/dechlorination system to provide for disinfection of 26.10 mgd (see Alternate N1).

B. Transfer Lift Station. Construct a lift station to transfer the effluent flows from the existing WWTF to the proposed constructed wetland.

Required: *Firm pumping capacity (largest pump out of service) must be adequate to pump peak flow to destination; three or more pumps (or duplex pumps with automatic variable speed control) required.*

Analysis: Firm capacity = 18,125 gpm

Five (5) pumps with firm capacity of 18,125 gpm with largest pump out of service.

Transfer force main sized for a minimum of 2 fps at low flow and a maximum 10 fps at peak flow.

Install two (2) force mains, one for ADF and one as a Peak Flow force main.

Proposed ADF force main = 18″ diameter and proposed peak flow force main = 30″ diameter.

Improvements: **Construct a transfer lift station with five (5) pumps (firm capacity of 18,125 gpm) and a dual 18"/30″ diameter transfer force main to the constructed wetland facility.**

C. Constructed Wetland Facility. Construct a surface flow wetland for polishing the effluent from the existing WWTF and discharge into STAR Enterprise/LNVA Canal for industrial reuse at STAR Enterprise.

Required: *Detention time required for a fraction BOD_5 remaining after secondary treatment of 0.30 (i.e. 60 mg/l / 200 mg/l) and a permitted BOD_5 of 10 mg/l is 11 days. In-situ or constructed clay liner or synthetic liner required. Wetland must be protected from a 100-year flood. Berms shall have 3H:1V sideslopes. Multiple cells required, multiple inlets/outlets required. Refer to section 317.15. Appendix G of the TNRCC Design Criteria for Sewerage Systems for additional requirements.*

Analysis: For NH_3 reduction a Marsh-Pond-Marsh configuration will be utilized. Marsh sections will encompass approximately 66% of total area at an average water depth of 8", and Pond section will have an average water depth of 36". Mean water depth across entire wetland will be approximately 17". Assume a porosity for wetland vegetation of 86%.

Per Jefferson County Soil Survey, Series 1960, No. 21, average monthly rainfall and evaporation rates are listed below.

	Average Rainfall	Average Evaporation	Net Contribution
January	4.34"	2.00"	2.34"
February	3.98"	2.32"	1.66"
March	3.25"	3.40"	- 0.15"
April	3.68"	4.27"	- 0.59"
May	4.47"	5.16"	- 0.69"
June	4.44"	5.49"	-1.05"
July	6.56"	5.48"	1.08"
August	5.32"	5.39"	- 0.07"
September	4.73"	4.41"	0.32"
October	3.19"	3.79"	- 0.60"
November	3.61"	2.65"	0.96"
December	5.11"	2.08"	3.03"

$$\frac{(15{,}220{,}000\ \text{gpd})(11\ \text{days})}{(7.48\ \text{gals./ft}^3)(17''/12\text{inch/ft.})(0.86)} = 18{,}371{,}288\ \text{ft}^2$$

$$= 422\ \text{acres}$$

Proposed wetland = 450 acres

$$\text{Rainfall contribution} = \frac{(3.03'')(450\ \text{ac.})(43560\ \text{ft}^2/\text{ac.})(7.48\ \text{gal/ft}^3)}{(12\ \text{inches/ft.})(30\ \text{days/month})}$$

$$= 1{,}234{,}077\ \text{gpd}$$

Average flow through wetland = (Influent + Effluent) / 2
= (15,220,000 + 16,454,077) / 2
= 15,837,038 gpd

$$\frac{(450\ \text{ac.})(43560\ \text{ft}^2/\text{ac.})(7.48\ \text{gals./ft}^3)(17''/12\ \text{inch/ft.})(0.86)}{15{,}837{,}038\ \text{gpd}} = 11.3\ \text{days}$$

Determine loading rates based on 450 acres

$$BOD_5 = (15.22\ \text{mgd})(8.345)(60\ \text{mg/l}) = \frac{7621\ \text{lbs./day}}{450\ \text{acres}}$$

$$= 16.9\ \text{lb./acre-day}$$

TSS = 16.9 lb./acre-day

$$NH_3 = (15.22\ \text{mgd})(8.345)(20\ \text{mg/l}) = \frac{2540\ \text{lbs./day}}{450\ \text{acres}}$$

$$= 5.6\ \text{lb./acre-day}$$

Improvements: **Construct a surface flow wetland with a total treatment area of 450 acres. Total area required for constructed wetlands, including perimeter easements, is approximately 600 acres.**

D. Effluent Works.

1. Post-aeration.

Improvements: **Construct a passive, cascade type aeration structure on the discharge from the constructed wetland capable of producing a 6 mg/l dissolved oxygen effluent.**

2. Flow Measurement.

Required: *Continuous effluent measurement required, with capacity for maximum expected peak flow.*

Analysis: Peak flow = 26.10 mgd + maximum rainfall + $Peak_{STAR}$.

Per 1960 Jefferson County Soils Survey report the wettest month is listed as May, 1946 with a total rainfall of 20.01 inches.

$$\text{Rainfall} = \frac{(20.01'')(450 \text{ ac.})(43560 \text{ ft}^2/\text{ac.})(7.48 \text{ gal/ft}^3)}{(12 \text{ inches/ft.})(30 \text{ days/month})}$$

$$= 8,149,793 \text{ gpd}$$

$$\text{Peak flow} = 26.10 \text{ mgd} + 8.15 \text{ mgd} + Peak_{STAR}$$
$$= 34.25 \text{ mgd} + Peak_{STAR}$$

Improvements: **Construct a parshall flume flow measurement structure on the discharge from the constructed wetland capable of measuring flows up to 125% of total peak flow.**

3. Effluent Lift Station. Construct an effluent lift station to pump effluent flows to STAR Enterprise for reuse. Size lift station to pump 150% ADF flows. Force Main shall be as required by STAR Enterprise.

Required: *Firm pumping capacity (largest pump out of service) must be adequate to pump peak flow to destination; three or more pumps (or duplex pumps with automatic variable speed control) required.*

Analysis: Firm capacity = 15,854 gpm

Five (5) pumps with firm capacity of 15,854 gpm with largest pump out of service.

Transfer force main sized for a minimum of 2 fps at low flow and a maximum 10 fps at peak flow.

Improvements: **Construct an effluent lift station with five (5) pumps (firm capacity of 15,854 gpm) and a transfer force main as required.**

E. Opinion of Probable Cost

Proposed Wetland Site - *Located at intersection of Highway 69 and Highway 73. Availability of required acreage is highly questionable. Possibility of natural wetlands will probably make using this available site unfeasible due to mitigation costs.*

1.	Upgrade influent lift station	$	210,000
2.	Additional chlorine contact/dechlorination facilities	$	317,000
3.	Additional sludge dewatering facilities	$	220,000
4.	Yard piping improvements	$	300,000
5.	Electrical and instrumentation	$	235,000
6.	Laboratory/Office	$	75,000
7.	Construct transfer lift station/force main	$	3,603,000
8.	Constructed wetland system*	$	3,090,000
9.	Effluent flow measurement/post-aeration*	$	100,000
10.	Effluent lift station	$	390,000
11.	Land Acquisition*		
	Subtotal	$	8,955,000
	Contingency (15%)	$	1,345,000
	TOTAL OPINION OF PROBABLE CONSTRUCTION COST	$	10,300,000

* *(34.29% of total costs. City of Nederland ADF / Total ADF)*

N5 - City of Nederland: Abandon Existing WWTF, Construct Lagoon/Wetland Treatment System, Discharge into Rhodair Gully (or Johns Gully)

Abandon existing WWTF, convert existing influent lift station to a transfer lift station to pump the raw wastewater flow to a lagoon/constructed surface flow wetland for full treatment of all flows and then discharge into Rhodair Gully (or Johns Gully) at the following effluent limits.

ADF	=	5.22 mgd
2-Hour Peak Flow	=	18,125 gpm (26.10 mgd)
BOD_5	=	10 mg/l
TSS	=	15 mg/l
NH_3	=	3 limit
D.O.	=	6 mg/l

A. Preliminary Treatment (at existing WWTF)

1. Screening

Existing: One mechanical bar screen, 7 ft. 5 in. ± length, 5 ft. total width, 30 bars, 3/8 in. width on 1 3/8" centers, with mechanical cleaning mechanism; design liquid depth 6 ft. maximum.

Required: Some form of screening; bar openings minimum ½" for mechanical screens; velocities @ design flow minimum 2 ft./sec through channel, < 3 ft./sec. through screen.

Analysis: Bar openings = 1"

Design Flow = 5.22 mgd = 8.08 cfs

Channel Velocity = 8.08 cfs / (5 ft. x 6 ft.)
= 0.27 fps

Screen Velocity = 8.08 cfs / (30 x 1/12 ft. x 6 ft.)
= 0.54 fps

Improvements: NONE

2. Influent (Transfer) Lift Station. Convert the existing influent lift station to transfer the raw wastewater flows to the proposed lagoon/constructed wetland.

Existing: Four pumps, submersible type, installed in dry pit.

Required: *Firm pumping capacity (largest pump out of service) must be adequate to pump peak flow to destination; three or more pumps (or duplex pumps with automatic variable speed control) required.*

Analysis: Firm capacity = 18,125 gpm

Four (4) pumps with a firm pumping capacity of 18,125 gpm with largest pump out of service.

Transfer force main sized for a minimum of 2 fps at low flow and a maximum 10 fps at peak flow.

Install two (2) force mains, one for ADF and one as a Peak Flow force main. Proposed ADF force main = 18" diameter proposed peak flow force main = 20" diameter.

Improvements: **Convert the influent lift station to an effluent lift station with five (5) pumps (firm capacity of 18,125 gpm) and a dual 18"/30" diameter transfer force main to the proposed lagoon/constructed wetland facility.**

B. Facultative Lagoon. Construct a facultative lagoon for primary treatment of all wastewater flows.

Required: *The organic loading, based on the surface area, shall not exceed 150 lbs* BOD_5 *per acre per day. 50% of* BOD_5 *removed in facultative lagoon.*

Analysis: (5.22 mgd)(8.345)(200 mg/l BOD_5) = 8712 lbs. BOD_5/day

Required Surface Area = (8,712 lb. BOD_5/day) / (150 lb. BOD_5/acre-day)
= 58.1 acres

Improvements: **Construct a 58.1 acre facultative lagoon.**

C. Constructed Wetland Facility. Construct a surface flow wetland for polishing the effluent from the existing WWTF and discharge into Rhodair Gully (or Johns Gully).

Required: *Detention time required for a fraction* BOD_5 *remaining after secondary treatment of 0.50 and a permitted* BOD_5 *of 10 mg/l is 15 days. In-situ or constructed clay liner or synthetic liner required. Wetland must be protected from a 100-year flood. Berms shall have 3H:1V sideslopes. Multiple cells required, multiple inlets/outlets required. Refer to section 317.15. Appendix G of the TNRCC Design Criteria for Sewerage Systems* *for additional requirements.*

Analysis: For NH_3 reduction a Marsh-Pond-Marsh configuration will be utilized. Marsh sections will encompass approximately 66% of total area at an average water depth of 8", and Pond section will have an average water depth of 36". Mean water depth across entire wetland will be approximately 17". Assume a porosity for wetland vegetation of 86%.

Per Jefferson County Soil Survey, Series 1960, No. 21, average monthly rainfall and evaporation rates are listed under Alternate D2

$$\frac{(5{,}220{,}000 \text{ gpd})(15 \text{ days})}{(7.48 \text{ gals./ft}^3)(17"/12\text{inch/ft.})(0.86)} = 8{,}591{,}995 \text{ ft}^2$$

$$= 198 \text{ acres}$$

Proposed wetland = 210 acres

$$\text{Rainfall contribution} = \frac{(3.03")(210 \text{ ac.})(43560 \text{ ft}^2/\text{ac.})(7.48 \text{ gal/ft}^3)}{(12 \text{ inches/ft.})(30 \text{ days/month})}$$

$$= 575{,}902 \text{ gpd}$$

Average flow through wetland = (Influent + Effluent) / 2
= (5,220,000 + 5,795,902) / 2
= 5,507,951 gpd

$$\frac{(210 \text{ ac.})(43560 \text{ ft}^2/\text{ac.})(7.48 \text{ gals./ft}^3)(17"/12\text{inch/ft.})(0.86)}{5{,}507{,}951 \text{ gpd}} = 15.1\text{days}$$

Determine loading rates based on 210 acres

$$BOD_5 = (5.22 \text{ mgd})(8.345)(100 \text{ mg/l}) = \frac{4356 \text{ lb./day}}{210 \text{ acres}}$$

$$= 20.7 \text{ lb./acre-day}$$

$$TSS = 20.7 \text{ lb./acre-day}$$

$$NH_3 = (5.22 \text{ mgd})(8.345)(20 \text{ mg/l}) = \frac{871 \text{ lb./day}}{210 \text{ acres}}$$

$$= 4.2 \text{ lb./acre-day}$$

Improvements: **Construct a surface flow wetland with a total treatment area of 210 acres.**

Total area required for Facultative Lagoon and Constructed Wetland, including perimeter easements, is approximately 350 acres.

D. Effluent Works.

1. Post-aeration.

Improvements: **Construct a passive, cascade type aeration structure on the discharge from the constructed wetland capable of producing a 6 mg/l dissolved oxygen effluent.**

2. Flow Measurement.

Required: *Continuous flow measurement required, with capacity for maximum expected peak flow.*

Analysis: Peak flow = 26.1 mgd + maximum rainfall.

Per 1960 Jefferson County Soils Survey report the wettest month is listed as May, 1946 with a total rainfall of 20.01 inches.

$$\text{Rainfall} = \frac{(20.01")(210\text{ ac.})(43560\text{ ft}^2/\text{ac.})(7.48\text{ gal/ft}^3)}{(12\text{ inches/ft.})(30\text{ days/month})}$$

$$= 3{,}803{,}237\text{ gpd}$$

$$\text{Peak flow} = 26.1\text{ mgd} + 3.80\text{ mgd}$$
$$= 29.9\text{ mgd}$$

Improvements: **Construct a parshall flume flow measurement structure on the discharge from the constructed wetland capable of measuring flows up to 35mgd.**

E. Opinion of Probable Cost

Proposed Wetland Site 'A' - Located between Highway 69,96,287 and West Port Arthur Road (SPUR 93), adjacent to and on the north side of Rhodair Gully. Availability questionable.

1.	Convert influent lift station into transfer lift station	$	365,000
2.	Electrical and instrumentation	$	150,000
3.	Construct transfer force main	$	2,907,000
4.	Construct facultative lagoon	$	1,138,000
5.	Constructed wetland system	$	4,112,000
6.	Effluent flow measurement/post-aeration	$	75,000
7.	Laboratory/Office	$	75,000
8.	Land Acquisition	$	600,000
9.	Abandon existing treatment facility	$	500,000
	Subtotal	$	9,922,000
	Contingency (15%)	$	1,488,000
	TOTAL OPINION OF PROBABLE CONSTRUCTION COST	$	**11,410,000**

Proposed Wetland Site 'B' - Located west of West Port Arthur Road (SPUR 93), south of the proposed Federal Prison site,adjacent to Johns Gully. Assumed effluent limits of 10/15/3 for Johns Gully. Availability likely.

1.	Convert influent lift station into transfer lift station	$	390,000
2.	Electrical and instrumentation	$	150,000
3.	Construct transfer force main	$	5,592,000
4.	Construct facultative lagoon	$	1,138,000
5.	Constructed wetland system	$	4,112,000
6.	Effluent flow measurement/post-aeration	$	75,000
7.	Laboratory/Office	$	75,000
8.	Land Acquisition	$	300,000
9.	Abandon existing treatment facility	$	500,000
	Subtotal	$	12,332,000
	Contingency (15%)	$	1,850,000
	TOTAL OPINION OF PROBABLE CONSTRUCTION COST	$	**14,182,000**

N6 - City of Nederland: **Divert Discharge to the Neches River**

Upgrade the existing the existing WWTF for discharge into the Neches River at the following effluent limits.

ADF	=	4.76 mgd
2-Hour Peak Flow	=	18,055 gpm (26 MGD)
BOD_5	=	20 mg/l
TSS	=	20 mg/l
NH_3	=	N/A
D.O.	=	4 mg/l

A. Preliminary Treatment (Before splitting into tracks)

1. Screening

Existing: One mechanical bar screen, 7 ft. 5 in. ± length, 5 ft. total width, 30 bars, 3/8 in. width on 1 3/8" centers, with mechanical cleaning mechanism; design liquid depth 6 ft. maximum.

Required: *Some form of screening; bar openings minimum ½" for mechanical screens; velocities @ design flow minimum 2 ft./sec through channel, < 3 ft./sec. through screen.*

Analysis: Bar openings = 1"

Design Flow = 4.76 mgd = 7.37 cfs

Channel Velocity = 7.37 cfs / (5 ft. x 6 ft.)
= 0.123 fps

Screen Velocity = 7.37 cfs / (30 x 1/12 ft. x 6 ft.)
= 0.49 fps

Improvements: **NONE**

2. Influent Lift Station.

Existing: Four pumps, submersible type, installed in dry pit, each 2900 gpm capacity for firm capacity of 8700 gpm. (Two of the pumps are two speed with a slower speed of 900 gpm, with pump speed automatically adjusted as a function of wet well level. The 2900 gpm rated capacities are based on an average pumping head between high and low wet well levels.)

Required: *Firm pumping capacity (largest pump out of service) must be adequate to pump peak flow to destination; three or more pumps (or duplex pumps with automatic variable speed control) required.*

Analysis: Existing firm capacity = 8700 gpm = 12.528 mgd peak flow.
Proposed firm capacity = 18,125 gpm = 26.10 mgd peak flow.

Improvements: **Upgrade pumping firm capacity of lift station to 18,125 gpm. Replace three of the existing pumps with 7613 gpm pumps.**

3. Aerated Grit Chamber

Existing: 20 ft. x 20 ft. chamber, 13 ft. water depth (less 5 ft. x 12 ft. x 4.5 ft. splitter box for effluent), plus hopper bottom with 1:1 slope (reported basin volume of 6240 ft^3); two air diffusers (112 cfm total) with 30" draft tube; concentrated grit/liquid mixture sent to degritter for final grit separation.

Required: *Grit removal recommended; if removal units are provided, must have method of removing grit from unit, and any unit with single chamber must have bypass.*

Analysis: Grit removal by grit pump below; piping allows flow to bypass grit chamber if needed. This unit also provides preaeration.

Improvements: **NONE**

4. Grit Pump

Existing: One vortex type pump, 250 gpm (pumps grit/liquid mixture from aerated grit chamber to degritter).

Improvements: **NONE**

5. Degritter

Existing: Hydrocyclone (10.5 ft. long) and grit classifier/washer (L shaped, approx. 5 ft. x 25 ft. plus 4 ft. x 3 ft. (dewaters grit from aerated grit chamber).

Improvements: **NONE**

B. Activated Sludge Process. Construct new activated sludge aeration basins.

Required: *Total volume shall be 1000 ft^3 per 45 lb. BOD_5/day. Diffused aeration shall be designed for 1800 SCF per lb. BOD_5. The diffuser system must be capable of providing 150% of design requirements.*

Analysis: lbs BOD_5/day = (4.76 MGD)(8.345)(200 mg/l) = 7944 lbs BOD_5/day (For Total Plant)

MAX BOD_5 LOAD = 45 lb. BOD_5/day/1000 ft^3

BOD_5 Loading = 45 lbs BOD_5 per day per 1000 ft^3

Activated Sludge Unit = $\frac{100^2 \text{ ft diameter} \times \pi}{4}$ x 15' SWD 117,809 ft^3

= (117,809) / 4000) x 45 = 5301 lbs BOD_5/day

Air Requirements = (5300 lb BOD_5/day) (1800 SCF/ lb BOD_5
= 9,540,000 SCF/day
= 6625 SCFM

Improvements: Convert one of the existing contact stabilization units to a complete mix activated sludge unit.

C. Final Clarifiers. Construct new final clarifiers.

Existing: Two (2) at 60 ft. diam. x 14 ft. side water depth, to be converted one to complete mix unit described above, the other to an aerobic digester. Final Clarifier serving trickling filters shell remain in service.

Required: *Maximum surface loading at Peak flow of 1200 gal./day/ft^2, and at Design flow of 600 gal./day/ft^2. Side water depth must be at least 10 ft. for surface areas of 1250 ft^2 or more. Effective detention times (based on liquid volume above a 3 ft. sludge blanket) must be 1.5 hr. @ Peak flow and 3.0 hr. @ Design flow.*

Analysis: Required Area based on surface area:
@ Peak Flow 26,000 - 5,440,000 (allowable peak for TF Clarifier) = 20,560,000/1200 17,133 ft^2
@ Design Flow 5.22 - 2.77 = 2.45 MGD = 4,083 ft^2

Required Area based on detention time:
Detention time is based on side water depth of 14 ft less 3 ft. sludge blanket.

@ Peak flow = (20,560,000 gpd)(1.5 hrs.)/[(24 hrs./day)(7.48 gal/ft^3)(11 ft)] = 15,617 ft^2

@ Design flow = 2,450,000 gpd)(3.0 hr.)/[(24 hrs./day)(7.48 gal/ft^3)(11 ft)] = 3,722 ft^2

Improvements: **Construct two (2) new 110 ft. diameter clarifiers with 14 ft. side water depth required (based on minimum required surface area). Provide flow splitting/collection structures/ piping, and sludge collection/pumping.**

D. Effluent Works.

1. Chlorine Contact Chamber.

Existing: Inside dimensions 98 ft. 6 in. x 38 ft. 2 in. including partitions and baffles; minimum liquid depth 6.25 ft. (6 ft. in final compartment); hopper bottoms in two 18.5 ft. x 18.75 ft. portions of chamber; fine bubble diffusers for mixing.

Required: *Detention time of 20 minutes @ peak flow.*

Analysis: Existing volume approximately 23,000 ft^3
23,000 ft^3/20 min. = 1150 cfm = 19.2 cfs = 12.39 mgd

Additional volume required = 8.17 mgd @ 20 minutes
= 15,170 ft^3

Improvements: **- Raise water surface elevation 4 feet to achieve needed volume for activated sludge treatment process**

- Construct new chlorine contact chamber with 11,140 ft^3 for trickling filter process.

2. Chlorine Feed Equipment.

Existing: Two systems, each 500 lb./day feed capacity (vacuum operated) including one standby; flow proportioned; chlorine gas from one ton size containers.

Required: *Feed equipment must be able to provide more than the highest dosage to be required at any time. Dosage must be adequate to maintain a chlorine residual of at least 1 mg/l after 20 minutes* *detention, prior to dechlorination.*

Analysis: In standard practice, feed equipment is designed to feed 10 ppm of Cl_2 in order to assure a 1 mg/l residual.

(500 lb./day)/(10 ppm)(8.345 lb./gal.) = 5.99 mgd
If both feeders can be used simultaneously during peak flows, they would have a theoretical capacity for 11.98 mgd peak.

Improvements: **Provide additional chlorine feed equipment as necessary to provide for chlorination of 26.00 mgd.**

3. Dechlorination.

Existing: Liquid ammonium bisulfate, 3000 gal. storage tank, one metering pump with 96 gal./day capacity; injection and reaction occur in a transitional area between chlorination and flow measurement. This dechlorination area is structurally an extension of the chlorine contact chamber, 8 ft. x 10 ft. 8 in. rectangle plus an adjacent trapezoidal area, 5 ft. long, width transitional from 8 ft. to 3 ft. 11 in.

Required: *The effluent, after chlorination and 20 minutes detention time, must be dechlorinated to less than 0.1 mg/l. For most dechlorination agents, 1 minute detention is generally considered adequate.*

Analysis: 26,10 MGD = 40.38 cfs = 2,423 cfm
2,423 cfm (1 min.) = 2,423 ft^3

Improvements: **Allow sufficient area for dechlorination in the chlorine contact basins.**

4. Flow Measurement.

Existing: 24 inch parshall flume; continuously indicating, recording, and totalizing flow meter calibrated to read up to 15 mgd.

Required: *Continuous effluent measurement required, with capacity for maximum expected peak flow.*

Analysis: Existing effluent measurement is not adequate for peak flows up to 26.10 mgd.

Improvements: **Construct a new rectangular weir with continuous flow recorder capable of measuring up to 30 mgd.**

E. Sludge Processing.

1. Sludge Thickener.

Existing: 38 ft. diam. x 14 ft. side water depth, bottom slopes to center @ 4:1; mechanical sludge collection with pickets; supernatant to Trickling Filter No. 1.

Required: *Aerobic digesters should be provided with sludge thickening.*

Improvements: **Supernatant from thickener shall be diverted back to head of the plant.**

2. Aerobic Digesters. Convert one contact stabilization unit to an aerobic digester.

Requirement: *Minimum solids retention time of 15 days (may be calculated as 20 ft^3 for each lb. influent BOD_5 per day). Diffused air requirement is 30 cfm per 1000 ft^3 of volume.*

Analysis: Digester Volume Required = Sufficient for 15 days SRT
lb. BOD_5/day = (4.76 mgd)(8.345)(200 mg/l) = 7944 lb. BOD_5/day

Required Digester Volume = (7944 lb. BOD_5/day)(14.75 ft^3/lb. BOD/day) = 117,174 ft^3

Existing Unit: $[\pi(100 \text{ ft.})^2/4](15 \text{ ft.}) = 117{,}809 \text{ ft}^3$ each
By utilizing the sludge thickener an SRT of 15 days is achievable allowing 14.75ft^2/lb BOD_5.

Required Aeration = (30 cfm/1000 ft^3)(117,809 ft^3) = 3534 cfm

Improvements: **Convert one (1) existing contact stabilization plants into an aerobic digester and provide 3,534 cfm aeration equipment capacity.**

3. Centrifuge Facility.

Existing: One sludge grinder; two sludge metering pumps, progressive cavity, 60 gpm; one polymer feed pump (for 6% solution); two 200 gallon polymer mixers; one polymer metering pump; one horizontal centrifuge, 60 gpm with 20 hp motor and mixing tank to introduce polymer into sludge.

Improvements: **Provide additional sludge dewatering facilities.**

4. Drying Beds.

Existing: Two sets of open sand beds, 76 ft. x 220 ft. and 50 x 100 ft.; used for standby only.

F. Blowers.

Existing: Four blowers, 1500 cfm each, existing firm capacity = 4500 cfm.

Required: *Blowers must be able to meet maximum aeration requirements with largest unit out of service.*

Improvements: **Provide additional blowers as necessary.**

G. Construct a Lift Station and associated Force Main adequate for pumping 4.76 (future 5.22) MGD to the Neches River. Excess flows occur only during rain events and are planned for discharge to the present receiving stream.

H. Opinion of Probable Cost

1.	Upgrade influent lift station	$	210,000
2.	Convert 1 Contract Unit to Activated Sludge	$	115,000
3.	Convert Contract Unit to Aerobic Digester	$	115,000
4.	Construct 2 110' diameter clarifiers	$	1,000,000
5.	RAS/WAS Pump Station	$	150,000
6.	Convert Existing Chlorine Basin for ASTU	$	50,000
7.	Construct New Chlorine Basin for TF	$	100,000
8.	Lift Station & Force Main to River	$	1,659,000
9.	Sludge Dewatering	$	440,000
10.	Additional Blowers & Diffussers	$	100,000
11.	Yard Piping	$	250,000
12.	Electrical	$	200,000
	Subtotal	$	4,389,000
	Contingency (10%)	$	439,000
	TOTAL OPINION OF PROBABLE CONSTRUCTION COST	$	**4,828,000**

This alternative can be upgraded to treat 5.05 MGD ADF to limits of 10/15/3 by construction of a new aerobic digester and conversion of the digester built in Phase I to a single stage nitrification basin. The only other work required would be the addition of blower capacity and some piping modifications which can be planned for. The anticipated cost for upgrading this facility to nitrify is **$660,000.00.**

DISCUSSION OF VARIOUS FACTORS REGARDING ALTERNATIVES

Comparison of Alternatives: Cost Breakdowns

In developing the various alternatives which involved receiving streams other than the sensitive drainage ditch system, the Engineers considered carefully the anticipated effluent parameters not only upon initial operation but also in future years. Since Rhodair and Johns Gullies were expected to have 10/15 parameters initially, the initial and future plant designs coincided with no problem.

However, the Neches River is a different matter. For Segment 601 of that stream, TWC/TNRCC stream modelling has indicated a 20/20 standard for flows even greater than all of the Midcounty flows combined. Unfortunately, our past experience with permitting agencies demonstrates that a 20/20 limit cannot be relied on as a permanent standard, especially in the case of large discharges into tidal or coastal streams. In light of our experience over the last 10 to 15 years, it would be shortsighted to involve our clients in major capital projects of this nature without making reasonable provisions for future tightening of standards.

In the case of activated sludge, a 10/15 capability can be provided at a relatively low incremental cost by increasing basin and blower sizing. A trickling filter plant could likewise be redesigned to 10/15 by increasing media volume. However, the ammonia limit which often accompanies a 10/15 permit may present a severe problem. The TNRCC does not recognize the nitrification ability of the trickling filter process and usually requires additional treatment such as solids contact. Since such additional units are outside the scope of the basic trickling filter process, rather than simply a matter of increasing initial sizing, they were not included in plant design.

An examination of the various alternative plant designs and associated cost estimates indicates that for a typical 10/15 activated sludge design, the cost could be reduced by only 6%-8% by downgrading to a 20/20 design. In almost every case, the trickling filter plant is still cost effective on a capital cost basis. In the case of the Groves South Plant, a 20/20 activated sludge plant appears to be marginally cost effective in comparison with a trickling filter plant, but any savings in capital cost would be negligible in comparison with the much higher operating costs of activated sludge.

Even when the reduced operating costs of 20/20 design for activated sludge are considered, this selection does not become cost effective in comparison with trickling filters. An examination of summaries of operating costs shows that the operating costs could be reduced by only 10% to 12%. The present worth of this operating cost reduction, along with the savings in capital cost over a 10/15 design,, is still insufficient to make the activated sludge process cost effective for any individual plant.

The 10/15 activated sludge alternatives reflect a loading of 30 lb/day of BOD_5 per 1000 ft^3 of aeration volume *(for single stage nitrification)* instead of the TNRCC maximum loading of 35 lb/ 1000 ft^3, showing an apparent safety factor of 15%. The safety factor is provided to reflect the Engineer's experience that the process functions more reliably when not loaded to its limit. Please note that the safety factor was not applied to the aeration system or to blower capacity. The blowers were sized for a firm capacity *(with largest unit out of service)* of 100% of requirements, including other blower usage such as aerobic digestion. The air piping and diffusers within the activated sludge basin were sized at 150% of requirement as a safety requirement of the TNRCC.

Staged construction is not advisable for any of the activated sludge alternatives. For these processes, considering the relatively small difference in plant design, it is more practical to construct all needed improvements at the beginning. Although staged construction may ultimately prove necessary for the trickling filter plants, the uncertainty of its extent and timing makes it inappropriate to address it in this report.

For a planning study of this scope, detailed cost breakdowns for each alternative are not necessary, since non-construction costs can reasonably be assumed proportional to construction costs in broad applications of this nature.

Selection of Regional Plant Site.

The vacant land adjacent to the Port Neches and Groves North plants, located on the east *(compass northeast)* side of those plants, is owned by Huntsman Corporation, which operates several petrochemical plants on contiguous land not far to the east.. During the course of the project the City of Groves has approached Huntsman about purchasing land for the expansion of the Groves North plant. Huntsman appears to be agreeable, but is reluctant to part with more than the minimum amount of land needed for a plant expansion. This reluctance is reflected in Huntsman's preference to grant a buffer easement outside the plant site rather than to sell a site large enough to include the required 150 foot buffer. It should also be noted that Huntsman was very reluctant to sell the City a large enough site to build a complete plant while the existing plant remained in operation during construction, but preferred to sell only enough land to supplement the existing site.

Discharge of Wet Weather Flows into Adjacent Drainage Ditches.

The concept of routing excess flows into the drainage ditch system during wet weather is not an unrealistic proposal. On December 16, 1994, the Engineer sent a letter* to the TNRCC inquiring as to the possibility of discharging flows *(from each plant)* in excess of the design flow into the ditch system during wet weather. The letter indicated clearly that the proposed effluent quality would be 20/20 for such discharges. The TNRCC replied by letter* of January 3, 1995 that such a practice could possibly be acceptable. The letter listed various information which would be needed to make a determination, including the quality and flow in the ditches upstream from the discharge points. Another factor cited was whether such discharges would include brief peak flows during days of normal dry weather.

*Copies of letters attached.

The cities were told what tests and measurements would be needed for the TNRCC to make any determinations. The required information is expected to be submitted to the TNRCC in the early stages of project implementation for each city, once a final determination is made regarding the relevant plant flows in each case.

Thus it is quite possible that excess flows can be allowed into the ditch system during wet weather flow conditions. Even if the TNRCC does not approve the plant design flow as the dividing point, it is reasonable to expect some other flow rate between the design flow and the two hour peak flow to be approved.

Schaumburg & Polk, Inc.
CONSULTING ENGINEERS

8865 College St., Suite 100
Beaumont, Texas 77707
Phone (409) 866-0341
FAX (409) 866-0337

December 16, 1994

Mr. Mark A. Rudolf
Texas Natural Research Conservation Commission
Watershed Management
P.O. Box 13087
Austin, Texas 78711-3087

Re: City of Nederland (Permit #10483-02)
City of Port Neches (Permit #10477-04)
City of Groves (Permit #10094-02)

Dear Mr. Rudolf:

As you are aware, all three of the above referenced wastewater treatment facilities (WWTF) currently discharge into the Alligator Bayou / Jefferson County Drainage District #7 drainage system. Per your letter dated February 4, 1993 to Mr. Gary Graham of our office, future permit limits for discharge into this receiving stream will require treatment to significantly lower levels. As a part of the regional wastewater study we are performing for Nederland, Port Neches and Groves, we are considering diverting the discharge from each of these three WWTF's to the Neches River.

The cost of pumping 100% of the effluent flow to the Neches River will be extremely costly. Therefore, we would like to consider pumping flows up to the maximum monthly ADF to the Neches River, and continue to discharge the wet weather peak flows in excess of ADF to the existing receiving stream. As the proposed discharge to the existing receiving stream would only occur during periods of wet weather (i.e. when the drainage system has a significant flow), we would expect that the stream could receive these wet weather peak flows at current discharge limits without effecting water quality.

We would like to know if this alternative would be acceptable to the Commission, and look forward to receiving your response.

Regional WW Study
SPI No. 4004.0/10101.0/10201.0
DF: \C:\OFFICE\WPWIN\WPDOCS\LTRHEAD.SPI

If you have any questions or need any additional information, please feel free to contact either myself or Mr. Gary Graham at our office.

Sincerely,
Schaumburg & Polk, Inc.

Jeffrey G. Beaver

Jeffrey G. Beaver, P.E.
Vice President

c: Mr. Steve Hamilton, P.E., City of Nederland
Mr. James L. Harrington, City of Port Neches
Mr. George Newsome, P.E., City of Groves

RECEIVED

THE STATE OF TEXAS

TEXAS NATURAL RESOURCE CONSERVATION COMMISSION

Protecting Texas by Reducing and Preventing Pollution

January 3, 1995

Jeffrey G. Beaver, P.E.
Schaumburg & Polk, Inc.
8865 College St., Suite 100
Beaumont, TX 77707

Dear Mr. Beaver:

Thank you for your letter inquiring about the possibility of discharging wet weather peak flows from the wastewater treatment plants permitted to the Cities of Nederland, Port Neches, and Groves. The alternative that you outlined would involve piping wastewater from each facility to the Neches River for effluent flows up to the permitted average daily flow. Wet weather flows in excess of this amount would be discharged into the canal system adjacent to these facilities. Furthermore, you indicated a desire to permit these wet weather discharges at currently permitted limits (20 mg/L BOD_5/20 mg/L TSS).

It is possible that discharging peak flows to the canal system during wet weather would be acceptable. However, if you wish for us to estimate whether this would be reasonable, you will need to provide us with additional detailed information regarding the project. Specifically, the following minimum information is needed:

- Background water quality in the canals upstream of the treatment plants during stormwater runoff conditions. Values for BOD_5, ammonia nitrogen, and dissolved oxygen are needed.
- The relationship between effluent flow (daily volume) discharged into the canals for each facility versus the stormwater flow in the canals upstream of the discharges. In other words, the background flow expected in the canals for various effluent discharge volumes.

In addition to this information it would be helpful if you would explain whether the pipelines would be sized to handle instantaneous peak flows associated with the normal daily variation in flow rate during dry weather conditions.

APPENDIX D - Wastewater Treatment Alternatives

PN-1	City of Port Neches:	All Flows
PN-2	City of Port Neches:	ADF Only
PN/G-1	City of Port Neches/ City of Groves North:	N All Flows
PN/G-2	City of Port Neches/ City of Groves North:	N ADF Only
PN/G-3	City of Port Neches/ City of Groves:	N & S ADF Only

Also: Refer to Appendix C for General Discussion.

PN-1 City of Port Neches: All Flows

PN-1 City of Port Neches: All Flows

This alternative is basically the same as PN/G1; however, this Alternate PN-1 is to construct an effluent lift station and force main(s) to serve only the City of Port Neches WWTF.

A. Lift Station/Force Main. The proposed flow capacity of the effluent lift station and force main(s) will provide for the following:

ADF	4.98 mgd
2-Hour Peak	25.13 mgd

Required: *Firm pumping capacity (largest pump out of service) must be adequate to pump peak flow to destination; three or more pumps (or duplex pumps with automatic variable speed control) required.*

Analysis: Firm capacity = 25.13 mgd (17,451 gpm)

Four (4) pumps total; firm capacity of 17,451 gpm with largest pump out of service (i.e. 3 pumps pumping + 1 spare).

Effluent force main(s) sized for a minimum of 2 fps at low flow and a maximum 10 fps at peak flow.

Proposed force main = 30" diameter

Velocity @ 17,451 gpm = 7.9 fps

Improvements: **Construct an effluent lift station with four (4) pumps (firm capacity of 17,451 gpm) and a 30" diameter force main to the Neches River.**

B. Outfall Route(s). Three possible routes to the Neches River for the proposed outfall force main have been considered.

Route 'A' From proposed effluent lift station, located at the Port Neches/Groves North WWTF site(s), to Hogaboom Road. Down Hogaboom Road to Highway 366. Down Highway 366 to Orchard Street. Then, down Orchard Street to the Neches River. Approximately **18,500 linear feet.**

Route 'B' From proposed effluent lift station, located at the Port Neches/Groves North WWTF site(s), to Hogaboom Road. Down Hogaboom Road to Highway 366. Cross Highway 366, then straight to the Neches River. Approximately **16,500 linear feet.** (Route from Highway 366 straight to the Neches River is questionable, would require easement)

Routh 'C'	From proposed effluent lift station, located at the Port Neches/Groves North WWTF site(s), to Highway 136. Down Highway 136 to the Neches River. Approximately **16,000 linear feet**. (Route down Highway 136 questionable, may not be any available space within right-of-way)

Route 'A' was used for deterimining opinion of probable construction cost. Each individual route should be analyze in depth during preliminary engineering phase of design/construction project.

C. Opinion of Probable Cost

1.	Construct effluent lift station		$	284,000
2.	Yard piping improvements		$	25,000
3.	Electrical and instrumentation		$	110,000
4.	Construct outfall force main(s)		$	1,721,000
		Subtotal	$	2,140,000
		Contingency (15%)	$	321,000
	TOTAL OPINION OF PROBABLE CONSTRUCTION COST		**$**	**2,461,000**

PN-2 City of Port Neches: ADF Only

PN - 2 City of Port Neches: **ADF Only**

In analysing Alternate PN-1 it was assumed that the TNRCC would require that all flows be diverted to the Neches River. During wet weather the existing receiving stream may have adequate flow to receive a discharge at less stringent quality standards. Therefore, if the TNRCC would allow all flows up to the permitted ADF from each treatment facility to be diverted to the Neches River and allow peak flow in excess of the permitted ADF to continue to be discharged to the existing receiving stream at current 20/20 limits, then the cost of this alternative could be significantly reduced.

Analysis: *Firm capacity = 4.98 mgd (3,458 gpm)*

Three (3) pumps total; firm capacity of 3,458 gpm with largest pump out of service (i.e. 2 pumps pumping + 1 spare).

Effluent force main(s) sized for a minimum of 2 fps at low flow and a maximum 10 fps at peak flow.

Proposed force main = 18" diameter

Improvements: ***Construct an effluent lift station with three (3) pumps (firm capacity of 3,458 gpm) and a 18" diameter force main to the Neches River.***

Opinion of Probable Cost

1.	Construct effluent lift station	$	142,000
2.	Yard piping improvements	$	25,000
3.	Electrical and instrumentation	$	55,000
4.	Construct outfall force main(s)	$	1,018,000
	Subtotal	$	1,240,000
	Contingency (15%)	$	186,000
	TOTAL OPINION OF PROBABLE CONSTRUCTION COST	$	**1,426,000**

PN/G-1 - City of Port Neches/Groves North: North All Flows

PN/G-1 - City of Port Neches/City of Groves North: North All Flows

The City of Port Neches WWTF is designed to produce a secondary effluent of 20 mg/l BOD_5 and 20 mg/l TSS; however, it is expected that the WWTF will be required to meet effluent limits of 5 mg/l BOD_5, 5 mg/l TSS, 2 mg/l NH_3 and 6 mg/l D.O. in the future for continued discharge into the existing receiving stream.. Therefore, this alternate proposes to divert the effluent discharge to the Neches River where the existing secondary effluent limits would still be required.

The City of Groves North WWTF (adjacent to the Port Neches Plant) is undersized and in need of replacement. Alternates G1 and G2 address possible treatment alternatives for the Groves North WWTF. However, as it is also proposed to divert the effluent discharge from the Groves North WWTF to the Neches River, it would be desirable to transport flows from both plants with common facilities.

Therefore, this Alternate PN/G1 is to construct an effluent lift station and force main(s) to serve both the City of Port Neches WWTF and the City of Groves North WWTF.

A. Lift Station/Force Main. The proposed flow capacity of the effluent lift station and force main(s) will provide for the following:

	Port Neches[1]	Groves North	Total
ADF	4.98 mgd	1.99 mgd	**6.97 mgd**
2-Hour Peak	25.13 mgd	5.97 mgd	**31.10 mgd**

[1] Use design capacity flow rates for Port Neches WWTF.

Required: *Firm pumping capacity (largest pump out of service) must be adequate to pump peak flow to destination; three or more pumps (or duplex pumps with automatic variable speed control) required.*

Analysis: Firm capacity = 31.10 mgd (21,597 gpm)

Five (5) pumps total; firm capacity of 21,597 gpm with largest pump out of service (i.e. 4 pumps pumping + 1 spare).

Effluent force main(s) sized for a minimum of 2 fps at low flow and a maximum 10 fps at peak flow.

Install two (2) force mains, one for ADF and one as a Peak Flow force main.

Proposed ADF force main = 24" diameter
Proposed Peak Flow force main = 30" diameter

Improvements: Construct an effluent lift station with five (5) pumps (firm

capacity of 21,597 gpm) and dual 24"/30" diameter force mains to the Neches River.

B. Outfall Route(s). Three possible routes to the Neches River for the proposed outfall force main have been considered.

Route 'A' From proposed effluent lift station, located at the Port Neches/Groves North WWTF site(s), to Hogaboom Road. Down Hogaboom Road to Highway 366. Down Highway 366 to Orchard Street. Then, down Orchard Street to the Neches River. Approximately **18,500 linear feet**.

Route 'B' From proposed effluent lift station, located at the Port Neches/Groves North WWTF site(s), to Hogaboom Road. Down Hogaboom Road to Highway 366. Cross Highway 366, then straight to the Neches River. Approximately **16,500 linear feet**. (Route from Highway 366 straight to the Neches River is questionable, would require easement)

Route 'C' From proposed effluent lift station, located at the Port Neches/Groves North WWTF site(s), to Highway 136. Down Highway 136 to the Neches River. Approximately **16,000 linear feet**. (Route down Highway 136 questionable, may not be any available space within right-of-way)

Route 'A' was used for determining opinion of probable construction cost. Each individual route should be analyze in depth during preliminary engineering phase of design/construction project.

C. Opinion of Probable Cost

1.	Construct effluent lift station		$	355,000
2.	Yard piping improvements		$	50,000
3.	Electrical and instrumentation		$	135,000
4.	Construct outfall force main(s)		$	3,015,500
		Subtotal	$	3,555,500
		Contingency (15%)	$	533,000
TOTAL OPINION OF PROBABLE CONSTRUCTION COST			**$**	**4,088,500**

PN/G-2 City of Port Neches/Groves: North ADF Only

PN/G-2: City of Port Neches/City of Groves North: North ADF Only

In analysing Alternate PN/G1 it was assumed that the TNRCC would require that all flows be diverted to the Neches River. During wet weather the existing receiving stream may have adequate flow to receive a discharge at less stringent quality standards. Therefore, if the TNRCC would allow all flows up to the permitted ADF from each treatment facility to be diverted to the Neches River and allow peak flow in excess of the permitted ADF to continue to be discharged to the existing receiving stream at current 20/20 limits, then the cost of this alternative could be significantly reduced.

Analysis: *Firm capacity = 6.97 mgd (4,840 gpm)*

Three (3) pumps total; firm capacity of 4,840 gpm with largest pump out of service (i.e. 2 pumps pumping + 1 spare).

Effluent force main(s) sized for a minimum of 2 fps at low flow and a maximum 10 fps at peak flow.

Proposed force main = 24" diameter

Improvements: ***Construct an effluent lift station with three (3) pumps (firm capacity of 4,840 gpm) and 24" diameter force main to the Neches River.***

Opinion of Probable Cost

1.	*Construct effluent lift station*	*$*	*220,000*
2.	*Yard piping improvements*	*$*	*50,000*
3.	*Electrical and instrumentation*	*$*	*95,000*
4.	*Construct outfall force main(s)*	*$*	*1,295,000*
	Subtotal	*$*	*1,660,000*
	Contingency (15%)	*$*	*249,000*
	TOTAL OPINION OF PROBABLE CONSTRUCTION COST	***$***	***1,909,000***

PN/G-3 City of Groves N & S/Port Neches: N & S ADF Only

PN/G-3 City of Groves North & South\City of Port Neches: N & S ADF Only

This alternative is basically the same as PN/G-2; however, Alternate PN/G-3 is to construct an effluent lift station and force main(s) to serve both the City of Port Neches WWTF and a Regional Groves WWTF located at the North Plant Site.

A. Lift Station/Force Main. The proposed flow capacity of the effluent lift station and force main(s) will provide for the following:

	Port Neches[1]	Groves North	Groves South	Total
ADF	4.98 mgd	1.99 mgd	3.33 mgd	**10.30 mgd**
2-Hour Peak	25.13 mgd	5.97 mgd	18.70 mgd	**49.80 mgd**

[1] Use design capacity flow rates for Port Neches WWTF.

Required: *Firm pumping capacity (largest pump out of service) must be adequate to pump peak flow to destination; three or more pumps (or duplex pumps with automatic variable speed control) required.*

Analysis: Firm capacity = 49.80 mgd (34,583 gpm)

Six (6) pumps total; firm capacity of 34,583 gpm with largest pump out of service (i.e. 5 pumps pumping + 1 spare).

Effluent force main(s) sized for a minimum of 2 fps at low flow and a maximum 10 fps at peak flow.

Proposed force main = 36" diameter and, = 24" diameter

Improvements: **Construct an effluent lift station with six (6) pumps (firm capacity of 34,583 gpm) and a parallel 24" diameter and 36" diameter force mains to the Neches River.**

B. Outfall Route(s). Three possible routes to the Neches River for the proposed outfall force main have been considered.

Route 'A' From proposed effluent lift station, located at the Port Neches/Groves North WWTF site(s), to Hogaboom Road. Down Hogaboom Road to Highway 366. Down Highway 366 to Orchard Street. Then, down Orchard Street to the Neches River. Approximately **18,500 linear feet.**

Route 'B' From proposed effluent lift station, located at the Port Neches/Groves North WWTF site(s), to Hogaboom Road. Down Hogaboom Road to Highway 366. Cross Highway 366, then straight to the Neches River. Approximately

16,500 linear feet. (Route from Highway 366 straight to the Neches River is questionable, would require easement)

Route 'C' From proposed effluent lift station, located at the Port Neches/Groves North WWTF site(s), to Highway 136. Down Highway 136 to the Neches River. Approximately **16,000 linear feet**. (Route down Highway 136 questionable, may not be any available space within right-of-way)

Route 'A' was used for determining opinion of probable construction cost. Each individual route should be analyze in depth during preliminary engineering phase of design/construction project.

C. Opinion of Probable Cost

1.	Construct effluent lift station	$	655,000
2.	Yard piping improvements	$	50,000
3.	Electrical and instrumentation	$	135,000
4.	Construct outfall force main(s)	$	367,000
	Subtotal	$	4,207,000
	Contingency (15%)	$	631,000
	TOTAL OPINION OF PROBABLE CONSTRUCTION COST	$	**4,838,000**

In analyzing Alternate PN/G-3 it was assumed that the TNRCC would require that all flows be diverted to the Neches River. During wet weather the existing receiving stream may have adequate flow to receive a discharge at less stringent quality standards. Therefore, if the TNRCC would allow all flows up to the permitted ADF from each treatment facility to be diverted to the Neches River and allow peak flow in excess of the permitted ADF to continue to be discharged to the existing receiving stream at current 20/20 limits, then the cost of this alternative could be significantly reduced.

Analysis: *Firm capacity = 10.30 mgd (7,153 gpm)*

Three (3) pumps total; firm capacity of 7,153 gpm with largest pump out of service (i.e. 2 pumps pumping + 1 spare).

Effluent force main(s) sized for a minimum of 2 fps at low flow and a maximum 10 fps at peak flow.

Proposed force main = 24" diameter (5.07 fps)

Improvements: ***Construct an effluent lift station with three (3) pumps (firm capacity of 7,153 gpm) and a 24" diameter force main to the Neches River.***

Opinion of Probable Cost

1.	*Construct effluent lift station*	*$*	*330,000*
2.	*Yard piping improvements*	*$*	*50,000*
3.	*Electrical and instrumentation*	*$*	*90,000*
4.	*Construct outfall force main(s)*		
	Subtotal	*$*	*1,765,000*
	Contingency (15%)	*$*	*265,000*
TOTAL OPINION OF PROBABLE CONSTRUCTION COST		***$***	***2,030,000***

Refer to Appendix C for
Discussion Regarding Design
and Selection of Alternatives

APPENDIX E - Wastewater Treatment Alternatives

G1- City of Groves:	Abandon Existing North, Construct New Trickling Filter Treatment Facility at North Plant Site, Divert Discharge to the Neches River
G2- City of Groves:	North Plant AS (Activated Sludge) 10/15
G3 - City of Groves:	South Plant AS (Activated Sludge) 10/15
G4 - City of Groves:	Upgrade South WWTF's Trickling Filter Treatment Facility Discharge to the Sabine Neches Canal
G5 - City of Groves:	Abandon Existing WWTF's, Construct New Regional Activated Sludge Treatment Facility for the City of Groves adjacent to existing North WWTF
G6 - City of Groves:	Abandon Existing WWTF's, Construct New Regional Activated Sludge Treatment Facility for the City of Groves adjacent to existing South WWTF
G7 - City of Groves:	Abandon Existing North and South WWTF's, Construct New Trickling Filter Treatment Facility at 32rd Street and SH 366 discharge to the Sabine Neches Canal
G8 - City of Groves:	Transport ADF Only to Neches River

Also: Refer to Appendix C for General Discussion.

G1 - City of Groves: Abandon Existing North, Construct New Trickling Filter Treatment Facility at North Plant Site, Divert Discharge to the Neches River

Abandon existing WWTF, construct a new trickling filter treatment facility for treatment of flows and then discharge into the Neches River at the following effluent limits.

ADF	=	1.99 mgd
2-Hour Peak	=	4,146 gpm (5.97 mgd)
BOD_5	=	20 mg/l
TSS	=	20 mg/l
NH_3	=	no limit
D.O.	=	4 mg/l

A. Preliminary Treatment

1. Screening.

Required: *Some form of screening; bar openings minimum ½" for mechanical screens; velocities @ design flow minimum 2ft./sec through channel, < 3 ft./sec. through screen.*

Analysis: Design Flow = 1.99 mgd = 3.08 cfs

Improvements: **Construct a dual channel influent structure with one mechanical bar screen in one channel, sized for a design flow of 1.99 mgd and a peak flow of 5.97 mgd, and a fixed bar screen in the second channel.**

2. Influent Lift Station. Construct an influent lift station to lift the raw wastewater flows into the proposed treatment facility.

Required: *Firm pumping capacity (largest pump out of service) must be adequate to pump peak flow to destination; three or more pumps (or duplex pumps with automatic variable speed control) required.*

Analysis: Firm capacity = 4,146 gpm

B. Primary Clarifier.

Required: Maximum surface loading at Peak flow of 1800 gal./day/ft², and at Design flow of 1000 gal./day/ft². Side water depth must be at least 7 ft.

Analysis: Required area:
@ Peak flow = 5,,970,000 gpd / 1800 gal./day/ft² = 3,317 ft²

@ Design flow = 1,990,000 gpd / 1000 gal./day/ft^2 = 1,990 ft^2

Improvements: **Construct one (1) 78 ft. diameter primary clarifier with 14 ft. side water depth. Provide sludge collection/pumping.**

C. Trickling Filter.

Required: *Application of synthetic media shall be evaluated on a case-by-case basis. The design engineer shall submit sufficient operating data from existing trickling filters of similar construction and operation to justify the efficiency calculations for the filters. Filter efficiency formula from a reliable source acceptable to the commission may be used.*

Analysis: Required BOD_5 reduction = [(65%)(200 mg/l)-(20 mg/l)] / [(65%)(200 mg/l)]
= 84.6% BOD_5 reduction

From Munter's BioDeck® 19060 literature for 85% BOD_5 reduction, use a loading of 63 lbs BOD_5/1,000 ft^3/day.

Volume Required
= (1.99 mgd)(8.345)(200 mg/l BOD_5)(65%)/(63 lb BOD_5/1000 ft^3/day

= 34,267 ft^3 media

Improvements: **Upgrade existing 100 ft. diameter x 6 ft. deep trickling filter with new synthetic media and distributor. Provide for 2 x ADF recirculation.**

D. Final Clarifier.

Required: *Maximum surface loading at Peak flow of 1600 gal./day/ft^2, and at Design flow of 800 gal./day/ft^2. Side water depth must be at least 10 ft. for surface areas of 1250 ft^2 or more. Effective detention times (based on liquid volume above a 3 ft. sludge blanket) must be 1.1 hr. @ Peak flow and 2.2 hr. @ Design flow.*

Analysis: Required area based on surface area:
@ Peak flow = 5,970,000 gpd / 1600 gal./day/ft^2 = 3,731 ft^2
@ Design flow = 1,990,000 gpd / 800 gal./day/ft^2 = 2,488 ft^2

Required area based on detention time:
Detention time is based on side water depth of 14 ft. less 3 ft. sludge blanket

@ Peak flow = (5,970,000 gpd) (1.1 hrs.) / [(24 hrs/day)(7.48 gal/ft^3)(11ft.)] = 3,326 ft^2

@ Design flow = (1,990,000) gpd) (2.2 hrs)/[(24 hrs/day) (7.48 gal/ft^3) 11ft.)] = 2,217 ft^2

Required area based on Peak flow detention requirements govern.

Improvements: **Construct one (1) 82 ft. diameter final clarifier with 14 ft. side water depth. Provide sludge collection/pumping.**

E. Effluent Works.

1. Chlorine Contact Chamber.

Required: *Detention time of 20 minutes @ peak flow.*

Analysis: (5,970,000 gpd/1440 min/day) (20 min) / (7.48 gal/ft^3) = 11,085 ft^3

Improvements: **Construct a 11,085 ft^3 chlorine contact chamber.**

2. Chlorine Feed Equipment.

Required: *Feed equipment must be able to provide more than the highest dosage to be required at any time. Dosage must be adequate to maintain a chlorine residual of at least 1 mg/l after 20 minutes detention, prior to dechlorination.*

Improvements: **Provide chlorine feed equipment as necessary to provide for chlorination of 5.97 mgd.**

3. Dechlorination.

Required: *The effluent, after chlorinatioin and 20 minutes detention time, must be dechlorinated to less than 0.1 mg/l. For most dechlorination agents, 1 minute detention is generally considered adequate.*

Analysis: 5,970,000 gpd/1440 min/day) (1 min) / (7.48 gal/ft^3) = 554 ft^3

Improvements: **Construct a 554 ft^3 dechlorination chamber and provide sodium bisulfate feed equipment as necessary to provide for dechlorination 5.97 mgd.**

4. Flow Measurement.

Required: *Continuous effluent measurement required, with capacity for*

maximum expected peak flow.

Improvements: **Construct a parshall flume capable of measuring flows up to 6 mgd.**

5. Postaeration.

Improvements: **Construct a passive, cascade type aeration structure on the discharge from the constructed wetland capable of producing a 4 mg/l dissolved oxygen effluent.**

6. Effluent Lift Station.

Improvements: **See Alternate PN/G-1 or PN/G-2.**

F. Sludge Processing.

1. Sludge Thickener.

Required: *Aerobic digesters should be provided with sludge thickening.*

Improvements: **Provide piping in the digester to allow for settling and decanting for sludge thickening.**

2. Aerobic Digesters.

Requirement: *Minimum solids retention time of 15 days (may be calculated as 20 ft^3 for each lb. influent BOD_5 per day). Diffused air requirement is 30 cfm per 1000 ft^3 of volume.*

Analysis: Digester volume required = 20 ft^3 / lb. influent BOD_5 per day
Influent BOD_5 = (1.99 mgd)(8.345)(200 mg/l) = 3321 lb. BOD_5/day

Required Digester Volume = (20)(3321) = 66,420 ft^3

Proposed Digester = 22 ft. SWD
= [66,420 ft^3 / 22 ft.]/2 basins
= 1,510 ft^2/basin = 39 ft. x 39 ft. ea.

Required aeration = (30 cfm/1000 ft^3)(66,420 ft^3) = 1993 cfm

Improvements: **Construct a deep tank type aerobic digester with two (2) basins (39 ft. x 39 ft. x 22 ft. SWD ea.) and provide 1993 cfm aeration equipment capacity.**

3. Sludge Dewatering Facility.

Required: *Sludge shall be dewatered sufficiently to meet the requirements of the ultimate form of disposal.*

Improvements: **Provide a means of sludge dewatering.**

G. Blowers.

Required: *Blowers must be able to meet maximum aeration requirements with largest unit out of service.*

Improvements: **Provide blowers/aeration equipment as necessary.**

H. Opinion of Probable Costs - Phase I (North Side Flows Only)

1.	Influent headworks, screens	$	146,000
2.	Influent lift station	$	72,000
3.	Primary clarifier	$	419,000
4.	Trickling filter	$	273,000
5.	Recirculation Pumps	$	45,000
6.	Final Clarifier	$	476,000
7.	Chlorination/dechlorination chamber/feed equipment	$	345,600
8.	Effluent flow measurement and aeration	$	40,000
9.	Aerobic digestor	$	399,500
10.	Blowers for digester	$	150,000
11.	Sludge dewatering facilities	$	190,000
12.	Yard piping improvements	$	355,000
13.	Site work	$	316,000
14.	Electrical and instrumentation	$	187,000
15.	Laboratory/Office	$	100,000
16.	Site Acquisition	$	45,000
	Subtotal	$	3,559,100
	Contingency (15%)	$	533,900
	TOTAL OPINION OF PROBABLE PHASE I CONSTR. COST	$	**4,093,000**

G2 - City of Groves: **North Plant AS 10/15**

Abandon existing WWTF, construct a new activated sludge treatment facility for full treatment of all flows and then discharge into the Neches River at the following effluent limits.

ADF	=	1.99 mgd
2-Hour Peak Flow	=	4,146 gpm (5.97 mgd)
BOD_5	=	20 mg/l
TSS	=	20 mg/l
NH_3	=	no limit
D.O.	=	4 mg/l

A. Preliminary Treatment

1. Screening

Required: *Some form of screening; bar openings minimum ½" for mechanical screens; velocities @ design flow minimum 2 ft./sec through channel, < 3 ft./sec. through screen.*

Analysis: Design Flow = 1.99 mgd = 3.08 cfs

Improvements: **Construct a dual channel influent structure with one mechanical bar screen in one channel, sized for a design flow of 1.99 mgd and a peak flow of 5.97 mgd, and a fixed bar screen in the second channel.**

2. Influent Lift Station. Construct an influent lift station to lift the raw wastewater flows into the proposed treatment facility.

Required: *Firm pumping capacity (largest pump out of service) must be adequate to pump peak flow to destination; three or more pumps (or duplex pumps with automatic variable speed control) required.*

Analysis: Firm capacity = 4,146 gpm

Four (4) pumps of sufficient capacity to provide firm capacity of 4,146 gpm with largest pump out of service.

Improvements: **Construct an influent lift station with firm pumping capacity of 4,146 gpm.**

3. Grit Removal

Required: *Grit removal recommended; if removal units are provided, must have method of removing grit from unit, and any unit with single chamber must have bypass.*

Improvements: **Construct a grit removal system rated at a design flow 1.99 mgd and a peak flow of 5.97 mgd.**

B. Activated Sludge Process. Construct a single stage nitrification, activated sludge unit.

Required: *Total volume shall be 1000 ft^3 per 35 lb. BOD_5/day. Diffused aeration shall be designed for 3200 SCF per lb. BOD_5. The diffuser system must be capable of providing 150% of design requirements.*

Analysis: Use organic loading of 30 lbs. BOD_5/day (Conservative loading)

lbs. BOD_5/day = (1.99 mgd)(8.345)(200 mg/l) = 3,321 lbs. BOD_5/day

Required Volume = (3321 lb BOD_5/day)(1000 ft^3)/30 lb BOD_5/day
= 110,700 ft^3

Air Requirements = (3321 lbs. BOD_5/day)(3200 SCF/lbs. BOD_5)
= 10,627,200 SCF/day
= 7380 cfm

150% Design Req. = (7380 cfm)(1.5)
= 11,070 cfm

Improvements: **Construct a 110,700 ft^3 activated sludge aeration basin and provide 11,070 cfm aeration capacity.**

C. Final Clarifiers.

Required: *Maximum surface loading at Peak flow of 1200 gal./day/ft^2, and at Design flow of 600 gal./day/ft^2. Side water depth must be at least 10 ft. for surface areas of 1250 ft^2 or more. Effective detention times (based on liquid volume above a 3 ft. sludge blanket) must be 1.5 hr. @ Peak flow and 3.0 hr. @ Design flow.*

Analysis: Required area based on surface area:
@ Peak flow = 5,970,000 gpd / 1200 gal./day/ft^2 = 4,675 ft^2
@ Design flow = 1,990,000 gpd / 600 gal./day/ft^2 = 3,317 ft^2

Required area based on detention time:
Detention time is based on side water depth of 14 ft. less 3 ft. sludge

blanket

@ Peak Flow = (5,970,000 gpd)(1.5 hrs.)/[(24 hrs/day)(7.48 gal/ft^3)(11 ft.)]
= 4,535 ft^2

@ Design Flow = (1,990,000 gpd)(3.0 hrs.)/[(24 hrs/day)(7.48 gal/ft^3)(11 ft.)]
= 3,023 ft^2

Required area based on Peak flow surface loading requirements govern.

Improvements: **Construct one (1) 78 ft. diameter final clarifier with 14 ft. side water depth. Provide sludge collection/pumping.**

D. Effluent Works.

1. Chlorine Contact Chamber.

Required: *Detention time of 20 minutes @ peak flow.*

Analysis: (5,970,000 gpd/1440 min/day)(20 min)/(7.48 gal/ft^3) = 11,085 ft^3

Improvements: **Construct a 11,085 ft^3 chlorine contact chamber.**

2. Chlorine Feed Equipment.

Required: *Feed equipment must be able to provide more than the highest dosage to be required at any time. Dosage must be adequate to maintain a chlorine residual of at least 1 mg/l after 20 minutes detention, prior to dechlorination.*

Improvements: **Provide chlorine feed equipment as necessary to provide for chlorination of 5.97 mgd.**

3. Dechlorination.

Required: *The effluent, after chlorination and 20 minutes detention time, must be dechlorinated to less than 0.1 mg/l. For most dechlorination agents, 1 minute detention is generally considered adequate.*

Analysis: (5,970,000 gpd/1440 min/day)(1 min)/(7.48 gal/ft^3) = 554 ft^3

Improvements: **Construct a 554 ft^3 dechlorination chamber and provide sodium bisulfate feed equipment as necessary to provide for dechlorination of 5.97 mgd.**

4. Flow Measurement.

Required: *Continuous effluent measurement required, with capacity for maximum expected peak flow.*

Improvements: **Construct a parshall flume capable of measuring flows up to 6 mgd.**

5. Postaeration.

Improvements: **Construct a passive, cascade type aeration structure on the discharge from the new treatment plant capable of producing a 4 mg/l dissolved oxygen effluent.**

6. Effluent Lift Station and Effluent Force Main.

Improvements: **See Alternate PN/G-1 or PN/G2.**

E. Sludge Processing.

1. Sludge Thickener.

Required: *Aerobic digesters should be provided with sludge thickening.*

Improvements: **Provide piping in the digester to allow for settling and decanting for sludge thickening.**

2. Aerobic Digesters.

Requirement: *Minimum solids retention time of 15 days (may be calculated as 20 ft^3 for each lb. influent BOD_5 per day). Diffused air requirement is 30 cfm per 1000 ft^3 of volume.*

Analysis: Digester volume required = 20 ft^3 / lb. influent BOD_5 per day
Influent BOD_5 = (1.99 mgd)(8.345)(200 mg/l) = 3321 lb. BOD_5/day

Required Digester Volume = (20)(3321) = 66,420 ft^3

Proposed Digester = 22 ft. SWD

$= [66{,}420 \text{ ft}^3 / 22 \text{ ft.}]/2$ basins
$= 1{,}510 \text{ ft}^2/\text{basin} = 39$ ft. x 39 ft. ea.

Required aeration = $(30 \text{ cfm}/1000 \text{ ft}^3)(66{,}420 \text{ ft}^3) = 1993$ cfm

Improvements: **Construct a deep tank type aerobic digester with two (2) basins (39 ft. x 39 ft. x 22 ft. SWD ea.) and provide 1993 cfm aeration equipment capacity.**

3. Sludge Dewatering Facility.

Required: *Sludge shall be dewatered sufficiently to meet the requirements of the ultimate form of disposal.*

Improvements: **Provide a means of sludge dewatering.**

F. Blowers.

Required: *Blowers must be able to meet maximum aeration requirements with largest unit out of service.*

Improvements: **Provide blowers/aeration equipment as necessary.**

G. Opinion of Probable Cost

1.	Influent headworks, screens	$	122,200
2.	Influent lift station	$	96,700
3.	Grit removal system	$	87,900
4.	Activated sludge basin(s)	$	644,000
5.	Final Clarifier	$	440,000
6.	Chlorination/dechlorination chamber/feed equipment	$	196,000
7.	Effluent flow measurement and aeration	$	27,500
8.	Aerobic digester	$	348,300
9.	Sludge dewatering facilities	$	196,200
10.	Aeration blower equipment	$	426,400
11.	Sludge pumping equipment	$	105,000
12.	Yard piping improvements	$	560,000
13.	Electrical and instrumentation	$	700,000
14.	Office/Laboratory building	$	75,000
15.	Site work	$	560,000
	Subtotal	$	4,585,200
	Contingency (15%)	$	687,800
	TOTAL OPINION OF PROBABLE CONSTRUCTION COST	$	**5,273,000**

G3 - City of Groves: South Plant AS 10/15

Abandon existing WWTF, construct a new activated sludge treatment facility for full treatment of all flows at the following effluent limits.

ADF	=	3.33 mgd
2-Hour Peak Flow	=	12,986 gpm (18.70 mgd)
BOD_5	=	20 mg/l
TSS	=	20 mg/l
NH_3	=	no limit
D.O.	=	5 mg/l

A. Preliminary Treatment

1. Influent Lift Station All influent flow to the South WWTF are pumped from the Taft Avenue lift station. This lift station will have to be upgraded to provide a firm capacity of 12,986 gpm (18.70 mgd). Upgrading of this lift station is addressed in detail under the proposed collection system improvements for the City of Groves.

2. Screening

Required: *Some form of screening; bar openings minimum ½" for mechanical screens; velocities @ design flow minimum 2 ft./sec through channel, < 3 ft./sec. through screen.*

Analysis: Design Flow = 18.70 mgd = 28.93 cfs

Improvements: **Construct a dual channel influent structure with one mechanical bar screen in one channel, sized for a design flow of 3.33 mgd and a peak flow of 18.70 mgd, and a fixed bar screen in the second channel.**

3. Grit Removal

Required: *Grit removal recommended; if removal units are provided, must have method of removing grit from unit, and any unit with single chamber must have bypass.*

Improvements: **Construct a grit removal system rated at a design flow 3.33 mgd and a peak flow of 18.70 mgd.**

B. Activated Sludge Process. Construct a single stage nitrification, activated sludge unit.

Required: *Total volume shall be 1000 ft^3 per 35 lb. BOD_5/day. Diffused aeration* *shall be designed for 3200 SCF per lb. BOD_5. The diffuser system must be capable of providing 150% of design requirements.*

Analysis: Use BOD_5 loading of 30 lb BOD_5/ 1000 ft^3. (Conservative loading)

lbs. BOD_5/day = (3.33 mgd)(8.345)(200 mg/l) = 5558 lbs. BOD_5/day

Required Volume = (5558 lb BOD_5/day)(1000 ft^3)/30 lb BOD_5/day
= 185,267 ft^3

Air Requirements = (5558 lbs. BOD_5/day)(3200 SCF/lbs. BOD_5)
= 17,785,600 SCF/day
= 12,351 cfm

150% Design Req. = (12,351 cfm)(1.50)
= 18,527 cfs

Improvements: Construct a 185,267 ft^3 activated sludge aeration basin and provide 18,527 cfm aeration capacity.

C. Final Clarifiers.

Required: *Maximum surface loading at Peak flow of 1200 gal./day/ft^2, and at Design flow of 600 gal./day/ft^2. Side water depth must be at least 10 ft. for surface areas of 1250 ft^2 or more. Effective detention times (based on liquid volume above a 3 ft. sludge blanket) must be 1.5 hr. @ Peak flow and 3.0 hr. @ Design flow.*

Analysis: Required area based on surface area:
@ Peak flow = 18,700,000 gpd / 1200 gal./day/ft^2 = 15,583 ft^2
@ Design flow = 3,330,000 gpd / 600 gal./day/ft^2 = 5,550 ft^2

Required area based on detention time:
Detention time is based on side water depth of 14 ft. less 3 ft. sludge blanket

@ Peak Flow = (18,700,000 gpd)(1.5 hr.)/[(24 hrs/day)(7.48 gal/ft^3)(11 ft.)]
= 14,205 ft^2

@ Design Flow = (3,330,000 gpd)(3.0 hrs.)/[(24 hrs/day)(7.48 gal/ft^3)(11 ft.)]
= 5,059 ft^2

Required area based on Peak flow surface loading requirements govern.

Improvements: **Construct two (2) 100 ft. diameter final clarifiers with 14 ft. side water depth. Provide sludge collection/pumping.**

D. Effluent Works.

1. Chlorine Contact Chamber.

Required: *Detention time of 20 minutes @ peak flow.*

Analysis: (18,700,000 gpd/1440 min/day)(20 min)/(7.48 gal/ft^3) = 34,722 ft^3

Improvements: **Construct a 34,722 ft^3 chlorine contact chamber.**

2. Chlorine Feed Equipment.

Required: *Feed equipment must be able to provide more than the highest dosage to be required at any time. Dosage must be adequate to maintain a chlorine residual of at least 1 mg/l after 20 minutes detention, prior to dechlorination.*

Improvements: **Provide chlorine feed equipment as necessary to provide for chlorination of 18.70 mgd.**

3. Dechlorination.

Required: *The effluent, after chlorination and 20 minutes detention time, must be dechlorinated to less than 0.1 mg/l. For most dechlorination agents, 1 minute detention is generally considered adequate.*

Analysis: (18,700,000 gpd/1440 min/day)(1 min)/(7.48 gal/ft^3) = 1,736 ft^3

Improvements: **Construct a 1,736 ft^3 dechlorination chamber and provide sodium bisulfate feed equipment as necessary to provide for dechlorination of 18.70 mgd.**

4. Flow Measurement.

Required: *Continuous effluent measurement required, with capacity for maximum expected peak flow.*

Improvements: **Construct a parshall flume capable of measuring flows**

up to 20 mgd.

5. Postaeration.

Improvements: **Construct a passive, cascade type aeration structure on the discharge from the new treatment plant capable of producing a 5 mg/l dissolved oxygen effluent.**

E. Sludge Processing.

1. Sludge Thickener.

Required: *Aerobic digesters should be provided with sludge thickening.*

Improvements: **Construct piping in the digester to allow for settling and decanting for thickening.**

2. Aerobic Digesters.

Requirement: *Minimum solids retention time of 15 days (may be calculated as 20 ft^3 for each lb. influent BOD_5 per day). Diffused air requirement is 30 cfm per 1000 ft^3 of volume.*

Analysis: Digester volume required = 20 ft^3 / lb. influent BOD_5 per day
Influent BOD_5 = (3.33 mgd)(8.345)(100 mg/l) = 5558 lb.
BOD_5/day

Required Digester Volume = (20)(5558) = 111,160 ft^3

Proposed Digester = 22 ft. SWD
= [111,160 ft^3 / 22 ft.]/2 basins
= 2,526 ft^2 = 50 ft. x 50 ft. ea.

Required aeration = (30 cfm/1000 ft^3)(111,160 ft^3) = 3335 cfm

Improvements: **Construct a deep tank type aerobic digester with two (2) basins (50 ft. x 50 ft. x 22 ft. SWD ea.) and provide 3335 cfm aeration equipment capacity.**

3. Sludge Dewatering Facility.

Required: *Sludge shall be dewatered sufficiently to meet the requirements of the ultimate form of disposal.*

Improvements: **Provide a means of sludge dewatering.**

F. Blowers.

Required: *Blowers must be able to meet maximum aeration requirements with largest unit out of service.*

Improvements: **Provide blowers/aeration equipment as necessary.**

G. Opinion of Probable Cost

1.	Upgrade Taft Street Lift Station	$	802,000
2.	Influent headworks, screens	$	149,700
3.	Grit removal system	$	94,100
4.	Activated sludge basin(s)	$	929,000
5.	Final Clarifier	$	1,155,400
6.	Chlorination/dechlorination chamber/feed equipment	$	399,600
7.	Effluent flow measurement and aeration	$	27,500
8.	Aerobic digestor	$	504,100
9.	Sludge dewatering facilities	$	328,300
10.	Aeration blower equipment	$	560,000
11.	Sludge Pumps	$	106,000
12.	Yard piping improvements	$	810,000
13.	Electrical and instrumentation	$	920,000
14.	Office/Laboratory	$	85,000
15.	Site Work	$	735,000
	Subtotal	$	7,605,700
	Contingency (15%)	$	1,141,300
	TOTAL OPINION OF PROBABLE CONSTRUCTION COST	$	**8,747,000**

G4- City of Groves: Upgrade South WWTF's Trickling Filter Treatment Facility Discharge to the Sabine Neches Canal.

Abandon existing WWTF, construct a new trickling filter treatment facility for full treatment of all flows and then discharge into the Neches River at the following effluent limits.

ADF	=	3.33 mgd
2-Hour Peak	=	12,986 gpm (18.7 mgd)
BOD_5	=	20 mg/l
TSS	=	20 mg/l
NH_3	=	no limit
D.O.	=	5 mg/l

A. Preliminary Treatment

1. Screening.

Required: *Some form of screening; bar openings minimum ½" for mechanical screens; velocities @ design flow minimum 2ft./sec through channel, < 3 ft./sec. through screen.*

Analysis: Design Flow = 3.33 mgd = 5.15 cfs

Improvements: **Construct a dual channel influent structure with one mechanical bar screen in one channel, sized for a design flow of 3.33 mgd and a peak flow of 18.7 mgd, and a fixed bar screen in the second channel.**

2. Influent Lift Station. Construct an influent lift station to lift the raw wastewater flows into the proposed treatment facility.

Required: *Firm pumping capacity (largest pump out of service) must be adequate to pump peak flow to destination; three or more pumps (or duplex pumps with automatic variable speed control) required.*

Analysis: Firm capacity = 12,986 gpm

B. Primary Clarifier.

Required: Maximum surface loading at Peak flow of 1800 gal./day/ft^2, and at Design flow of 1000 gal./day/ft^2. Side water depth must be at least 7 ft.

Analysis: Required area:
@ Peak flow = 18,700,000 gpd / 1800 gal./day/ft^2 = 10,389 ft^2

@ Design flow = 3,330,000 gpd / 1000 gal./day/ft^2 = 3,330 ft^2

Improvements: **Construct two (2) 82 ft. diameter primary clarifiers with 14 ft. side water depth. Provide sludge collection/pumping.**

C. Trickling Filter.

Required: *Application of synthetic media shall be evaluated on a case-by-case basis. The design engineer shall submit sufficient operating data from existing trickling filters of similar construction and operation to justify the efficiency calculations for the filters. Filter efficiency formula from a reliable source acceptable to the commission may be used.*

Analysis: Required BOD_5 reduction = [(65%)(200 mg/l)-(20 mg/l)] / [(65%)(200 mg/l)]

= 84.6% BOD_5 reduction

From Munter's BioDeck® 19060 literature for 85% BOD_5 reduction, use a loading of 63 lbs BOD_5/1,000 ft^3/day.

Volume Required

= (3.33 mgd)(8.345)(200 mg/l BOD_5)(65%)/(63 lb BOD_5/1000 ft^3/day

= 57,342 ft^3 media (Existing Volume = 46,651 ft^3)

Improvements: **Construct one (1) new 60 ft. diameter x 5.5 ft. deep trickling filter with synthetic media. Upgrade existing 60 ft. diameter x 5.5 ft. deep trickling filters (3) with new synthetic media and distributor. Provide for 2 x ADF recirculation.**

D. Final Clarifier.

Required: *Maximum surface loading at Peak flow of 1600 gal./day/ft^2, and at Design flow of 800 gal./day/ft^2. Side water depth must be at least 10 ft. for surface areas of 1250 ft^2 or more. Effective detention times (based on liquid volume above a 3 ft. sludge blanket) must be 1.1 hr. @ Peak flow and 2.2 hr. @ Design flow.*

Analysis: Required area based on surface area:

@ Peak flow = 18,700,000 gpd / 1600 gal./day/ft^2 = 11,687 ft^2

@ Design flow = 3,300,000 gpd / 800 gal./day/ft^2 = 4,163 ft^2

Required area based on detention time:

Detention time is based on side water depth of 14 ft. less 3 ft. sludge blanket

@ Peak flow = (18,700,000 gpd) (1.1 hrs.) / [(24 hrs/day)(7.48 gal/ft^3)(11ft.)] = 10,417 ft^2

@ Design flow = (3,330,000) gpd) (2.2 hrs)/[(24 hrs/day) (7.48 gal/ft^3) (11ft.)] = 3,710 ft^2

Required area based on Peak flow, surface area requirements govern.

Improvements: **Construct three (3) 87 ft. diameter final clarifiers with 14 ft. side water depth. Provide sludge collection/pumping.**

E. Effluent Works.

1. Chlorine Contact Chamber.

Required: *Detention time of 20 minutes @ peak flow.*

Analysis: (18,700,000 gpd/1440 min/day) (20 min) / (7.48 gal/ft^3) = 34,722 ft^3

Improvements: **Construct a 34,722 ft^3 chlorine contact chamber.**

2. Chlorine Feed Equipment.

Required: *Feed equipment must be able to provide more than the highest dosage to be required at any time. Dosage must be adequate to maintain a chlorine residual of at least 1 mg/l after 20 minutes detention, prior to dechlorination.*

Improvements: **Provide chlorine feed equipment as necessary to provide for chlorination of 18.70 mgd.**

3. Dechlorination.

Required: *The effluent, after chlorination and 20 minutes detention time, must be dechlorinated to less than 0.1 mg/l. For most dechlorination agents, 1 minute detention is generally considered adequate.*

Analysis: 18,700,000 gpd/1440 min/day) (1 min) / (7.48 gal/ft^3) = 1,736 ft^3

Improvements: **Construct a 1,736 ft^3 dechlorination chamber and provide sulphur dioxide feed equipment as necessary to provide for dechlorination 18.7 mgd.**

4. Flow Measurement.

Required: *Continuous effluent measurement required, with capacity for maximum expected peak flow.*

Improvements: **Construct a parshall flume capable of measuring flows up to 20 mgd.**

5. Postaeration.

Improvements: **Construct a passive, cascade type aeration structure on the discharge capable of producing a 5 mg/l dissolved oxygen effluent.**

6. Effluent Lift Station.

Improvements: **Construct an effluent lift station capable of discharging 18.7 mgd to the Sabine Neches Canal considering high water levels equal to the height of the hurricane protection levee or 14' MLSL.**

F. Sludge Processing.

1. Sludge Thickener.

Required: *Digesters should be provided with sludge thickening.*

Improvements: **Sludge thickening will be provided by decanting inside the digestors.**

2. Aerobic Digesters.

Requirement: *Minimum solids retention time of 15 days (may be calculated as 20 ft^3 for each lb. influent BOD_5 per day). Diffused air requirement is 30 cfm per 1000 ft^3 of volume.*

Analysis: Digester volume required = 20 ft^3 / lb. influent BOD_5 per day

lbs. BOD_5 = (3.33 mgd) (8.345) (200 mg/l)
= 5,558 lbs. BOD_5/day

Required Digester Volume = (20) (5,558) = 111,160 ft^3

Proposed Digesters (2) = 22 ft. SWD
= 111,160 ft^3 / (22ft x 2)
= 2526 ft^2
= 51 ft x 51 ft

Required aeration = (30 cfm/1000 ft^3) (111,160 ft^3) = 3,335 cfm

Improvements: **Construct two (2) deep tank type aerobic digesters (51 ft. x 51 ft. x 22 ft. SWD) and provide 3,335 cfm aeration equipment capacity.**

3. Sludge Dewatering Facility.

Required: *Sludge shall be dewatered sufficiently to meet the requirements of the ultimate form of disposal.*

Improvements: **Provide a means of sludge dewatering.**

G. Blowers.

Required: *Blowers must be able to meet maximum aeration requirements with largest unit out of service.*

Analysis:	Pre Aeration	=	294 cfm
	Aerobic Digestion	=	3,335 cfm
			3,629 cfm

Improvements: **Provide blowers as required for Pre aeration and aerobic digestion.**

H. Opinion of Probable Costs

1.	Upgrade Taft Street Lift Station		$	802,000
2.	Influent Headworks, Screens		$	149,700
3.	Primary Clarifier		$	845,700
4.	Trickling Filter Upgrade		$	520,000
5.	Final Clarifier		$	903,400
6.	Chlorination/Dechlorination		$	399,600
7.	Effluent flow measurement		$	27,500
8.	Aerobic Digester		$	504,100
9.	Sludge Dewatering		$	328,300
10.	Blowers for Digester		$	210,000
11.	Waste Sludge Pumps		$	106,000
12.	Yard Piping		$	810,000
13.	Electrical & Instrumentation		$	920,000
14.	Office/Laboratory		$	85,000
15.	Site Work		$	735,000
		Subtotal	$	7,346,300
		Contingency (15%)	$	1,101,700
	TOTAL OPINION OF PROBABLE CONSTRUCTION COST		$	8,448,000

G5 - City of Groves Regional WWTF: Abandon Existing WWTF's, Construct New Regional Activated Sludge Treatment Facility for the City of Groves adjacent to existing North WWTF

Abandon existing WWTF's, upgrade the Taft Street transfer lift station to pump the raw wastewater flow from the South collection system to a regional activated sludge treatment facility adjacent to the North WWTF site, and then divert discharge to the Neches River at the following effluent limits.

	Groves North	Groves South	Total
ADF	1.99 mgd	3.33 mgd	**5.32 mgd**
2-Hour Peak	5.97 mgd	18.70 mgd	**24.67 mgd**

BOD_5	=	20 mg/l
TSS	=	20 mg/l
NH_3	=	no limit
D.O.	=	4 mg/l

A. Transfer Lift Station

1. Groves South. Upgrade the Taft Street transfer lift station to pump the raw wastewater flows to the proposed regional wastewater treatment facility.

Required: *Firm pumping capacity (largest pump out of service) must be adequate to pump peak flow to destination; three or more pumps (or duplex pumps with automatic variable speed control) required.*

Analysis: Firm capacity = 18.70 mgd (12,986 gpm)

Four (4) pumps total; firm capacity of 12,986 gpm with largest pump out of service (i.e. 3 pumps pumping + 1 spare).

Transfer force main(s) sized for a minimum of 2 fps at low flow and a maximum 10 fps at peak flow.

Proposed force main = 30" diameter

Improvements: **Upgrade transfer lift station with four (4) pumps (firm capacity of 12,986 gpm) and a 30" diameter force main to the regional wastewater treatment facility.**

B. Preliminary Treatment

1. Screening

Required: *Some form of screening; bar openings minimum ½" for mechanical screens; velocities @ design flow minimum 2 ft./sec through channel, < 3 ft./sec. through screen.*

Analysis:
ADF = 5.32 mgd = 8.23 cfs
2-Hour Peak = 24.67 mgd = 38.17 cfs

Channel Width = 2 channels @ 5 ft. each

Screen Size = 5 ft. wide x 0.5 inch bars x 0.75 inch openings
Assumed Screen Efficiency = 60 %

Improvements: **Construct a two channel (5 ft. wide/channel) influent structure with one mechanical bar screen and one fixed bar screen.**

2. Grit Chamber

Required: *Grit removal recommended; if removal units are provided, must have method of removing grit from unit, and any unit with single chamber must have bypass.*

Analysis:
Detention time = 20 minutes @ ADF, 5 minutes @ Peak
Air requirements = 20-25 cfm / 1,000 ft^3
Draft tube = 25 cfm / 1,000 ft^3

Volume required:
@ ADF = (5,320,000 gpd)(20 min.)/(7.48 gal/ft^3)(1 day/1440 min.)
= 9,878 ft^3

@ Peak = (24,670,000 gpd)(5 min.)/(7.48 gal/ft^3)(1 day/1440 min.)
= 11,452 ft^3

Use square basin = 28 ft. x 28 ft. x 15 ft. SWD w/ 5:12 bottom slope

Air required:
(28 ft. x 28 ft. x 15 ft.)(25 cfm/1,000 ft^3) = 294 cfm

Draft tube required:
Area = (294 cfm)(1 ft^2/25cfm) = 11.8 ft^2
Use 4 ft. diameter tube

Improvements: **Construct 28 ft. x 28 ft. x 15 ft. SWD aerated grit chamber with 294 cfm aeration within a 4 ft. draft tube.**

C. Activated Sludge Process. Construct a single stage nitrification, activated sludge unit.

Required: *Total volume shall be 1000 ft^3 per 35 lb. BOD_5/day. Diffused aeration shall be designed for 3200 SCF per lb. BOD_5. The diffuser system must be capable of providing 150% of design requirements.*

Analysis: Use 30 lb. BOD_5/day per 1000 ft^3 aeration volume. (Conservative loading)

lbs. BOD_5/day = [(5.32 mgd)(8.345)(200 mg/l)]
= 8,879 lbs. BOD_5/day

Required Volume = (8,879 lb BOD_5/day)(1000 ft^3)/30 lb BOD_5/day
= 295,967 ft^3

Proposed basins ratio = 2 basins, 22 ft. deep SWD, w/ 2length:1width

= 2 x 22 ft. deep x 58 ft. wide x 116 ft. long
= 296,032 ft^3

Air Requirements = (8,879 lbs. BOD_5/day)(3200 SCF/lbs. BOD_5)
= 28,412,800 SCF/day
= 19,731 cfm

150% Design Req. = (19,731 cfm)(1.5)
= 29,597 cfs

Improvements: **Construct a dual activated sludge deep tank aeration basin (each 22 ft. deep x 58 ft. wide x 116 ft. long) and provide 29,597 cfm aeration capacity.**

D. Final Clarifiers.

Required: *Maximum surface loading at Peak flow of 1200 gal./day/ft^2, and at Design flow of 600 gal./day/ft^2. Side water depth must be at least 10 ft. for surface areas of 1250 ft^2 or more. Effective detention times (based on liquid volume above a 3 ft. sludge blanket) must be 1.5 hr. @ Peak flow and 3.0 hr. @ Design flow.*

Analysis: Required area based on surface area:
@ Peak flow = 24,670,000 gpd / 1200 gal./day/ft^2 = 20,558 ft^2
@ Design flow = 5,320,000 gpd / 600 gal./day/ft^2 = 8,867 ft^2

Required area based on detention time:
Detention time is based on side water depth of 14 ft. less 3 ft. sludge blanket

$$@\ \text{Peak Flow} = \frac{(24{,}670{,}000\ \text{gpd})(1.5\ \text{hrs.})}{(24\ \text{hrs/day})(7.48\ \text{gal/ft}^3)(11\ \text{ft.})} = 18{,}739\ \text{ft}^2$$

@ Design Flow = $\dfrac{(5,320,000 \text{ gpd})(3.0 \text{ hrs.})}{(24 \text{ hrs/day})(7.48 \text{ gal/ft}^3)(11 \text{ ft.})} = 8,082 \text{ ft}^2$

Required area based on Peak flow surface area requirements govern.

Two (2) - 116 ft. diameter clarifiers = 20,558 ft^2 effective surface area. 14 ft. side water depth.

Improvements: **Construct two (2) 116 ft. diameter final clarifiers with 14 ft. side water depths. Provide flow splitting/collection structures/piping, and sludge collection/pumping.**

E. Effluent Works.

1. Chlorine Contact Chamber.

Required: *Detention time of 20 minutes @ peak flow.*

Analysis: (24,670,000 gpd/1440 min/day)(20 min)/(7.48 gal/ft^3) = 45,807 ft^3

Basin Dimensions: 1 Basin

Width	= 30 ft./basin
Length	= 109 ft.
Depth	= 14.0 ft. SWD

Improvements: **Construct a chlorine contact chamber (30 ft. wide x 109 ft. long by 14 ft. SWD).**

2. Chlorine Feed Equipment.

Required: *Feed equipment must be able to provide more than the highest dosage to be required at any time. Dosage must be adequate to maintain a chlorine residual of at least 1 mg/l after 20 minutes detention, prior to dechlorination.*

Improvements: **Provide chlorine feed equipment as necessary to provide for chlorination of 24.67 mgd.**

3. Dechlorination.

Required: *The effluent, after chlorination and 20 minutes detention time, must be dechlorinated to less than 0.1 mg/l. For most dechlorination agents, 1 minute detention is generally considered adequate.*

Analysis: (24,670,000 gpd/1440 min/day)(1 min)/(7.48 gal/ft^3) = 2290 ft^3

Basin Dimensions: 1 Basin

Length	= 30 ft.
Width	= 6.5 ft.
Depth	= 12.0 ft. SWD

Improvements: **Construct a dechlorination chamber (30 ft. long x 6.5 ft. wide x 12 ft. SWD) and provide chemical feed equipment as necessary to provide for dechlorination of 24.67 mgd.**

4. Flow Measurement.

Required: *Continuous effluent measurement required, with capacity for maximum expected peak flow.*

Improvements: **Construct a parshall flume capable of measuring flows up to 30 mgd.**

5. Postaeration.

Improvements: **Construct a passive, cascade type aeration structure on the discharge capable of producing a 4 mg/l dissolved oxygen effluent.**

6. Effluent Lift Station and Effluent Force Main

Improvements: **See Alternate D5(B)**

F. Sludge Processing.

1. Sludge Thickener.

Required: *Digesters should be provided with sludge thickening.*

Improvements: **Sludge thickening will be provided by decanting inside the digestors.**

2. Aerobic Digesters.

Requirement: *Minimum solids retention time of 15 days (may be calculated as 20 ft^3 for each lb. influent BOD_5 per day). Diffused air requirement is 30 cfm per 1000 ft^3 of volume.*

Analysis: Digester volume required = 20 ft^3 / lb. influent BOD_5 per day

lbs. BOD_5/day = (5.32 mgd)(8.345)(200 mg/l)

= 8,879 lbs. BOD_5/day

Required Digester Volume = (20)(8,879) = 177,580 ft^3

Proposed Digesters (2) = 22 ft. SWD
= 177,580 ft^3 / (22 ft. x 2)
= 4,036 ft^2
= 64 ft. x 64 ft.

Required aeration = (30 cfm/1000 ft^3)(177,580 ft^3) = 5,327 cfm

Improvements: **Construct two (2) deep tank type aerobic digesters (64 ft. x 64 ft. x 22 ft. SWD) and provide 5,327 cfm aeration equipment capacity.**

3. Sludge Dewatering Facility.

Required: *Sludge shall be dewatered sufficiently to meet the requirements of the ultimate form of disposal.*

Improvements: **Provide sludge dewatering facilities.**

G. Blowers.

Required: *Blowers must be able to meet maximum aeration requirements with largest unit out of service.*

Analysis:

Aerated Grit Chamber	=	294 cfm
Activated Sludge Aeration	=	19,731 cfm
Aerobic Digestion	=	5,327 cfm
	=	25,352 cfm

Improvements: **Provide blowers as required for activated sludge aeration, aerobic digestion and other needs.**

H. Opinion of Probable Cost

1.	Transfer lift station/force main - Taft Lift Station	$	2,090,000
2.	Influent lift station (N.Side flows)	$	72,000
3.	Influent headworks, screens	$	146,000
4.	Grit Chamber	$	198,000
5.	Activated sludge basin(s)	$	1,395,000
6.	Final Clarifiers	$	1,521,000
7.	Chlorination/dechlorination/feed equipment	$	378,000
8.	Effluent flow measurement/post-aeration	$	40,000
9.	Aerobic digestors	$	745,000
10.	Sludge dewatering facilities	$	495,000
11.	Aeration blower equipment	$	610,000
12.	Yard piping	$	925,000
13.	Site work	$	825,000
14.	Electrical and instrumentation	$	1,010,000
15.	Laboratory/Office	$	100,000
16.	Site Acquisition	$	45,000
	Subtotal	$	10,595,000
	Contingency (15%)	$	1,589,000
	TOTAL OPINION OF PROBABLE CONSTRUCTION COST	**$**	**12,184,000**

G6 - City of Groves Regional WWTF: Abandon Existing WWTF's, Construct New Regional Activated Sludge Treatment Facility for the City of Groves adjacent to existing South WWTF

Abandon existing WWTF's, upgrade the Taft Street transfer lift station to pump the raw wastewater flow from the South collection system to the regional activated sludge treatment facility at the South WWTF site, and discharge to the Sabine Neches Canal at the following effluent limits.

	Groves North	Groves South	Total
ADF	1.99 mgd	3.33 mgd	**5.32 mgd**
2-Hour Peak	5.97 mgd	18.70 mgd	**24.67 mgd**

BOD_5	=	20 mg/l
TSS	=	20 mg/l
NH_3	=	no limit
D.O.	=	5 mg/l

A. Transfer Lift Station

1. a. Groves South. Upgrade the Taft Street transfer lift station to pump the raw wastewater flows to the proposed regional wastewater treatment facility.

Required: *Firm pumping capacity (largest pump out of service) must be adequate to pump peak flow to destination; three or more pumps (or duplex pumps with automatic variable speed control) required.*

Analysis: Firm capacity = 18.70 mgd (12,986 gpm)

Four (4) pumps total; firm capacity of 12,986 gpm with largest pump out of service (i.e. 3 pumps pumping + 1 spare).

Transfer force main(s) sized for a minimum of 2 fps at low flow and a maximum 10 fps at peak flow.

Proposed force main = 30" diameter

Improvements: **Upgrade transfer lift station with four (4) pumps (firm capacity of 12,986 gpm) and a 30" diameter force main to the regional wastewater treatment facility.**

b. <u>Groves North.</u> Upgrade the North WWTF lift station to pump the raw wastewater flows to the proposed regional wastewater treatment facility.

Analysis: Firm capacity = 5.97 mgd (4146 gpm)

Four (4) pumps total; firm capacity of 4146 gpm with largest pump out of service.

Transfer force main sized for a minimum 2 fps velocity at low flow and maximum 10 fps at peak flow.

Proposed force main = 20'

Improvements: Upgrade North lift station with four (4) pumps, firm capacity of 4,146 gpm and a 20" diameter force main from the present North WWTF to the South WWTF.

B. Preliminary Treatment

1. Screening

Required: Some form of screening; bar openings minimum ½" for mechanical screens; velocities @ design flow minimum 2 ft./sec through channel, < 3 ft./sec. through screen.

Analysis:

ADF	= 5.32 mgd	= 8.23 cfs
2-Hour Peak	= 24.67 mgd	= 38.17 cfs

Channel Width = 2 channels @ 5 ft. each

Screen Size = 5 ft. wide x 0.5 inch bars x 0.75 inch openings
Assumed Screen Efficiency = 60 %

Improvements: Construct a two channel (5 ft. wide/channel) influent structure with one mechanical bar screen and one fixed bar screen.

2. Grit Chamber

Required: Grit removal recommended; if removal units are provided, must have method of removing grit from unit, and any unit with single chamber must have bypass.

Analysis: Detention time = 20 minutes @ ADF, 5 minutes @ Peak
Air requirements = 20-25 cfm / 1,000 ft^3
Draft tube = 25 cfm / 1,000 ft^3

Volume required:
@ ADF = (5,320,000 gpd)(20 min.)/(7.48 gal/ft^3)(1 day/1440 min.)
= 9,878 ft^3

@ Peak = (24,670,000 gpd)(5 min.)/(7.48 gal/ft^3)(1 day/1440 min.)
= 11,452 ft^3

Use square basin = 28 ft. x 28 ft. x 15 ft. SWD w/ 5:12 bottom slope

Air required:
(28 ft. x 28 ft. x 15 ft.)(25 cfm/1,000 ft^3) = 294 cfm

Draft tube required:
Area = (294 cfm)(1 ft^2/25cfm) = 11.8 ft^2
Use 4 ft. diameter tube

Improvements: **Construct 28 ft. x 28 ft. x 15 ft. SWD aerated grit chamber with 294 cfm aeration within a 4 ft. draft tube.**

C. Activated Sludge Process. Construct a single stage nitrification, activated sludge unit.

Required: *Total volume shall be 1000 ft^3 per 35 lb. BOD_5/day. Diffused aeration shall be designed for 3200 SCF per lb. BOD_5. The diffuser system must be capable of providing 150% of design requirements.*

Analysis: Use 30 lb. BOD_5/day per 1000 ft^3 aeration volume

lbs. BOD_5/day = [(5.32 mgd)(8.345)(200 mg/l)]
= 8,879 lbs. BOD_5/day

Required Volume = (8,879 lb BOD_5/day)(1000 ft^3)/30 lb BOD_5/day
= 295,967 ft^3

Proposed basins ratio = 2 basins, 22 ft. deep SWD, w/ 2length:1width

= 2 x 22 ft. deep x 58 ft. wide x 116 ft. long
= 296,032 ft^3

Air Requirements = (8,879 lbs. BOD_5/day)(3200 SCF/lbs. BOD_5)
= 28,412,800 SCF/day
= 19,731 cfm

150% Design Req. = (19,731 cfm)(1.5)
= 29,597 cfs

Improvements: **Construct a dual activated sludge deep tank aeration basin (each 22 ft. deep x 58 ft. wide x 116 ft. long) and provide 29,597 cfm aeration capacity.**

D. Final Clarifiers.

Required: *Maximum surface loading at Peak flow of 1200 gal./day/ft², and at Design flow of 600 gal./day/ft². Side water depth must be at least 10 ft. for surface areas of 1250 ft² or more. Effective detention times (based on liquid volume above a 3 ft. sludge blanket) must be 1.5 hr. @ Peak flow and 3.0 hr. @ Design flow.*

Analysis: Required area based on surface area:
@ Peak flow = 24,670,000 gpd / 1200 gal./day/ft² = 20,558 ft²
@ Design flow = 5,320,000 gpd / 600 gal./day/ft² = 8,867 ft²

Required area based on detention time:
Detention time is based on side water depth of 14 ft. less 3 ft. sludge blanket

$$\text{@ Peak Flow} = \frac{(24{,}670{,}000 \text{ gpd})(1.5 \text{ hrs.})}{(24 \text{ hrs/day})(7.48 \text{ gal/ft}^3)(11 \text{ ft.})} = 18{,}739 \text{ ft}^2$$

$$\text{@ Design Flow} = \frac{(5{,}320{,}000 \text{ gpd})(3.0 \text{ hrs.})}{(24 \text{ hrs/day})(7.48 \text{ gal/ft}^3)(11 \text{ ft.})} = 8{,}082 \text{ ft}^2$$

Required area based on Peak flow surface area requirements govern.

Two (2) - 116 ft. diameter clarifiers = 20,558 ft² effective surface area. 14 ft. side water depth.

Improvements: **Construct two (2) 116 ft. diameter final clarifiers with 14 ft. side water depths. Provide flow splitting/collection structures/piping, and sludge collection/pumping.**

E. Effluent Works.

1. Chlorine Contact Chamber.

Required: *Detention time of 20 minutes @ peak flow.*

Analysis: (24,670,000 gpd/1440 min/day)(20 min)/(7.48 gal/ft³) = 45,807 ft³

Basin Dimensions: 1 Basin

Width	= 30 ft./basin
Length	= 109 ft.

Depth = 14.0 ft. SWD

Improvements: **Construct a chlorine contact chamber (30 ft. wide x 109 ft. long by 14 ft. SWD).**

2. Chlorine Feed Equipment.

Required: *Feed equipment must be able to provide more than the highest dosage to be required at any time. Dosage must be adequate to maintain a chlorine residual of at least 1 mg/l after 20 minutes detention, prior to dechlorination.*

Improvements: **Provide chlorine feed equipment as necessary to provide for chlorination of 24.67 mgd.**

3. Dechlorination.

Required: *The effluent, after chlorination and 20 minutes detention time, must be dechlorinated to less than 0.1 mg/l. For most dechlorination agents, 1 minute detention is generally considered adequate.*

Analysis: (24,670,000 gpd/1440 min/day)(1 min)/(7.48 gal/ft^3) = 2290 ft^3

Basin Dimensions: 1 Basin

Length = 30 ft.
Width = 6.5 ft.
Depth = 12.0 ft. SWD

Improvements: **Construct a dechlorination chamber (30 ft. long x 6.5 ft. wide x 12 ft. SWD) and provide chemical feed equipment as necessary to provide for dechlorination of 24.67 mgd.**

4. Flow Measurement.

Required: *Continuous effluent measurement required, with capacity for maximum expected peak flow.*

Improvements: **Construct a parshall flume capable of measuring flows up to 30 mgd.**

5. Postaeration.

Improvements: **Construct a passive, cascade type aeration structure on the discharge capable of producing a 5 mg/l dissolved**

oxygen effluent.

6. Effluent Lift Station and Effluent Force Main

Improvements: **Construct effluent lift station and force main capable of discharging to the Sabine Neches Canal ADF and force main of 5.32 mgd.**

F. Sludge Processing.

1. Sludge Thickener.

Required: *Digesters should be provided with sludge thickening.*

Improvements: **Sludge thickening will be provided by decanting inside the digestors.**

2. Aerobic Digesters.

Requirement: *Minimum solids retention time of 15 days (may be calculated as 20 ft^3 for each lb. influent BOD_5 per day). Diffused air requirement is 30 cfm per 1000 ft^3 of volume.*

Analysis: Digester volume required = 20 ft^3 / lb. influent BOD_5 per day

lbs. BOD_5/day = (5.32 mgd)(8.345)(200 mg/l)
= 8,879 lbs. BOD_5/day

Required Digester Volume = (20)(8,879) = 177,580 ft^3

Proposed Digesters (2) = 22 ft. SWD
= 177,580 ft^3 / (22 ft. x 2)
= 4,036 ft^2
= 64 ft. x 64 ft.

Required aeration = (30 cfm/1000 ft^3)(177,580 ft^3) = 5,327 cfm

Improvements: **Construct two (2) deep tank type aerobic digesters (64 ft. x 64 ft. x 22 ft. SWD) and provide 5,327 cfm aeration equipment capacity.**

3. Sludge Dewatering Facility.

Required: *Sludge shall be dewatered sufficiently to meet the requirements of the ultimate form of disposal.*

Improvements: **Provide sludge dewatering facilities.**

G. Blowers.

Required: *Blowers must be able to meet maximum aeration requirements with largest unit out of service.*

Analysis:	Aerated Grit Chamber	= 294 cfm
	Activated Sludge Aeration	= 19,731 cfm
	Aerobic Digestion	= 5,327 cfm
		= 25,352 cfm

Improvements: **Provide blowers as required for activated sludge aeration and aerobic digestion.**

H. Opinion of Probable Cost

1.	Upgrade Taft Street Lift Station	$	802,000
2.	Transfer lift station/force main - North Plant Flows	$	1,970,000
3.	Influent headworks, screens	$	146,000
4.	Grit Chamber	$	198,000
5.	Activated sludge basin(s)	$	1,395,000
6.	Final Clarifiers	$	1,521,000
7.	Chlorination/dechlorination/feed equipment	$	378,000
8.	Effluent flow measurement/post-aeration	$	40,000
9.	Outfall	$	187,000
10.	Aerobic digester	$	745,000
11.	Sludge dewatering facilities	$	495,000
12.	Aeration blower equipment	$	610,000
13.	Yard Piping	$	925,000
14.	Site work	$	825,000
15.	Electrical and Instrumentation	$	1,010,000
16.	Laboratory/Office	$	100,000
17.	Site Acquisition	$	250,000
	Subtotal	$	11,597,000
	Contingency (15%)	$	1,740,000
	TOTAL OPINION OF PROBABLE CONSTRUCTION COST	**$**	**13,337,000**

G7 - City of Groves: Abandon Existing North and South WWTF's, Construct New Trickling Filter Treatment Facility at 32nd Street and SH 366 discharge to the Sabine Neches Canal

Abandon existing WWTF, construct a new trickling filter treatment facility for full treatment of all flows and then discharge into the Sabine Neches Canal at the following effluent limits.

ADF	=	5.32 mgd
2-Hour Peak	=	17,132 gpm (24.67 mgd)
BOD_5	=	20 mg/l
TSS	=	20 mg/l
NH_3	=	no limit
D.O.	=	5 mg/l

A. Preliminary Treatment

1. Screening.

Required: *Some form of screening; bar openings minimum ½" for mechanical screens; velocities @ design flow minimum 2ft./sec through channel, < 3 ft./sec. through screen.*

Analysis: Design Flow = 5.32 mgd = 8.23 cfs

Improvements: Construct a dual channel influent structure with one mechanical bar screen in one channel, sized for a design flow of 5.32 mgd and a peak flow of 24.67 mgd, and a fixed bar screen in the second channel.

2. Influent Lift Station. Construct an influent lift station to lift the raw wastewater flows into the proposed treatment facility.

Required: *Firm pumping capacity (largest pump out of service) must be adequate to pump peak flow to destination; three or more pumps (or duplex pumps with automatic variable speed control) required.*

Analysis: Firm capacity = 17,132 gpm

B. Primary Clarifier.

Required: Maximum surface loading at Peak flow of 1800 gal./day/ft^2, and at Design flow of 1000 gal./day/ft^2. Side water depth must be at least 7 ft.

Analysis: Required area:
@ Peak flow = 24,670,000 gpd / 1800 gal./day/ft^2 = 13,706 ft^2

@ Design flow = 5,320,000 gpd / 1000 gal./day/ft^2 = 5,320 ft^2

Improvements: **Construct three (3) 78 ft. diameter primary clarifiers with 14 ft. side water depth. Provide sludge collection/pumping.**

C. Trickling Filter.

Required: *Application of synthetic media shall be evaluated on a case-by-case basis. The design engineer shall submit sufficient operating data from existing trickling filters of similar construction and operation to justify the efficiency calculations for the filters. Filter efficiency formula from a reliable source acceptable to the commission may be used.*

Analysis: Required BOD_5 reduction = [(65%)(200 mg/l)-(20 mg/l)] / [(65%)(200 mg/l)]

= 84.6% BOD_5 reduction

From Munter's BioDeck® 19060 literature for 85% BOD_5 reduction, use a loading of 63 lbs BOD_5/1,000 ft^3/day.

Volume Required

= (5.32 mgd)(8.345)(200 mg/l BOD_5)(65%)/(63 lb BOD_5/1000 ft^3/day

= 91,610 ft^3 media

Improvements: **Construct two (2) new 100 ft. diameter x 6 ft. deep trickling filter with synthetic media. Provide for 2 x ADF recirculation.**

D. Final Clarifier.

Required: *Maximum surface loading at Peak flow of 1600 gal./day/ft^2, and at Design flow of 800 gal./day/ft^2. Side water depth must be at least 10 ft. for surface areas of 1250 ft^2 or more. Effective detention times (based on liquid volume above a 3 ft. sludge blanket) must be 1.1 hr. @ Peak flow and 2.2 hr. @ Design flow.*

Analysis: Required area based on surface area:

@ Peak flow = 24,670,000 gpd / 1600 gal./day/ft^2 = 15,419 ft^2

@ Design flow = 5,320,000 gpd / 800 gal./day/ft^2 = 6,650 ft^2

Required area based on detention time:

Detention time is based on side water depth of 14 ft. less 3 ft. sludge blanket

@ Peak flow = (24,670,000 gpd) (1.1 hrs.) / [(24 hrs/day)(7.48 gal/ft^3)(11ft.)] = 13,742 ft^2

@ Design flow = (5,320,000) gpd) (2.2 hrs)/[(24 hrs/day) (7.48 gal/ft^3) (11ft.)] = 5,927 ft^2

Required area based on Peak flow detention requirements govern.

Improvements: **Construct three (3) 82 ft. diameter final clarifiers with 14 ft. side water depth. Provide sludge collection/pumping.**

E. Effluent Works.

1. Chlorine Contact Chamber.

Required: *Detention time of 20 minutes @ peak flow.*

Analysis: (24,670,000 gpd/1440 min/day) (20 min) / (7.48 gal/ft^3) = 45,807 ft^3

Improvements: **Construct a 45,807 ft^3 chlorine contact chamber.**

2. Chlorine Feed Equipment.

Required: *Feed equipment must be able to provide more than the highest dosage to be required at any time. Dosage must be adequate to maintain a chlorine residual of at least 1 mg/l after 20 minutes detention, prior to dechlorination.*

Improvements: **Provide chlorine feed equipment as necessary to provide for chlorination of 24.67 mgd.**

3. Dechlorination.

Required: *The effluent, after chlorination and 20 minutes detention time, must be dechlorinated to less than 0.1 mg/l. For most dechlorination agents, 1 minute detention is generally considered adequate.*

Analysis: 24,670,000 gpd/1440 min/day) (1 min) / (7.48 gal/ft^3) = 2,290 ft^3

Improvements: **Construct a 2,290 ft^3 dechlorination chamber and provide sodium bisulfate feed equipment as necessary to provide for dechlorination 24.67 mgd.**

4. Flow Measurement.

Required: *Continuous effluent measurement required, with capacity for maximum expected peak flow.*

Improvements: **Construct a parshall flume capable of measuring flows up to 30 mgd.**

5. Postaeration.

Improvements: **Construct a passive, cascade type aeration structure on the discharge capable of producing a 5 mg/l dissolved oxygen effluent.**

6. Effluent Lift Station.

Improvements: **Construct effluent lift station and force main capable of discharging to the Sabine Neches Canal ADF and force main of 5.32 mgd.**

F. Sludge Processing.

1. Anaerobic Digester.

Required: *For sludge from primary clarifiers plus sludge from clarifiers following trickling filters, 19.0 ft^3 for each lb. influent BOD_5* per day for a heated digester.

Analysis: Influent BOD_5 = (5.32 mgd)(8.345) (200 mg/l) = 8879 lb. BOD_5/day

Required Digester Volume = (19.0)(8879) = 168,700 ft^3

Proposed Digester = 24 ft. SWD
= 168,700 ft^3 / 24 ft.

Improvements: **Construct an anaerobic digester (two (2) basins 67 ft. dia. x 24 ft. SWD)**

2. Sludge Dewatering Facility.

Required: *Sludge shall be dewatered sufficiently to meet the requirements of the ultimate form of disposal.*

Improvements: **Provide a means of sludge dewatering.**

G. Opinion of Probable Costs

1.	Transfer lift stations/force mains		
	a. Taft Street Lift Station & F.M.	$	863,000
	b. North Plant Lift Station & F.M.	$	1,526,000
2.	Influent headworks, screens	$	146,000
3.	Primary Clarifiers(s)	$	1,194,000
4.	Trickling Filters(s)	$	897,150
5.	Recirculation Pumps	$	70,000
6.	Final Clarifiers	$	1,414,200
7.	Chlorination/dechlorination/feed equipment	$	498,000
8.	Effluent flow measurement/post-aeration	$	40,000
9.	Effluent lift station & F.M.	$	1,244,000
10.	Aerobic digester	$	745,000
11.	Blowers for Digester	$	225,000
12.	Sludge dewatering facilities	$	495,000
13.	Yard Piping	$	925,000
14.	Site Work	$	825,000
15.	Electrical and Instrumentation	$	1,010,000
16.	Laboratory/Office	$	100,000
17.	Site Acquisition	$	250,000
	Subtotal	$	12,467,350
	Contingency (15%)	$	1,869,650
	TOTAL OPINION OF PROBABLE CONSTRUCTION COST	$	14,337,000

G-8 City of Groves: **ADF Only**

In analysing Alternates PN-1 and PN/G-1 it was assumed that the TNRCC would require that all flows be diverted to the Neches River. During wet weather the existing receiving stream may have adequate flow to receive a discharge at less stringent quality standards. Therefore, if the TNRCC would allow all flows up to the permitted ADF from each treatment facility to be diverted to the Neches River and allow peak flow in excess of the permitted ADF to continue to be discharged to the existing receiving stream at current 20/20 limits, then the cost of this alternative could be significantly reduced.

Analysis: *Firm capacity = 1.99 mgd (1,382 gpm)*

Three (3) pumps total; firm capacity of 1,382 gpm with largest pump out of service (i.e. 2 pumps pumping + 1 spare).

Effluent force main(s) sized for a minimum of 2 fps at low flow and a maximum 10 fps at peak flow.

Proposed force main = 12" diameter

Improvements: ***Construct an effluent lift station with three (3) pumps (firm capacity of 3,458 gpm) and a 12" diameter force main to the Neches River.***

Opinion of Probable Cost

1.	*Construct effluent lift station*	*$*	*100,000*
2.	*Yard piping improvements*	*$*	*25,000*
3.	*Electrical and instrumentation*	*$*	*55,000*
4.	*Construct outfall force main(s)*	*$*	*863,500*
	Subtotal	*$*	*1,043,500*
	Contingency (15%)	*$*	*156,500*
	TOTAL OPINION OF PROBABLE CONSTRUCTION COST	***$***	***1,200,000***

Refer to Appendix C for
Discussion Regarding Design
and Selection of Alternatives

G1 - City of Groves
Single New Trickling Filter Treatment Facilities to Serve Entire City
(Located at the North WWTF Site)

ANNUAL OPERATING COSTS

Assumed energy cost $0.0900 /kW-hr

Unit/Equipment Description	Total hp	Estimated Run Time %	Power kW-hr/day	Power Cost $/yr.	Misc. Expenses $/yr.
1. Preliminary Treatment					
a. Mechanical bar screen	7.5	16.67%	22.37	$734.89	
b. Lift Station (Influent & Offsite)					
4 - 4400 gpm pumps (200 hp ea.)	800.0	33.20%	4,753.97	$156,167.79	
4 - 1725 gpm pumps (30 hp ea.)	120.0	32.87%	705.96	$23,190.91	
2. Primary Clarifier					
a. 3 @ 5 hp	15.0	100.00%	268.45	$8,818.65	
b. Waste Sludge Pumps (4 - 10 hp)	40.0	25.00%	178.97	$5,879.10	
3. Trickling Filter					
2 - Distributor Drives (5 hp ea.)	10.0	100.00%	178.97	$5,879.10	
Recirculation pumps (4 - 15 hp)	60.0	25.00%	268.45	$8,818.65	
4. Final Clarifiers					
a. 2 @ 5 hp ea.	10.0	100.00%	178.97	$5,879.10	
b. Return Sludge Pumps (4 - 30 hp ea.)	120.0	44.75%	961.06	$31,570.94	
5. Effluent Works					
a. Chlorination					$86,200.00
b. Dechlorination - SO2					$12,500.00
c. Postaeration					N/A
6. Effluent Lift Station					
4-5800 gpm pumps (200 hp ea.)	800.0	25.63%	3,668.84	$120,521.53	
7. Sludge Processing					
a. Sludge Thickening					N/A
b. Anaerobic Digester					
3 - 100 hp Blowers	300.0	66.67%	3,579.36	$117,581.98	
c. Sludge Dewatering					
i. Polymer					$34,400.00
ii. 2 - 1.5m Belt Presses (7.5 hp ea.)	15.0	8.93%	23.97	$787.38	
iii. Sludge Metering Pumps (2 @ 15 hp)	30.0	8.93%	47.94	$1,574.76	
iv. Sludge Conveyor (1 @ 5 hp)	5.0	8.93%	7.99	$262.46	
v. Sludge Hauling					$37,500.00
8. Misc. Power & Lighting					$5,000.00
9. Equipment Replacement					$25,000.00

Subtotal	$307,573.63	$200,600.00
Total O&M Cost Per Year		**$508,173.63**
Total O&M Costs Per Month		**$42,347.80**
Total O&M Costs (30 Years)		**$15,245,208.86**
Total Opinion of Capital Costs		**$15,258,000.00**
Total Costs (30 yrs.)		**$30,503,208.86**

G4 - City of Groves South Plant
New Trickling Filter Facility Using Existing Filter Structures

ANNUAL OPERATING COSTS

Assumed energy cost $0.0900 /kW-hr

Unit/Equipment Description	Total hp	Estimated Run Time %	Power kW-hr/day	Power Cost $/yr.	Misc. Expenses $/yr.
1. Preliminary Treatment					
a. Mechanical bar screen	7.5	16.67%	22.37	$734.89	
b. Lift Station (Influent & Offsite)					
Existing 5850 gpm firm	300.0	33.91%	1,820.83	$59,814.39	
2-3750 gpm pumps (175 bhp)	525.0	11.11%	1,043.98	$34,294.74	
2. Primary Clarifier					
a. 1 @ 5 hp	5.0	100.00%	89.48	$2,939.55	
b. Waste Sludge Pumps (2 - 20 hp)	40.0	25.00%	178.97	$5,879.10	
3. Trickling Filter					
3 - Distributor Drives (5 hp ea.)	15.0	100.00%	268.45	$8,818.65	
Recirculation pumps (3 - 15 hp)	60.0	25.00%	268.45	$8,818.65	
4. Final Clarifiers					
a. 1 @ 5 hp ea.	5.0	100.00%	89.48	$2,939.55	
b. Sludge Pumps (4 - 30 hp ea.)	45.0	44.75%	360.40	$11,839.10	
5. Effluent Works					
a. Chlorination					$54,989.66
b. Dechlorination - SO2					$7,824.25
c. Postaeration					N/A
6. Sludge Processing					
a. Sludge Thickening					N/A
b. Anaerobic Digester					
3 - 100 hp Blowers	300.0	66.67%	3,579.36	$117,581.98	
c. Sludge Dewatering					
i. Polymer					$21,532.33
ii. 2 - 1.5m Belt Presses (7.5 hp ea.)	15.0	8.93%	23.97	$787.38	
iii. Sludge Metering Pumps (2 @ 15 hp)	30.0	8.93%	47.94	$1,574.76	
iv. Sludge Conveyor (1 @ 5 hp)	5.0	8.93%	7.99	$262.46	
v. Sludge Hauling					$23,472.74
7. Misc. Power & Lighting					$5,000.00
8. Equipment Replacement					$25,000.00

Subtotal	**$161,441.17**	**$137,818.98**
Total O&M Cost Per Year		**$299,260.15**
Total O&M Costs Per Month		**$24,938.35**
Total O&M Costs (30 Years)		**$8,977,804.43**
Total Opinion of Capital Costs		**$8,448,000.00**
Total Costs (30 yrs.)		**$17,425,804.43**

G5 - City of Groves
Single New Activated Sludge Treatment Facilities to Serve Entire City
(Located at North Plant Site)

ANNUAL OPERATING COSTS

Assumed energy cost $0.0900 /kW-hr

Unit/Equipment Description	Total hp	Estimated Run Time %	Power kW-hr/da	Power Cost $/yr.	Misc. Expenses $/yr.
1. Preliminary Treatment					
a. Mechanical bar screen	7.5	16.67%	22.37	$734.89	
b. Lift Station (Influent & Offsite)					
4 - 4400 gpm pumps (200 hp ea.)	800.0	33.20%	4,753.97	$156,167.79	
4 - 1725 gpm pumps (30 hp ea.)	120.0	32.87%	705.96	$23,190.91	
c. Aerated Grit Chamber (except aeration)	5.0	16.67%	14.91	$489.92	
d. Grit pump	10.0	16.67%	29.83	$979.85	
e. Degritter	15.0	16.67%	44.74	$1,469.77	
2. Activated Sludge Process					
5 - 175 hp blowers (20,025 scfm)	875.0	80.00%	12,527.76	$411,536.92	
3. Final Clarifiers					
a. 2 @ 5 hp ea.	10.0	100.00%	178.97	$5,879.10	
b. RAS Pumps (4 - 30 hp ea.)	120.0	44.75%	961.06	$31,570.94	
c. WAS Pumps (2 - 10 hp)	20.0	8.33%	29.83	$979.85	
4. Effluent Works					
a. Chlorination					$86,200.00
b. Dechlorination - SO2					$12,500.00
c. Postaeration					N/A
5. Effluent Lift Station					
4-5800 gpm pumps (200 hp ea.)	800.0	25.63%	3,668.84	$120,521.53	
6. Sludge Processing					
a. Sludge Thickening					N/A
b. Aerobic Digester					
3 - 100 hp Blowers (5,327 scfm)	300.0	66.67%	3,579.36	$117,581.98	
c. Sludge Dewatering					
i. Polymer					$34,400.00
ii. 2 - 1.5m Belt Presses (7.5 hp ea.)	15.0	8.93%	23.97	$787.38	
iii. Sludge Metering Pumps (2 @ 15 hp)	30.0	8.93%	47.94	$1,574.76	
iv. Sludge Conveyor (1 @ 5 hp)	5.0	8.93%	7.99	$262.46	
v. Sludge Hauling					$37,500.00
7. Misc. Power & Lighting					$5,000.00
8. Equipment Replacement					$25,000.00
			Subtotal	$693,144.53	$200,600.00

Total O&M Cost Per Year	**$893,744.53**
Total O&M Costs Per Month	**$74,478.71**
Total O&M Costs (30 Years)	**$26,812,335.75**
Total Opinion of Capital Costs	**$15,146,000.00**
Total Costs (30 yrs.)	**$41,958,335.75**

G6 - City of Groves
Single New Activated Sludge Treatment Facilities to Serve Entire City
Located at South Plant Site

ANNUAL OPERATING COSTS

Assumed energy cost $0.0900 /kW-hr

Unit/Equipment Description	Total hp	Estimated Run Time %	Power kW-hr/day	Power Cost $/yr.	Misc. Expenses $/yr.
1. Preliminary Treatment					
a. Mechanical bar screen	7.5	16.67%	22.37	$734.89	
b. Lift Station (Offsite)					
4 - 4400 gpm pumps (200 hp ea.)	800.0	33.20%	4,753.97	$156,167.79	
3 - 3100 gpm pumps (150 hp ea.)	450.0	29.25%	2,355.56	$77,380.14	
c. Aerated Grit Chamber (except aeration)	5.0	16.67%	14.91	$489.92	
d. Grit pump	10.0	16.67%	29.83	$979.85	
e. Degritter	5.0	16.67%	14.91	$489.92	
2. Activated Sludge Process					
5 - 175 hp blowers (20,025 scfm)	875.0	80.00%	12,527.76	$411,536.92	
3. Final Clarifiers					
a. 2 @ 5 hp ea.	10.0	100.00%	178.97	$5,879.10	
b. RAS Pumps (4 - 30 hp ea.)	120.0	44.75%	961.06	$31,570.94	
c. WAS Pumps (2 - 10 hp)	20.0	8.33%	29.83	$979.85	
4. Effluent Works					
a. Chlorination					$86,200.00
b. Dechlorination - SO2					$12,500.00
c. Postaeration					N/A
5. Sludge Processing					
a. Sludge Thickening					N/A
b. Aerobic Digester					
3 - 100 hp Blowers (5,327 scfm)	300.0	66.67%	3,579.36	$117,581.98	
c. Sludge Dewatering					
i. Polymer					$34,400.00
ii. 2 - 1.5m Belt Presses (7.5 hp ea.)	15.0	8.93%	23.97	$787.38	
iii. Sludge Metering Pumps (2 @ 15 hp)	30.0	8.93%	47.94	$1,574.76	
iv. Sludge Conveyor (1 @ 5 hp)	5.0	8.93%	7.99	$262.46	
v. Sludge Hauling					$37,500.00
6. Misc. Power & Lighting					$5,000.00
7. Equipment Replacement					$25,000.00

Subtotal	**$571,643.15**	**$200,600.00**
Total O&M Cost Per Year		**$772,243.15**
Total O&M Costs Per Month		**$64,353.60**
Total O&M Costs (30 Years)		**$23,167,294.50**
Total Opinion of Capital Costs		**$13,337,000.00**
Total Costs (30 yrs.)		**$36,504,294.50**

G7 - City of Groves - Divert ADF Flows Only
Single New Trickling Filter Treatment Facilities to Serve Entire City
(Located at the New 32nd & 366 Site Near Fina)

ANNUAL OPERATING COSTS

Assumed energy cost $0.0900 /kW-hr

Unit/Equipment Description	Total hp	Estimated Run Time %	Power kW-hr/day	Power Cost $/yr.	Misc. Expenses $/yr.
1. Preliminary Treatment					
a. Mechanical bar screen	7.5	16.67%	22.37	$734.89	
b. Lift Station (Offsite)					
4 - 4400 gpm pumps (100 hp ea.)	400.0	23.26%	1,665.28	$54,704.57	
3 - 2100 gpm pumps (125 hp ea.)	375.0	29.25%	1,962.97	$64,483.45	
2. Primary Clarifier					
a. 2 @ 5 hp	10.0	100.00%	178.97	$5,879.10	
b. Waste Sludge Pumps (4 - 10 hp)	40.0	25.00%	178.97	$5,879.10	
3. Trickling Filter					
2 - Distributor Drives (5 hp ea.)	10.0	100.00%	178.97	$5,879.10	
Recirculation pumps (4 - 15 hp)	60.0	25.00%	268.45	$8,818.65	
4. Final Clarifiers					
a. 2 @ 5 hp ea.	10.0	100.00%	178.97	$5,879.10	
b. Return Sludge Pumps (4 - 30 hp ea.)	120.0	44.75%	961.06	$31,570.94	
5. Effluent Works					
a. Chlorination					$86,200.00
b. Dechlorination - SO2					$12,500.00
c. Postaeration					N/A
6. Effluent Lift Station					
2 - 3700 gpm pumps (150 hp ea.)	300.0	49.92%	2,680.17	$88,043.48	
3 - 7400 gpm pumps (200 hp ea.)	600.0	16.67%	1,789.68	$58,790.99	
7. Sludge Processing					
a. Sludge Thickening					N/A
b. Anaerobic Digester					
3 - 100 hp Blowers	300.0	66.67%	3,579.36	$117,581.98	
c. Sludge Dewatering					
i. Polymer					$34,400.00
ii. 2 - 1.5m Belt Presses (7.5 hp ea.)	15.0	8.93%	23.97	$787.38	
iii. Sludge Metering Pumps (2 @ 15 hp)	30.0	8.93%	47.94	$1,574.76	
iv. Sludge Conveyor (1 @ 5 hp)	5.0	8.93%	7.99	$262.46	
v. Sludge Hauling					$37,500.00
8. Misc. Power & Lighting					$5,000.00
9. Equipment Replacement					$25,000.00
			Subtotal	$330,947.02	$200,600.00
		Total O&M Cost Per Year			$531,547.02
		Total O&M Costs Per Month			$44,295.58
		Total O&M Costs (30 Years)			$15,946,410.57
		Total Opinion of Capital Costs			$14,337,000.00
		Total Costs (30 yrs.)			$30,283,410.57