GATLIN, WOODS & CARLSON
Attorneys at Law
a partnership including a professional association

The Mahan Station
1709-D Mahan Drive
Tallahassee, Florida 32308

B. KENNETH GATLIN, P.A.
THOMAS F. WOODS
JOHN D. CARLSON
WAYNE L. SCHIEFELBEIN

TELEPHONE (904) 877-7191
TELECOPIER (904) 877-9031

May 3, 1996

ORIGINAL FILE COPY

HAND DELIVERY

Ms. Blanca S. Bayo, Director
Division of Records and Reporting
Florida Public Service Commission
2540 Shumard Oak Blvd.
Tallahassee, Florida 32399-0850

RE: Docket No. 950387-SU
Application of Florida Cities Water Company, North Ft. Myers Division, for an Increase in Wastewater Rates in Lee County, Florida

Dear Ms. Bayo:

Enclosed for filing are an original and fifteen copies of our Certificate of Service and Late-Filed Hearing Exhibits Nos. 14 and 27, in reference to the above docket.

Please acknowledge receipt of foregoing by stamping the enclosed extra copy of this letter and returning same to my attention.

Very truly yours,

B. Kenneth Gatlin

B. Kenneth Gatlin

BKG/met
Enclosures

ACK ✓
AFA 2
APP
CAF
CMU
CTR
EAG
LEG Jaeger
LIN org to Linda; 3
OPC
RCH
SEC 1
WAS
OTH

RECEIVED & FILED
FPSC-BUREAU OF RECORDS

Exhibit 14
DOCUMENT NUMBER-DATE
05043 MAY-3 96
FPSC-RECORDS/REPORTING

Exhibit 27
DOCUMENT NUMBER-DATE
05044 MAY-3 96
FPSC-RECORDS/REPORTING

BEFORE THE FLORIDA PUBLIC SERVICE COMMISSION

Re: Application of Florida Cities Water) Company, North Ft. Myers Division,) for an increase in wastewater rates in) Lee County, Florida)	Docket No. 950387-SU Filed: May 3, 1996

CERTIFICATE OF SERVICE

I HERBY CERTIFY that a true and correct copy of Late-Filed Hearing Exhibits Nos. 14 and 27 have been furnished by hand delivery to Mr. Ralph Jaeger, Esquire, Division of Legal Services, Florida Public Service Commission, 2540 Shumard Oak Boulevard, Tallahassee, Florida 32399-0850, and to Harold McLean, Esquire, Office of Public Counsel, 111 W. Madison Street, Room 812, Claude Pepper Building, Tallahassee, Florida 32399-1400, and by regular U.S. Mail on this 3rd day of May, 1996 to:

Harry Bowne
4274 Harbour Lane
N. Ft. Myers, FL 33903

Eugene W. Brown
2069 W. Lakeview Boulevard
N. Ft. Myers, FL 33903

Fay A. Schweim
4640 Vinsetta Avenue
N. Ft. Myers, FL 33903

Eugene F. Pettenelli
4300 Glasgow Court
N. Ft. Myers, FL 33903

Jerilyn L. Victor
1740 Dockway Drive
N. Ft. Myers, FL 33903

Beverly and Robert Hemenway
4325 S. Atlantic Circle
N. Ft. Myers, FL 33903

Nancy L. McCullough
683 Camellia Drive
N. Ft. Myers, FL 33903

C. Belle Morrow
691 Camellia Drive
N. Ft. Myers, FL 33903

Dawn E. Coward
951 Tropical Palm Avenue
N. Ft. Myers, FL 33903

Kevin A. Morrow
905 Poinsettia Drive
N. Ft. Myers, FL 33903

Doris T. Hadley
1740 Dockway Drive
N. Ft. Myers, FL 33903

Cheryl Walla
1750 Dockway Drive
N. Ft. Myers, FL 33903

Respectfully submitted

B. Kenneth Gatlin

B. Kenneth Gatlin
Fla. Bar #0027966
Gatlin, Woods, Carlson & Cowdery
1709-D Mahan Drive
Tallahassee, Florida 32308
(904) 877-7191

Attorneys for
FLORIDA CITIES WATER COMPANY

Florida Cities Water Company
North Fort Myers Division
Wastewater Operations
Docket No. 950387-SU
Late Filed Exhibit 27

Attached is a letter by Tom Cummings of Black & Veatch dated May 1, 1996 addressed to Mr. Young, Florida Cities Water Company, which provides the peaking factors utilized in the design of the 1.25 MGD Waterway Estates Advanced Wastewater Treatment Plant. This plant was completed March 15, 1996 (Exhibit 24).

Attached to Mr. Cummings' letter is a copy of the relevant portion of the 1992 preliminary engineering report prepared by Black & Veatch.

Also enclosed is a copy of the engineering study for the 1.0 MGD plant prepared by Source, Inc. In May, 1989 and to which Mr. Cummings refers in his letter.

BLACK & VEATCH

201 South Orange Avenue, Suite 500, Orlando, Florida 32801, (407) 419-3500, Fax (407) 419-3501

Florida Cities Water Company

B&V Project 19440.800
B&V File B
May 1, 1996

Mr. Doug Young
Florida Cities Water Company
4837 Swift Road, Suite 100
Sarasota, FL 34231

Subject: Florida Cities Water Company
North Fort Myers Division
Wastewater Operations
Docket No. 950387-SU

Dear Mr. Young:

Commissioner Garcia at the hearing in this proceeding requested the peaking factors used for the biological and hydraulic design flows of the Waterway Estates Advanced Wastewater Treatment Plant (WWEAWWTP).

Attached are copies of pages from the Waterway Estates Preliminary Design Reports for both the previous and present design of WWEAWWTP which provide this information.

The original 1.0 MGD advanced wastewater treatment plant was designed by Source, Inc. The criteria for design of the biological treatment process is shown under the influent characteristics section of the design calculations. Based on review of the report it appears that the peak biological design flow was 1.0 MGD and the peak hydraulic design flow was 3.0 MGD.

The expansion to 1.25 MGD was designed by Black & Veatch. The peak biological design flow was based on an average design flow of 1.25 MGD with an increased organic load. For carbonaceous loading, a factor of 1.5 times the maximum design load for Biochemical Oxygen Demand (BOD) (eg. 1.5 x 312 mg/L BOD = 468 mg/L BOD at peak) and Total Suspended Solids (TSS) was used. For nitrogenous loadings, a peak organic loading factor of 1.3 was used. Attached are graphs from the preliminary engineering report which illustrate the previous six years of influent characteristics. These graphs formed the basis of the maximum biological design loads.

Florida Cities Water Company
Mr. Doug Young

B&V Project 19440.800
May 1, 1996

The peak hydraulic design flow of the plant was based on a peak daily flow of two times the average daily flow after the equalization basin. This results in a peak design flow of 2.5 MGD with an average annual daily design flow of 1.25 MGD.

Very truly yours,

BLACK & VEATCH

Thomas A. Cummings

Thomas A. Cummings, P.E.
Project Manager

Enclosure

2. *Influent Concentrations*

Historical wastewater concentrations serve as the basis of design for sizing or setting the capacity of the expanded wastewater treatment facility. Process loading design criteria that were used in evaluating the unit operations and processes at the Waterway Estates WWTP are as follow:

Average Design Loading - Mean concentration based on historical data. This load is used to estimate sludge production and turndown capability for blowers and RAS pumps.

Maximum Design Loading - Estimated as the mean plus two times the standard deviation of the data. This value represents the 95th percentile of the data range and is approximately equal to the maximum monthly value. This loading is used in the modeling and sizing of the biological treatment process and sludge treatment processes.

Peak Design Loading - Computed as the maximum design loading times a peaking factor of 1.5 for carbonaceous load and 1.3 for nitrogenous load. This loading represents the peak day load to the biological system. This load is used to calculate the peak standard oxygen transfer rate (SOTR) required for the biological system. This rate is utilized in sizing blowers for the aeration system.

The average monthly influent concentrations for the Waterway Estates WWTP from January 1986 to March 1992 are summarized in Appendix A. The statistical analysis of the monthly average influent concentrations yielded the following for the mean and mean plus two standard deviations (2S):

	Mean	Mean+2S
Biochemical Oxygen Demand (BOD_5),mg/l	200	312
Total Suspended Solids (TSS), mg/l	242	379
Total Kjeldahl Nitrogen (TKN), mg/l	33.3	53.2
Total Phosphorus (as PO_4), mg/l	7.8	12.4

The mean + 2S, or maximum design concentrations will be used throughout the preliminary design. Average monthly BOD_5, TSS, TKN, and PO_4 are illustrated in Figures 2 to 5. The average and maximum design concentrations are indicated on

Figure 2

WATERWAY ESTATES WWTP
INFLUENT BOD_5

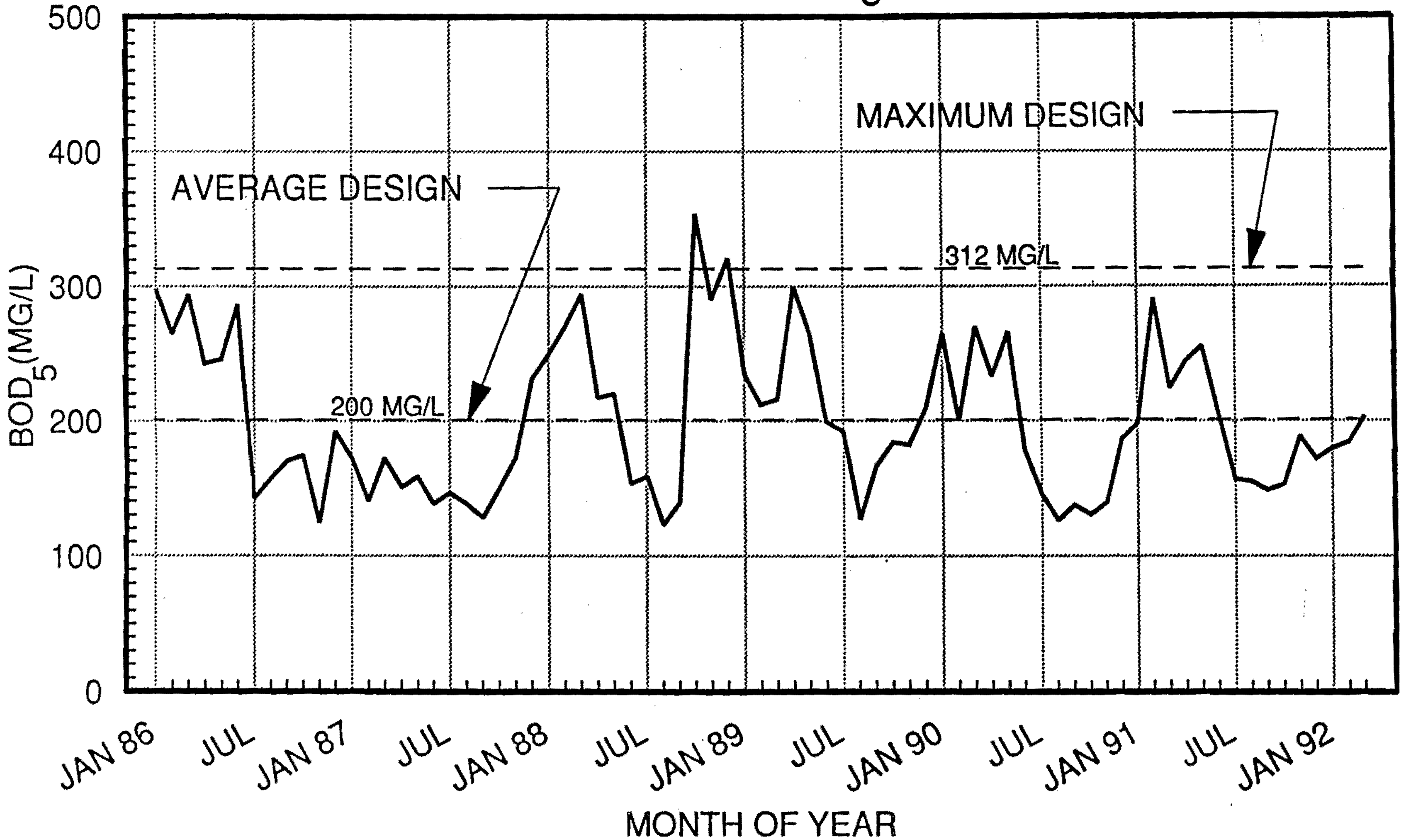


Figure 3

WATERWAY ESTATES WWTP
INFLUENT TSS

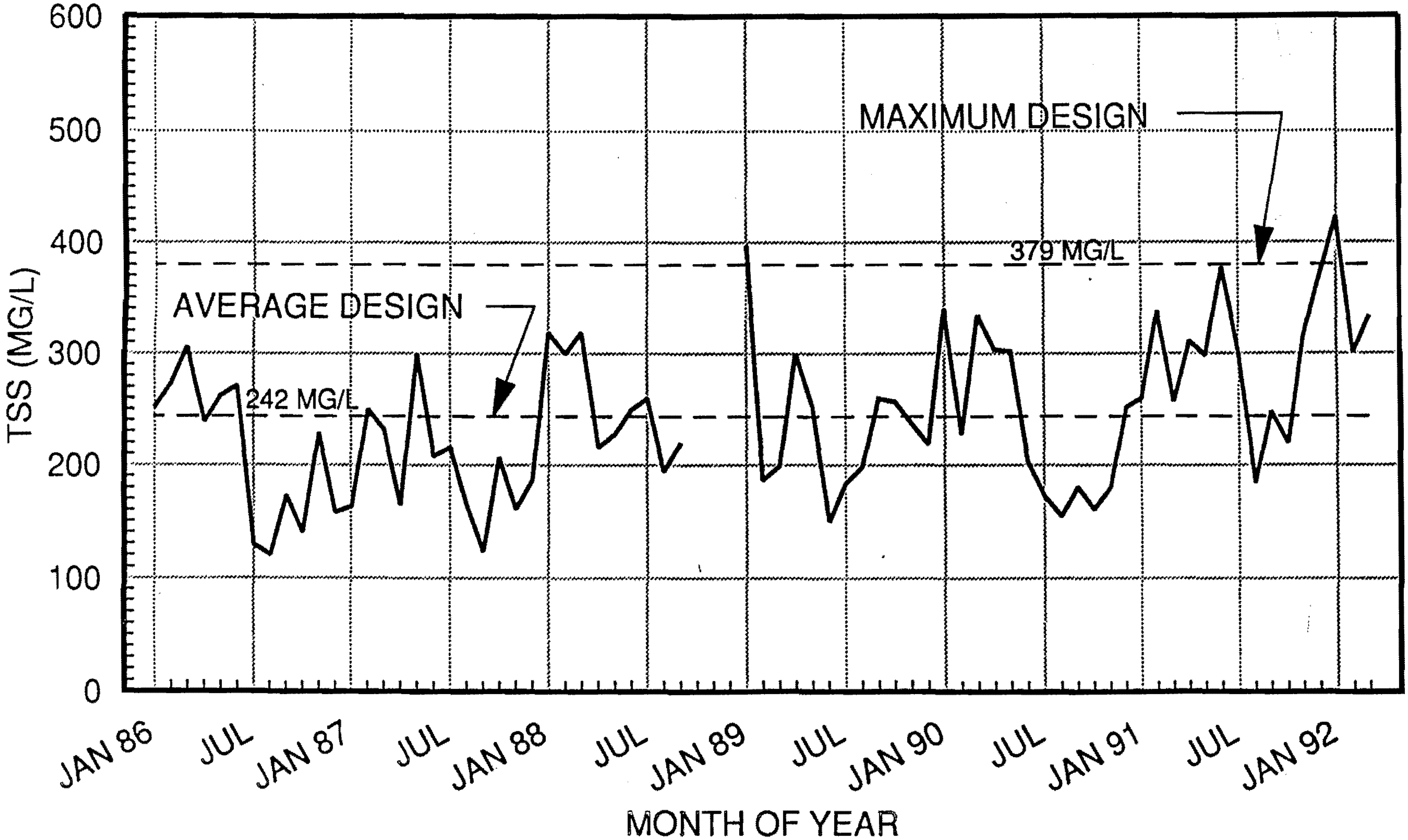


Figure 4

WATERWAY ESTATES WWTP
INFLUENT TKN

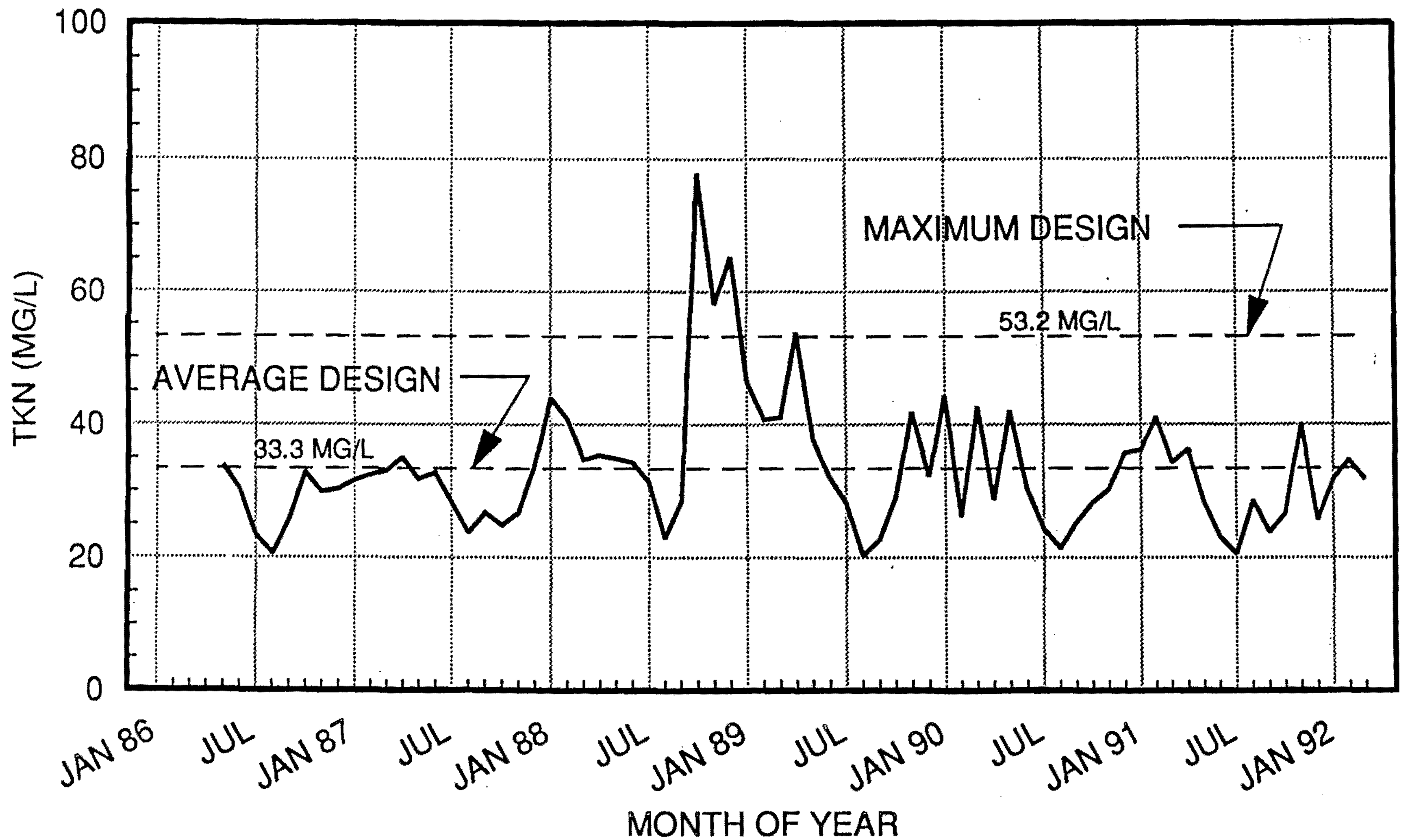
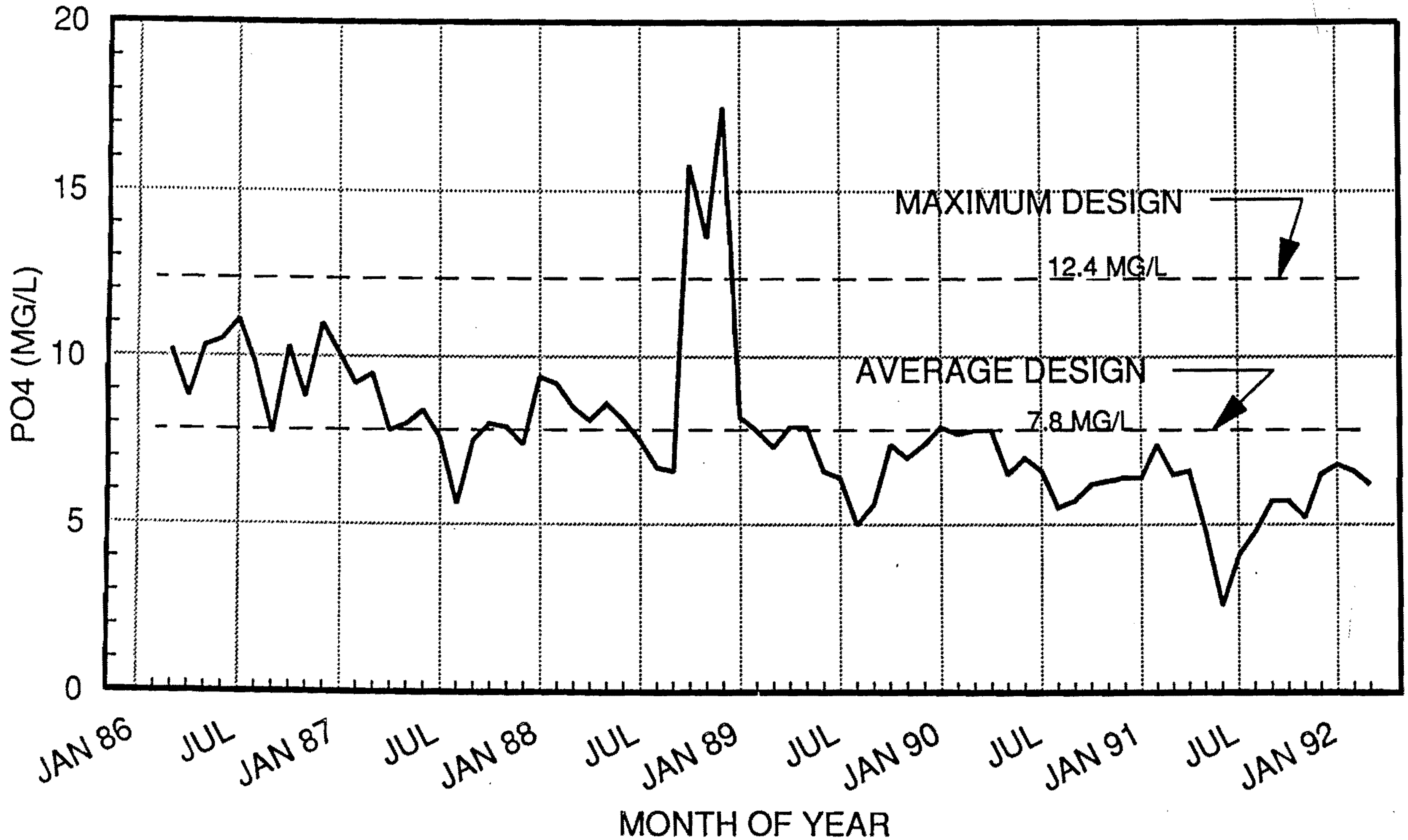


Figure 5

WATERWAY ESTATES WWTP
INFLUENT PO4



CHAPTER 3 - EXPANDED PLANT DESIGN

A. Design Approach

The goal of the preliminary design of the plant expansion was to maximize the capacity of existing facilities. All evaluations were based on achieving Class I reliability within the facility. A hydraulic analyses of the plant was performed at the Phase I peak of 2.6 mgd and at the Phase II projected peak flow of 3.0 mgd to determine necessary modifications. The biological process was modeled using the existing sequence of anoxic and aerated (oxic) tankage at the design average daily flow. A construction cost opinion was then developed for the facility modifications. Sludge handling facilities are discussed briefly in the text but not included in the construction cost opinion. The collection system was not reviewed.

B. Hydraulic Analyses

A hydraulic analysis of the existing facilities was performed at the Phase I average and peak flows of 1.3 mgd and 2.6 at the Phase II average and peak flows of 1.5 mgd and 3.0 mgd, respectively. A peaking factor of two times the average daily flow was used for peak flow to account for diurnal fluctuations in excess of the existing equalization basin capacity. A detailed design of the equalization basin was performed by others under a previous design report, and appears to have the capacity to shave daily peaks in flow. The flow split between biological treatment units (BTUs) was assumed to be 45% to BTU #1 and 55% to BTU #2, based on the relative tank volumes.

Hydraulic calculations are presented in Appendix B for both the existing plant and expanded plant design. The following formulas were used in the hydraulic calculations: friction loss in pipes -Hazen and Williams formula with a C value of 100, velocity head - Bernoulli's equation with standard "K" factors, V-notched weirs - Thompson formula assuming a 2.5" notch height and 6" spacing, and rectangular suppressed weirs - Francis Formula. **Existing facilities which must be expanded due to insufficient hydraulic capacity at 2.6 mgd are the comminutors (2 mgd firm capacity) and secondary transfer pumps (0.7 mgd firm capacity, each BTU).** No additional facilities beyond the Phase I modifications must be expanded to meet the Phase II requirements.

FLORIDA CITIES WATER COMPANY

WATERWAY ESTATES WWTP

Secondary Treatment Design Calculations
for
Proposed Modification
to
1.0 MGD Advanced Waste Treatment

May, 1989

SOURCE, INC.
Engineers - Planners
1334 Lafayette Street
Cape Coral, Florida

By William D. Harrop
William D. Harrop, Jr. P.E.
Florida Registration No. 23949

Date 5-16-89

1. DESIGN CONCEPT

Modification of the Waterway Estates WWTP is planned to be accomplished by construction of a circular steel treatment plant that is presently located at the Fiesta Village site. This plant has a total aeration capacity of 556,495 gallons and a clarifier volume at 191,967 gallons. It is also proposed to construct a flow equalization tank from which effluent will be pumped to the two treatment plants that will be operated in parallel and ahead of the denitrification filters.

The treatment method proposed is the extended aeration modification of the activated sludge process. This was chosen to assure near complete nitrification prior to discharge to the biological denitrification filters and ultimate disposal to the Calossahatchee River. An anoxic zone is provided to enhance nitrification.

2. EFFLUENT LIMITATIONS

Disposal to the Caloosahatchee River requires that the effluent from the treatment plant contain no more than 5 mg/l of BOD5 and suspended solids, 3 mg/l total nitrogen and 0.5 mg/l total phosphorus. Discharge to the river will take place at the minus 6 foot contour interval, approximately 2,500 feet off shore.

3. DESIGN SEWAGE FLOW

Design ADF - 1.0 mgd

Diurnal Peak Factor - 3.0

4. INFLUENT CHARACTERISTICS

Influent characteristics listed below are based upon a minimum 12 months analysis of the wastewater entering the existing system.

PARAMETER	CONCENTRATION (MG/L)	LBS. PER DAY AT 1.0 MGD
BOD5	270	2,252
TSS	320	2,669
Total N	35	292
NO2-NO3	0.6	5.0
Organ. N	12	100
TKN	33	275
P	9	75
NH3-N	25	209

5. Design Standards

Design standards are based upon the recommendations for extended aeration contained in the October, 1977 edition of the "Process Design Manual - Wastewater Treatment Facilities for Small Sewered Communities" published by the U.S. Environmental Protection Agency, and the Ten State Standards.

ITEM	DESIGN CRITERIA
F/Mv	0.05 to 0.15 lb BOD5/day/lb MLVSS
Sludge Residence Time	20 to 30 days
MLSS	3000 to 6000 mg/l
Volumetic Loading	10-25 lbs/BOD5/day/1000 C.F.
Hydraulic Detention Time	18 to 36 hours
Recycle Ratio (R)	0.75 to 1.5
SCFM Air/lb BOD5 Removed	3000 to 4000
lb. Oz/lb BOD5 Removed	1.5 to 1.8
Reduction of NH3 as N	90% min
Volatile part of MLSS	0.6 to 0.7 (verified)

6. SECONDARY TREATMENT UNIT CAPACITIES

UNIT	EXISTING (GAL)	NEW (GAL)	TOTAL (GAL)
Aeration	442,012	556,445	998,507
Equalization		165,000	165,000
Clarifier	164,429	191,967	356,396
ASD	98,246	1,866	100,112

Chlorine Contact (None - UV disinfection to be used)

7. AERATION CAPACITY DESIGN - 1.0 mgd

MLVSS = 4500 x .65 = 2925 mg/l
Influent BOD5 = Li = 270 mg/l
Effluent BOD5 = Le = 5 mg/l

BOD Loading:

$F = 8.34(Q)(Li-Le)/1 \times 10^6$
$F = 8.34(1.0 \times 10^6)(270-5)/1 \times 10^6$
$F = 2,210$ lbs per day

Solids in Aerator:

$F/Mv = 0.15$
$Mv = F/0.15$
$Mv = 2,210$ lbs/day/0.15
$Mv = 14,733$ lbs

Aerator Volume:

$V = [(0.133)(Q)Li-Le)]/[(MLVSS)(F/Mv)]$
$V = [(0.133)(1.0 \times 10^6)(270-5)]/[(2925)(.15)]$
$V = 35,245,000/438.8$
$V = 80,321$ C.F. = 600,804 gal.
o.k. less than 998,507 gal.

F/Mv based on 133,490 C.F. aerator

$F/Mv = [0.133(Q/V)(Li-Le)]/MLVSS$
$F/Mv = [0.133(1.0 \times 10^6/133,490)(270-5)]/2925$
$F/Mv = 264/2925$
$F/Mv = 0.090$ o.k. less than 0.15, greater than 0.05

Sludge Retention Time:

$SRT = 1 \div [(a)(F/Mv)-b]$ a = 1.1, b = .08 for unsettled sewage
$SRT = 1 \div [(1.1)(0.090) - .08]$
SRT = 53 days o.k. greater than 20

Net Sludge Production:

$Mw = (Mv)[a(F/Mv) - b]$
$Mw = (14,733)[1.1(0.09) - .08]$
$Mw = 280$ lbs/day

Solids to Clarifier with 100 percent return:

$Mv/A = [(MLVSS)(8.34)(Q)]/[(A)(24)(10^6)]$
$Mv/A = [(4500)(8.34)(1.0 \times 10^6)]/[(3652)(24)(10^6)]$
$Mv/A = 0.25$ o.k. less than 1.25
(Note: Total clarifier surface area = 3652 S.F.)

Return Flow to Clarifier:

$Qr = Q\ Ss \div (Cs - Ss)$
$Qr = (1.0 \times 10^6)(4500)/(10,000 - 4500)$
$Qr = 0.818$ mgd = 568 gpm

Hydraulic Detention Time in Aerator:

$T = (V/Q)\ 24$ hr
$T = (998,507/1.0 \times 10^6)/24$
$T = 23.96$ o.k. greater than 18

9. Clarifier Design

Total Volume = 356,396 gal = 47,646 C.F.
Total Surface Area = 3,652 S.F.
Total Weir Length = 295 ft.

Detention Time:

$T = (v/Q)\ 24$
$T = (356{,}396/1.0 \times 10^6)\ 24$
$T = 8.55$ hrs

Surface Settling Rate:

$SSR = Q/A$
$SSR = 1.0 \times 10^6/3652$
$SSR = 274$ gal/SF/day

Weir Rate:
$WR = Q/L$
$WR = 1.0 \times 10^6/295$
$WR = 3390$ gal/day/L.F.

FLORIDA CITIES WATER COMPANY

WATERWAY ESTATES WWTP

NITRIFICATION CALCULATIONS

for

PROPOSED MODIFICATION

to

1.0 MGD ADVANCED WASTE TREATMENT

MAY 1989

Prepared by

SOURCE, INC.
ENGINEERS AND PLANNERS
1334 LAFAYETTE STREET
CAPE CORAL, FL 33910

William D Harrop Jr
WILLIAM D. HARROP, JR., P.E.
FL REG. NO. 23949

DATE: 5-16-89

1. GENERAL INFORMATION

The reader is referred to the secondary treatment calculations prepared for the proposed extended aeration system. The design capacity of the proposed treatment facility is 1.0 mgd. The design presented will follow the recommendations and procedures contained in the PROCESS DESIGN MANUAL FOR NITROGEN CONTROL published by the U.S. Environmental Protection Agency dated October 1975.

2. EFFLUENT LIMITATIONS

Due to the outfall discharge to the Caloosahatchee River, which is a Class III State surface water, the following effluent limitations have been imposed by DER for this project:

BOD_5	–	5 mg/l
Total Suspended Solids	–	5 mg/l
Total Nitrogen	–	3 mg/l
Total Phosphorus	–	0.5 mg/l

3. DESIGN PARAMETERS

Influent BOD_5	–	270 mg/l
TKN	–	33 mg/l
Temperature	–	17°C., 63°F.

Alkalinity	-	225 mg/l as CaCo3
MLSS	-	4500 mg/l
MLVSS	-	2925 mg/l
Aerator Volume	-	998,507 gal.
SRT	-	53 days
HDT	-	24 hrs.
Aerator D.O.	-	3.0 mg/l min.
Saftey Factor	-	3.0

4. PROCESS pH

Approximately 7.14 mg/l of alkalinity as $CaCO_3$ is consumed per mg/l of NH_4^+-N oxidized.

$$225 \text{ mg/l} - [\, 7.14\ (33)\,] = -10.6 \text{ mg/l}$$

This calculation indicates that the available alkalinity will be consumed by the ammonia oxidation process. To prevent pH depression below the recommended value of 7.2 mg/l it will be necesssay to add a buffering solution to the mixed liquor. It is proposed to use a 50% solution caustic soda (NaOH) that will be fed into the system at the beginning of the first aeration stage. It is calculated that 244 gals. per day will maintain an alkalinity of 50 mg/l in the mixed liquor. Caustic soda will be stored on site in an insulated fiberglass reinforced plastic tank installed below grade. A minimum capacity of 9,000 gals. will be provided.

5. GROWTH RATE OF NITRIFYING BACTERIA

Temperature = 17°C., DO = 3 mg/l, pH = 7.2

$\dot{u}_n = \hat{u}_n$ [DO/K_{O2} + DO] [1 − 8.33 (7.2 − pH)]

where: $\dot{u}n$ = maximum possible nitrifier growth rate.

$\hat{u}_n$ = maximum nitrifer growth rate.

K_{O2} = half saturation constant for O_2

$\dot{u}_n$ = 0.47 [$(e^{.098(T-15)}$] [3.0 / 3.0 +2.0] [1]

$\dot{u}_n$ = 0.73 days^{-1} = maximum nitrifier growth rate.

6. MINIMUM SOLIDS RETENTION TIME FOR NITRIFICATION

O_c^m = 1 / $\dot{u}_n$

where: O_c^m = minimum solids retention time in aerator for nitrification.

O_c^m = 1 / .73 = 1.37 days.

7. DESIGN SOLIDS RETENTION TIME

O_c^d = saftey factor x O_c^m

where: O_c^d = design solids retention time in days.

O_c^d = 3 x 1.37 = 4.11 days.

Actual solids retention time is 53 days.

8. NITRIFIER GROWTH RATE OF NITROSOMONAS

$u_n = 1 / 0_c{}^d$

$u_n = 1 / 4.11 = .227$ days^{-1}

9. <u>HALF SATURATION CONSTANT FOR AMMONIA OXIDATION AT 17 DEG.</u>

$Kn = 10^{0.051T-1.155}$

Where: K_n = half saturation constant for NH^4, mg/l.

T = temperature - 17° C.

$K_n = 0.51$

10. <u>STEADY STATE AMMONIA CONTENT OF EFFLUENT</u>

$u_n = \dot{u}_n [N_1 / K_n + N_1]$

$u_n = .73 [N_1 / .51 + N_1]$

$N_1 = 0.25$ mg/l

11. <u>ORGANIC REMOVAL RATE</u>

$u_b = 1 / 0_c{}^d = Y_b q_b - K_d$

where: Y_b = heterotropic yield coefficient, lb. VSS grown per lb. BOD_5 removed.

q_b = rate of substrate removal, lb. BOD_5 removed per lb. VSS per day.

K_d = decay coefficient, day^{-1}

Assume: Y_b = 0.65 lb. VSS / lb. BOD_5 removed.

K_d = 0.05 day^{-1}.

.243 = 0.65 q_b - 0.05

q_b = 0.45 lbs. BOD5 removed per lb. MLVSS per day.

12. DETERMINE MINIMUM HYDRAULIC DETENTION TIME

HDT_{MIN} = [S_o - S_1] / [MLVSS x q_b]

where: S_o = influent BOD_5, 270 mg/l

S_1 = effluent soluable BOD_5, 5 mg/l x 65%

MLVSS = 2925 mg/l

HDT_{MIN} = [270 - (5 x .65] / [2925 x .45]

HDT_{MIN} = 0.203 days = 4.9 hrs.

Actual HDT = 24 hrs.

13. ORGANIC LOADING PER UNIT VOLUME

Minimum required volume:

1.0 mgd: V = HDT x Q

V = .203 days x 1,000,000 gals. = 203,000 gal.

Actual aerator volume = 998,507 gal.

BOD_5 loading: 1 mgd x 8.33 x 270 mg/l = 2,249 lbs./day

BOD_5 per 1000 cu.ft. of total aerator plus anoxic zone:

2,249 / (998,507 / 7.48 / 1000) = 16.85 lbs.

BOD_5 per 1000 cu.ft. of aeration less anoxic:

2,249 / (778,834 / 7.48 /1000) = 21.63 lbs.

14. SLUDGE WASTING

Sludge inventory:

$I = 8.33 (X_1 \times V)$

where: I = VSS under aeration, lbs.

X_1 = MLVSS

V = Volume of aeration tank.

$I = 8.33 (2925 \times .9985)$

I = 24,328 lbs.

Solids Wasted per Day:

$S = I / \theta_c^d$

where: S = solids wasted per day.

θ_c^d = solids retention time, 53 days.

S = 24,328 lbs. / 53 days

S = 459 lbs./ day

FLORIDA CITIES WATER COMPANY

WATERWAY ESTATES W.W.T.P.

PROPOSED WASTEWATER TREATMENT PLANT IMPROVEMENTS

DENITRIFICATION CALCULATIONS

TETRA Technologies, Inc.
Proposal No. 1830
December, 1988

1. GENERAL INFORMATION

These calculations are based on the design calculations prepared for the nitrification process which is to precede the proposed denitrification process for Waterways Estates Wastewater Treatment Plant. The calculations have been made at the present design flow of 1.0 mgd and the anticipated future flowrate of 1.5 mgd. The design is for an attached-growth denitrification system as described in section 5.3 of the PROCESS DESIGN MANUAL FOR NITROGEN CONTROL, published by the U.S. Environmental Protection AGency in October 1975. The reader is referred to pages 5-23 through 5-25 of the referenced manual for the description of the Dravo denitrifying filters, which are the exact units proposed for the Waterways Estates plant. (TETRA Technologies, Inc. now designs and manufactures these units.) In addition to the recommended EPA sizing basis (loading in lbs. NOx-N per square foot of filter), kinetic data from suspended growth systems and data on the size and shape of the media will be used as a basis for performance prediction as well.

2. EFFLUENT LIMITATIONS

The following effluent limitations have been mandated by the Florida DER for this discharge to the Caloosahatchee River:

BOD_5	5 mg/l
TSS	5 mg/l
Total N	3 mg/l
Total P	0.5 mg/l

3. DESIGN PARAMETERS

Parameter	Treatment Plant Plant Influent[1]	Secondary Effluent (Feed to Denite® Filters) (1.0 mgd)	(1.5 mgd)
BOD_5, mg/l	270	13.5[2]	16[2]
TSS, mg/l	320	25[2]	30[2]
Total P, mg/l	9	1.0[2]	1.2[2]
Organic N, mg/l	12	0.4[2]	0.4[2]
NH_3-N, mg/l	23	0.5[2]	0.5[2]
NO_x-N, mg/l	0.6	11.5[2]	13.7[2]
Total N, mg/l	35.6	12.4[2]	14.6[2]
Alkalinity, mg/l	210	50[3]	50[3]
Lowest Temp	17°C(63°F)	--	--
pH		7.2+	7.2+

[1]Source, Inc. September '88 Nitrification Calculations
[2]Barth-Tec. November '88 letter
[3]Source, Inc. September '88 Nitrification Calculations indicate $CaCO_3$ feed to this level.

Flow: Present 1.0 MGD (700 gpm) average, 3.0 MGD (2085 gpm) peak hour.

Future 1.5 MGD (1040 gpm) average, 4.5 MGD (3125 gpm) peak hour.

4. <u>SIZING BASED ON SURFACE LOADING CRITERIA-EPA MANUAL</u>

From page 5-23, EPA Manual, the recommended surface loading on deep bed filters at 21°C is 2.5 gpm/ft^2, for municipal wastewater containing 20 mg/l NO_3-N. Since the lowest temperature of the wastewater based on historical data is 17°C, an adjustment must be made for temperature. Interpolating from figures 5-2 and 5-13 of the EPA Manual, a reduction in the rate of denitrification of about 22% will occur.

The impact of the nitrate-nitrogen concentration in the feed to the denitrifying filters being less than 20 mg/l must also be considered. As indicated in section 3, the total concentration of nitrate and nitrite nitrogen will be less than 14 mg/l. This constitutes a 30% reduction in loading to the filters.

Based on these two factors, the design average loading rate should be:

$$2.5\ gpm/ft^2 - 0.22\ (2.5\ gpm/ft^2) + 0.30\ (2.5\ gpm/ft^2) = 2.7\ gpm/ft^2$$

Based on this loading rate, at an average flow of:

<u>1.0 mgd</u>, the required filter surface area would be <u>260 ft^2</u> and at

<u>1.5 mgd</u>, the required filter surface area would be <u>385 ft^2</u>.

5. PRESENTLY USED SIZING CRITERIA

Studies over many years of the Dravo denitrification filter design have produced data which fits the equation:

$$\text{Det. Time} = 2 \frac{(Ci^{1/2} - C^{1/2})}{K}$$

Based on data from operating facilities using the Dravo filter design, K at 17°C = 0.75. The typical filter design depth is 6 ft. The influent NO_x-N concentration is 13.7 mg/l.

If the effluent NO_x-N concentration (C) is 1.0 mg/l, the required detention time is

$$\frac{2(13.7^{1/2} - 1^{1/2})}{0.75} = 7 \text{ minutes}$$

At a 6' media depth, the surface loading rate would be 6'/7 min. x 7.48 gal/ft.^{3} = 6.4 gpm/ft^2

At the future flow conditions (1.5 mgd avg., 4.5 mgd max.)

Average flow requires 173 ft^2 surface area
Peak flow requires 490 ft^2 surface area

If the effluent NO_x concentration (C) is 1.5 mg/l, then the required detention time is reduced to

$$\frac{2(13.7^{1/2} - 1.5^{1/2})}{0.75} = 6.5 \text{ minutes}$$

This would produce a surface loading rate of 6.9 gpm/ft^2

At future peak flows, the required surface area would be 450ft^2.

Based on these calculations, the proposed design of 462 ft^2 of surface area will produce at average flows an effluent NO_x-N of less than 1.0 mg/l, and at peak flows, an effluent NO_x-N of less than 1.5 mg/l.

6. PERFORMANCE ESTIMATION USING SUSPENDED GROWTH KINETICS

Little data exists in the literature (and in the EPA manual) regarding the kinetics of attached growth microorganisms for denitrification. In order to evaluate the sizing of the denitrification process, the available attached growth kinetic data will be supplemented with suspended growth data where necessary. Additionally, the manufacturer has developed some kinetic data as well, which will be used for comparison purposes. These calculations will serve as a check on those presented in section 4.

6.1 from EPA manual

$\hat{u}_D$ = maximum denitrification growth rate d^{-1}
$\hat{q}_D$ = maximum nitrate removal rate $\#NO_3$-N/#VSS/d
K_D = half-saturation constant for Denite[R] = 0.06 mg/l (p3-37)
Y_D = denitrifier gross yield (lb VSS/lb NO_3-N/d)
Kd = 0.04 d^{-1}

EPA manual values for

$\hat{q}_D = 0.11 \frac{\#NO_x\text{-Nd}}{\#VSS}$ (from fig 5-2, for suspended growth)

Y_D = 0.6 lb VSS/lb NO_3-N/d (Table 3-10, for susp. growth)

In contrast to this, TETRA has determined that for attached growths in sand filters $\hat{q}_D$ is frequently 0.6 $\frac{\text{lb } NO_3\text{-N}}{\text{lb VSS/d}}$ or higher. Calculations will be conducted using both values of q_D, to illustrate the difference.

eg. 3-49: $\hat{q}_D = \frac{\hat{u}D}{Y_D}$ using EPA $\hat{q}_D$: $\hat{u}_D = 0.066\ d^{-1}$

using TETRA $\hat{q}_D$: $\hat{u}_D = 0.36\ d^{-1}$

6.2 SOLIDS RESIDENCE TIME

for Waterways Estates - using Plug Flow model (eg. 5-5)

$$\frac{1}{\theta_c} = \frac{Yd\ (q_D)\ (D_oD_1)}{(D_o-D_1) + K_D \text{ In } \frac{(Do)}{(D_1)}} - Kd \quad \text{for } D^1 = 1\text{mg/l effl } NO_x\text{-N}$$

for $\hat{q}_D = 0.11\ d^{-1}$, θ_c^d = 53 days   for q_D = 0.6, θ_c^d = 3.6 days

6.3 DESIGN NITRATE-N REMOVAL

eg. 3-50 $\frac{1}{\theta_c} = Y_D\ \dot{q}_D - Kd$

for $\hat{q}_D = 0.11\ d^{-1}$ $\frac{1}{53 \text{ days}} = 0.6\ (q_D) - 0.04$: q_D = 0.10$\#NO_3$-N/#VSS-d

for $\hat{q}_D = 0.6\ d^{-1}$ $\frac{1}{3.6 \text{ days}} = 0.6\ q_D - 0.04$: q_D = 0.52 $\#NO_3$-N/#VSS-d

6.4 STEADY STATE NITRATE EFFLUENT CONCENTRATION

(eg. 5-1) $q_0 = q_0 \dfrac{D_1}{K_D + D_1}$

for $\hat{q}_0 = 0.11d^{-1}$, $D_1 = 0.5$ mg/l NO_3-N

for $\hat{q}_0 = 0.6d^{-1}$, $D_1 = 0.4$ mg/l NO_3-N

6.5 HYDRAULIC DETENTION TIME

(eg. 5-2) $q_0 = \dfrac{D_o - D_1}{X_1 - HT}$ where X_1 = MLVSS (mg/l)
HT = hydraulic d.t. (days)

To evaluate X_1, - for 3 reactors, each 6' deep (media) by 14' diameter
void volume - 40% for sand
assume 15% voids are filled with biomass (conservative)
assume 60% of biomass solids are active and viable (conservative)
per reactor, MLVSS = 923 ft^3 sand x 0.40 x 0.15 x 0.60
= 33.25 $ft.^3$ biomass
= 2074 lb biomass/923 ft^3 = 36 g/l MLVSS
= 36,000 mg/l

$\hat{q}_0 = 0.11d^{-1}$ HT - $\dfrac{13.7-0.5}{0.10\ (36000\ mg/l)}$ = $0.0037d^{-1}$ = 5.4 minutes

Hydraulic Loading Permissible = 6 ft/5.4 min. x 7.48 = 8.3 gpm/ft^2

$\hat{q}_0 = 0.6d^{-1}$: HT = $0.0007d^{-1}$ = 1.0 min.: Hydraulic Loading = not limiting (44 gpm/SF)

6.6 SLUDGE WASTING SCHEDULE

NOTE: In filtration, solids will accumulate from the removal of effluent TSS from the preceding biological treatment process, as well as the generation of solids through denitrification. The capacity of the filter to accumulate solids is limited to about 30% of the total void space available.

a. Solids accumulating from secondary effluent

(30 mg/l -5 mg/l) @ 1040 gpm = 312#/d

b. Solids produced by denitrification

eg. 5-3 $S = \frac{I}{\theta_c}$ = for $\hat{q}_0 = 0.11$: $\frac{6220\#}{53 \text{ days}} = 117$ lb/dd

for $\hat{q}D = 0.6$: $\frac{6220\#}{3.6 \text{ days}} = 1730$ lb/d

Check-based on yield
0.6 #VSS/#NO_3-N/d (160#/d) = 96 lb/d VSS formed

Total solids wasting:

for $\hat{q}_0 = 0.11$ -- 312#/d + 117#/d = 429#/d

for $\hat{q}_0 = 0.6$ -- 312#/d + 1730#/d = 2040#/d

c. Capacity of filters to store solids

a) Assume reseeding the filter after backwash fills 10% of the voids: 20% of voids remain to be filled. 40% of 20% of 923 $_{ft}3$ = 74 ft_3 @ 62#/ft_3 = 4590# solids/filter.

for $\hat{q}_0$ - 0.11, filter run = 32 days

for $\hat{q}_0 = 0.6$, filter run = 6.8 days

NOTE: In actual operation, filters are typically backwashed once or more per week which provides some confirmation for the higher nitrate removal rate.

7. **CONCLUSIONS**

A. Based on the EPA Manual's suggested sizing method (Section 4) using an average surface loading rate, 385 ft^2 of filter surface area must be provided.

B. Based on the manufacturer's sizing method which in turn is based on the successful operation of several denitrifying filters since the publication of the EPA Manual, 462 ft_2 of filter surface area will provide a satisfactory effluent.
(below 1.5 mg/l NO_x-N) at future peak flows.

C. Using the kinetic data provided in the EPA manual, a permissible hydraulic loading of 8.3 gpm/ft^2 will produce an effluent quality of 0.5 mg/l NO_3-N, and will at future maximum flows require a surface area of 377 ft^2. Using kinetic data collected since the EPA Manual publication by the manufacturer predicted effluent quality will be 0.4 mg/l NO_3-N. (It should be noted that, due to data gaps, some suspended growth kinetic data was utilized.)

Based on these calculations, the proposed installation of three 14' diameter deep bed filters, as designed and provided by TETRA Technologies, will be sufficient to produce an effluent total nitrogen of 3.0 mg/l or less, so long as the filter influent characteristics are as indicated in section 3.