Table 1: Water Quality Data to Collect During Jar Testing		
Sampling Point	**Parameter(s)**	**Measurement Purpose(s)**
Settled Water	TOC	• Develop TOC-coagulant dose relationship
	pH	• Determine effect of pH on settled water quality
	Turbidity	• Evaluate turbidity removal
	UV-254	• Characterize removal of organics and DBP precursors
	DBPs	• Evaluate effect of enhanced coagulation on TTHM and HAA formation under simulated distribution system (SDS) conditions

5.0 TESTING RESULTS

Table and figures detailing the results from the jar tests are attached.

6.0 TESTING CONCLUSIONS

Jar tests indicated that enhanced coagulation can achieve between 30 and 35 % removal at a moderate coagulant dose of 50 mg/L. These values are confirmed by the UV_{254} results, which indicate a removal of 30 –50 % at a coagulant dose of 50 mg/L.

ENHANCED COAGULATION TESTING PROTOCOL

1.0 - Treatment Study Times

Refer to Section 2 for discussion of study times. As a guide, to represent the seasonal variation of raw water, the jar testing would ideally be conducted during the following times:

- December/January
- May
- August

2.0 - Treatment Study Samples

The jar testing was conducted on 4 sets of raw water samples (prior to coagulant and oxidant addition) for the following conditions:

- Raw water coagulated at ambient pH at alum dosages of 0, 10, 20, 50, 80 mg/L.
- Raw water coagulated at ambient pH at ferric chloride dosages of 0, 10, 20, 50, 80 mg/L.
- Raw water coagulated at ambient pH at aluminum chlorohydrate dosages of 0, 10, 20, 50, 80 mg/L.

- Raw water coagulated at pH = 6.5 at an alum dose of 50 mg/L.
- Raw water coagulated at pH = 6.5 at a ferric chloride dose of 50 mg/L.
- Raw water coagulated at pH = 6.5 at an aluminum chlorohydrate dose of 50 mg/L.

- 20 mg/L of PAC addition to raw water followed by coagulation with alum and ferric chloride dosages of 50 mg/L. After PAC addition, the water in the jars should be mixed at slow speed (20 rpm) for 3 hours. (Total = 2 samples)

- 20 mg/L of PAC addition to raw water (with the pH previously lowered to 6.5) followed by coagulation with alum and ferric chloride dosages of 50 mg/L. After PAC addition, the water in the jars should be mixed at slow speed (20 rpm) for 3 hours. (Total = 2 samples)

- 30 mg/L of PAC addition to raw water followed by coagulation with alum and ferric chloride dosages of 50 mg/L. After PAC addition, the water in the jars should be mixed at slow speed (20 rpm) for 3 hours. (Total = 2 samples).

Results of the first testing may allow a more compact range to be used in subsequent tests, if necessary.

3.0 - Treatment Study Steps

The following procedure outlines the steps that were followed for the jar tests.

3.1 *Raw Water Characterization*

1. Extract one raw water sample prior to chemical addition and take the following measurements:
   - pH
   - turbidity
   - bromide
   - alkalinity
   - TOC
   - UV_{254} (filtered through a 0.45μm filter)

   These parameters should be measured on the same batch of water used for the enhanced coagulation testing.

2. Conduct an SDS-DBP test as described in step 7 of Section 3.2.

3.2 *Chlorine Dose Determination for Primary and Secondary Disinfection*

1. Follow Steps 1 – 4 of A.3.3 with the following exceptions.
   - To three jars, add a dose of 50 mg/L of each coagulant (medium dose).
   - To one jar, do not add any coagulant

2. Split each of the settled jars into three beakers. Add 1.0, 2.0, and 3.0 mg/L chlorine to the samples. Allow for 30 minute contact time. Measure the chlorine residual. After approximately 24 hours, measure residual chlorine. Under the assumption that 2/3 of the decay has occurred in the first 24 hours, select the appropriate dose to use in the tests to achieve residual of 0.5 - 1.0 mg/L in 72 hrs. Record the appropriate dose (to be used in 3.2 step 7) for each coagulant that achieves the residual objective. (We can assume that after 20 to 30 minutes of contact time, the chlorine concentration will drop by half again).

3.3 *Enhanced Coagulation*

1. Fill up six 2-liter square jars up to the 2-liter mark. Place the jars under the six-place mixer and set mixing at >200 rpm.

2. To four jars, add the desired alum dose. To one jar, add an alum dose of 50 mg/L and add sulfuric acid to lower the pH to 6.5. (Prior to beginning this step, determine the correct amount of acid needs to be added. Get the pH down to at least 6.8 and then add the coagulant. Acid needs to be added prior to the coagulant to maximize coagulation.) Leave one jar alone (without a coagulant dose). Conduct rapid mixing at >200 rpm (usually full speed) for one minute after chemical addition. Disposal syringes could be used to measure out and add coagulant.

3. Conduct flocculation for 20 minutes at about 50 rpm.

4. Cease mixing and allow the water to settle for 30 minutes.

5. After settling, conduct the following measurements on all jars:
   - pH [1]
   - turbidity [1]
   - TOC [2]
   - UV_{254} (filtered through a 0.45μm filter) [3]

     1 pH and turbidity on the settled water can be measured without further precipitation.
     2 The TOC samples should be collected in laboratory designated sample bottles with acid for preparation.
     3 UV_{254} sample should be filtered through a 0.45 μm syringe filter (rinsed previously with DI water. See attached UV spec.)

6. Repeat steps 1 through 5 for the remaining coagulants (ferric and aluminum chlorohydrate).

7. For the tests conducted at the alum dosages of 40 mg/L, after the settled water measurements are taken, also perform the following procedure for the SDS-DBP tests:
   - Add chlorine (dose previously determined in Step 1 of A.3.2) to the sample to achieve a target residual of about 2 mg/L after 30 minutes of contact time.
   - Chlorination can be done in beakers or glass stopped BOD bottles.
   - Add chlorine *(dose determined in step 2 of 3.3)* to the sample. Allow the sample to remain undisturbed in darkness for about 24 hours at room temperature.
   - Pour samples from the beakers or BOD bottles into the appropriate lab-supplied sample bottles which contain preservatives to quench the chlorine

residual. Analyze the samples for TTHMs and HAAs.
- Repeat measurements at 3 days for THMs and HAAs.

8. Repeat step 7 for the other coagulants (ferric and aluminum chlorohydrate) and for one jar without coagulant. (Total of 4 samples analyzed for TTHM, HAA).

3.3 *Enhanced Coagulation with PAC Addition*

1. Repeat step 1 from Section 3.2.

2. To jar 2 and 4 add acid to reduce pH to 6.5. To jar 1, 2, 4, and 5 add 20 mg/L PAC. To jars 4 and 6 add 30 mg/L PAC. Mix all jars at 20 rpm for 3 hours. At the end of the 3 hour period, increase mixing speed to >200 rpm.

3. Repeat steps 2 through 7 from Section 3.2 with the following exceptions:
   - in the coagulant addition step, add 50 mg/L of alum to jars 1,2, and 3; and 50 mg/L of ferric in jars 3, 4, and 5.
   - conduct DBP formation test for jars 3 and 6 (30 mg/L PAC, ambient pH, 50 mg/L coagulant)

4.0 - Sampling Requirements

The following table summarizes the total number of samples that will need laboratory analysis for each seasonal testing.

Table1 Samples for Analysis					
	Parameters				
Jar Test	Alk	TOC	Br[1]	THM[1]	HAA[1]
Raw water	1	1	1	1	1
EC at coagulant dose = 50 mg/L, ambient pH	0	15	0	3	3
EC at coagulant dose = 50 mg/L, pH = 6.5	0	3	0	3*	3*
EC with PAC = 20 mg/L	0	3	0	0	0
EC with PAC = 30 mg/L, ambient pH, (alum & ferric = 40 mg/L)	0	3	0	2	2
Total	**1**	**25**	**1**	**9**	**9**

1 Analysis of Br, THM, and HAA are not required to determine enhanced coagulation compliance. However, analysis of these parameters will allow us to estimate DBP levels after implementing enhanced coagulation.

* If 3 is too many, than just analyze the sample with ferric coagulant.

Key to Parameter Abbreviations

Alk - Alkalinity

Br - Bromide

TOC - Total organic carbon

THM - Trihalomethanes

HAA - Haloacetic acids

5.0 Water and Equipment Requirements

All water used in the testing described in previous sections should be representative of raw water for the proposed water treatment plant. Water for enhanced coagulation should be collected at the lake at a depth of 15 feet below the surface, the anticipated intake depth). A total of approximately 70 liters (20 gallons) of water will be necessary to conduct jar testing at each season.

The following is a list of equipment necessary for testing:

- One or two 6-place gang mixer with 2-liter square beakers.
- pH meter
- Turbidimeter
- UV spectrophotometer
- 0.45 μm syringe filters
- Colorimetric residual chlorine analyzer
- Sample bottles
- Beakers and/or BOD bottles

WESTERN COMAL REGIONAL WATER SYSTEM
ENHANCED COAGULATION JAR TEST
ANALYTICAL RESULT SUMMARY

Coagulant	Dose (mg/L)	Turbidity (NTU)	TOC[1] (mg/L as C)	TTHM[2] (mg/L)	HAA5[2] (mg/L)
Aluminum Sulfate	0	1.33	1.70	NM[3]	NM
	10	0.78	1.80	NM	NM
	20	0.43	1.70	NM	NM
	50	0.58	1.20	42	20
	80	0.48	1.40	NM	NM
	50	0.53	1.30	27	13
Ferric Chloride	0	1.28	1.70	NM	NM
	10	0.43	1.60	NM	NM
	20	0.43	1.60	NM	NM
	50	0.47	1.10	28	12
	80	0.43	1.10	NM	NM
	50	0.68	1.00	14	7.0
Aluminum Chorohydrate	0	1.18	1.60	NM	NM
	10	0.47	1.40	NM	NM
	20	0.28	1.40	NM	NM
	50	0.28	1.10	40	11
	80	0.23	1.10	NM	NM
	50	0.73	1.30	28	7.7
Alum+PAC=20	50	0.07	1.4	NM	NM
Ferric+PAC=20	50	0.05	1.1	NM	NM
Alum+PAC=20 (pH=6.5)	50	0.08	1.2	NM	NM
Ferric+PAC=20 (pH=6.5)	50	0.055	0.90	NM	NM
Alum+PAC=30	50	0.05	1.2	29	11
Ferric+PAC=30	50	0.02	1.1	18	7.7

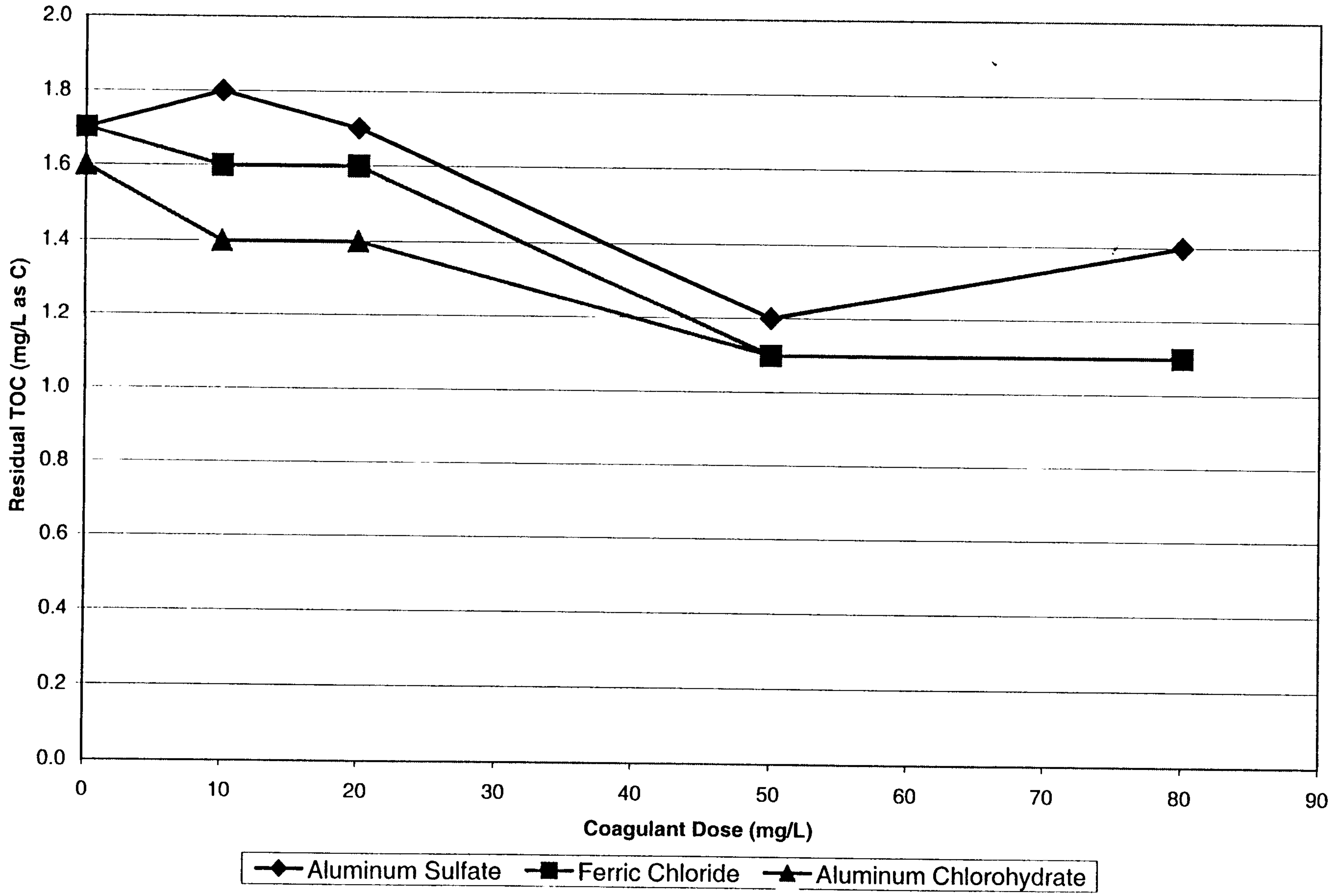
1 - TOC values reported in Table obtained from Montgomery Watson Laboratories. Samples were approximately 27 days old, but had been preserved. During shipping to MW, the samples were left in the shipping cooler for approx. 48 hours. Upon reciept at MW lab, their temp was 17C (compliance temp = 6C). The analyst suggested continuing with analysis b/c any microbial activity would not have been enough to alter TOC readings.
2 - DBP values reported in Table obtained from Montgomery Watson Laboratories. Samples were quenched after 72 hours for chlorine doses of 2.5 mg/L.
3 - NM=Not Measured

GBRA - REGIONAL WATER SYSTEM
ENHANCED COAGULATION JAR TEST
Raw Water Conditions

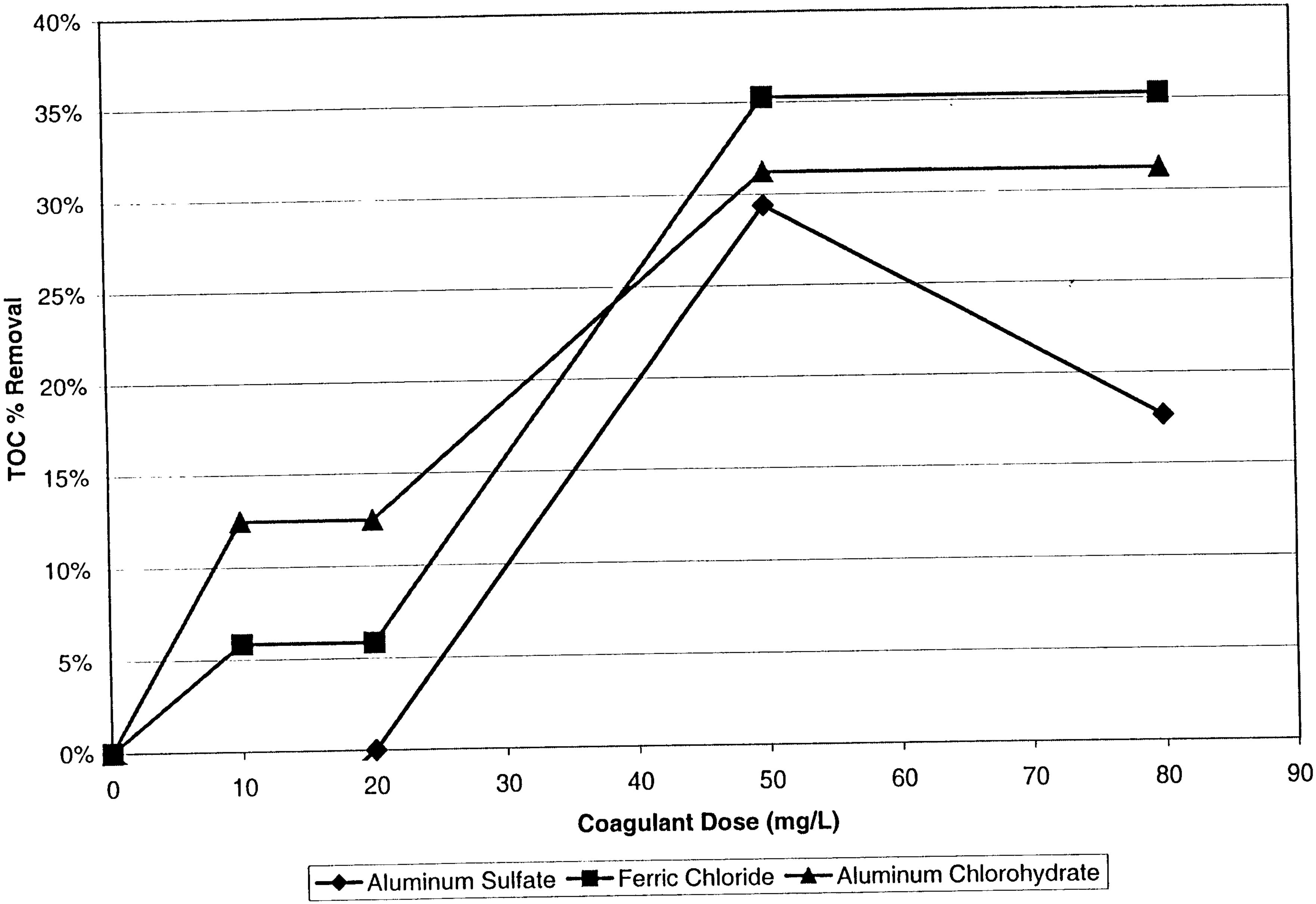
WESTERN COMAL REGIONAL WATER SYSTEM
ENHANCED COAGULATION JAR TEST
RAW WATER ANALYTICAL RESULTS

Parameter	Units	02/29/2000	03/01/2000	03/02/2000	03/03/2000
pH		8.3	8.24	8.3	8.23
Turbidity	NTU	1.5	1.4	1.4	1.25
UV-254	cm^{-1}	0.031	0.042	0.031	0.031
TOC	mg/L	2.0 (MW) 3.58 (GBRA)			
SUVA	L/mg-m	1.55			
Alkalinity	mg/L as $CaCO_3$	162			
Total Hardness	mg/L as $CaCO_3$	187			
Br-	µg/L	95			
TTHM	µg/L	<0.5			
HAA5	µg/L	NA	NA	NA	NA
Temperature	C	22.5	20.6	20.1	19.6

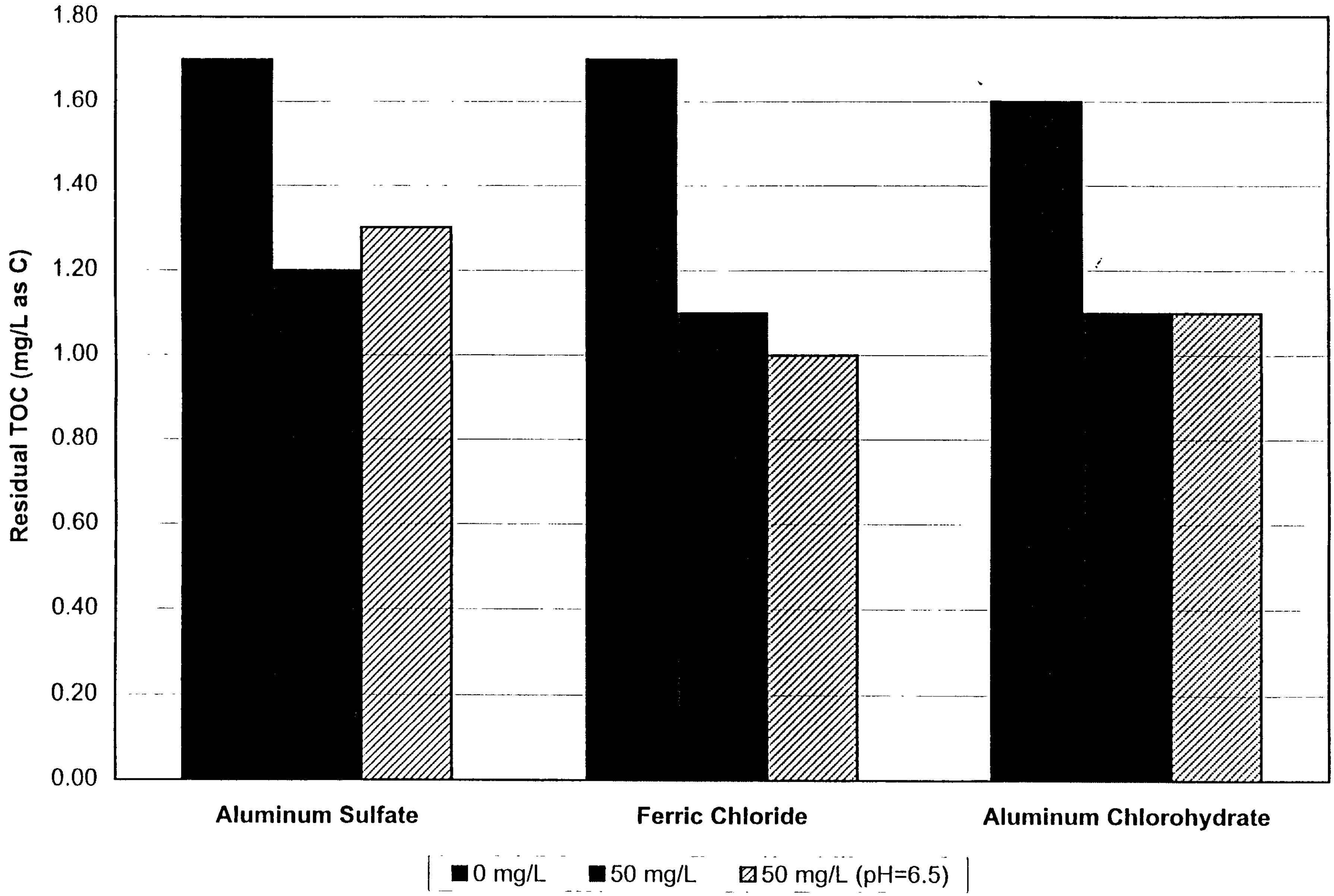
TOC vs. Coagulant Dose
(MW lab)



TOC % Removal vs. Coagulant Dose
(MW Lab)

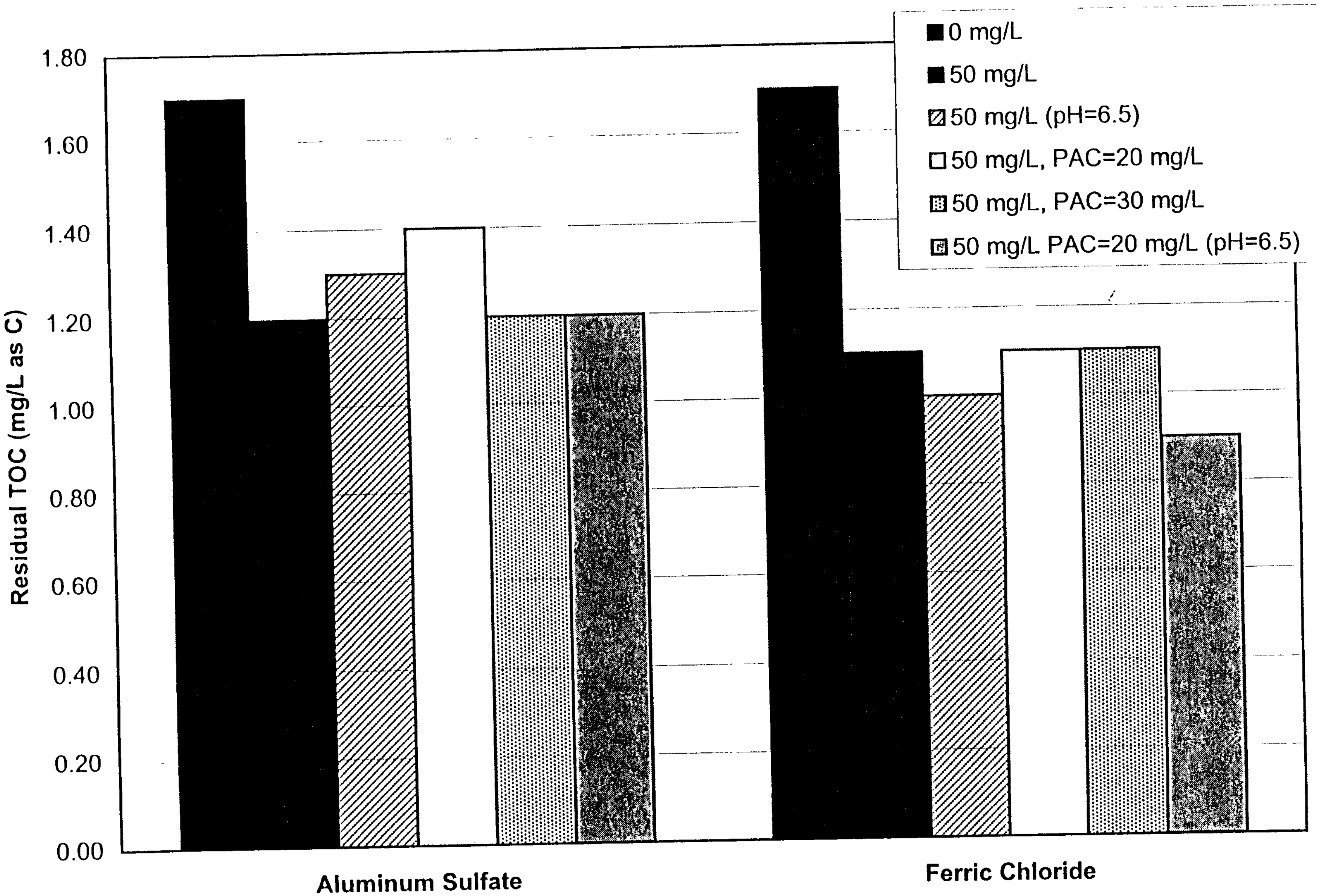


TOC - pH
(MW lab)

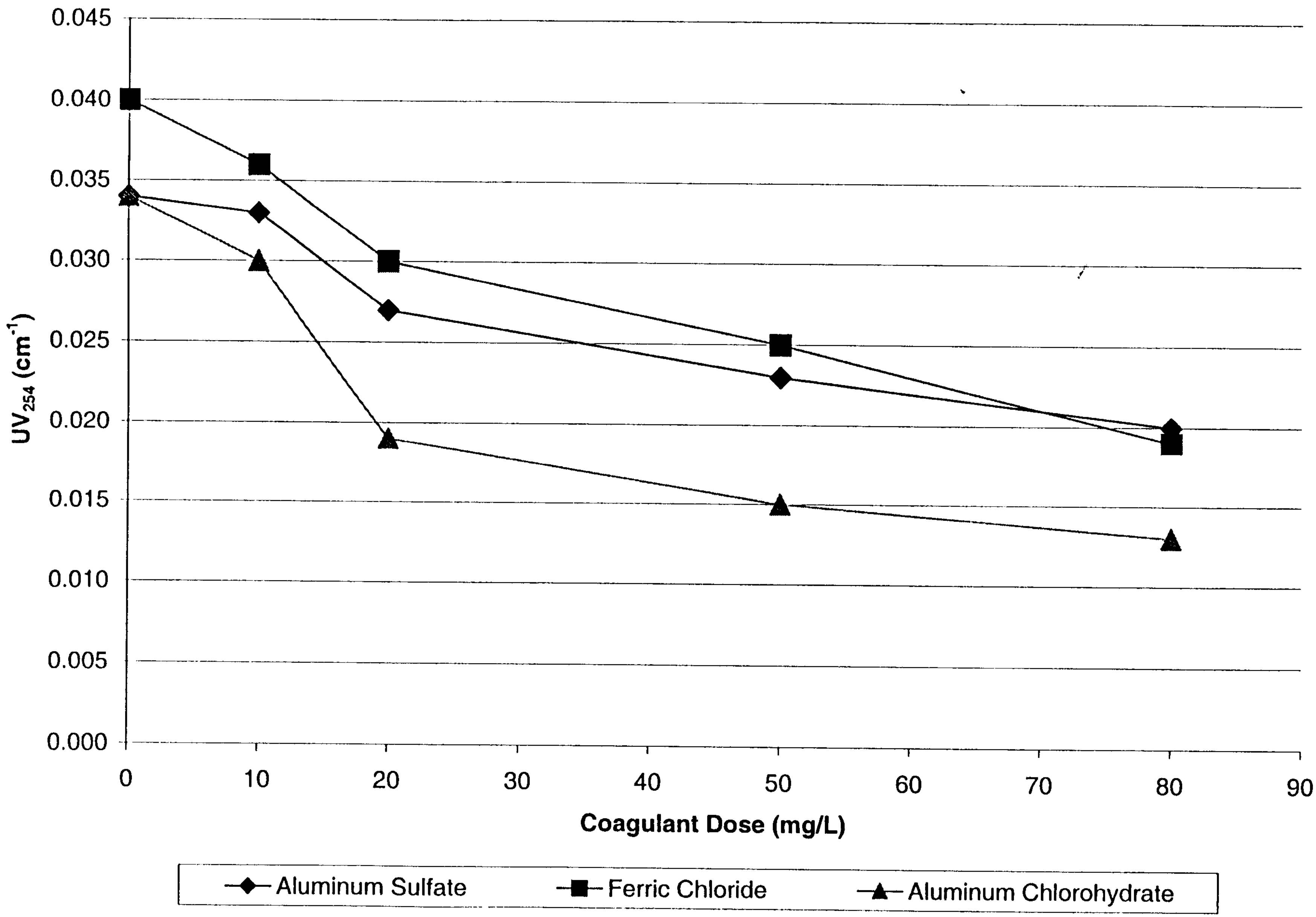


Residual TOC (mg/L as C)
1.80
1.60
1.40
1.20
1.00
0.80
0.60
0.40
0.20
0.00
Aluminum Sulfate
Ferric Chloride
Aluminum Chlorohydrate
0 mg/L
50 mg/L
50 mg/L (pH=6.5)

TOC-PAC
(MW lab)

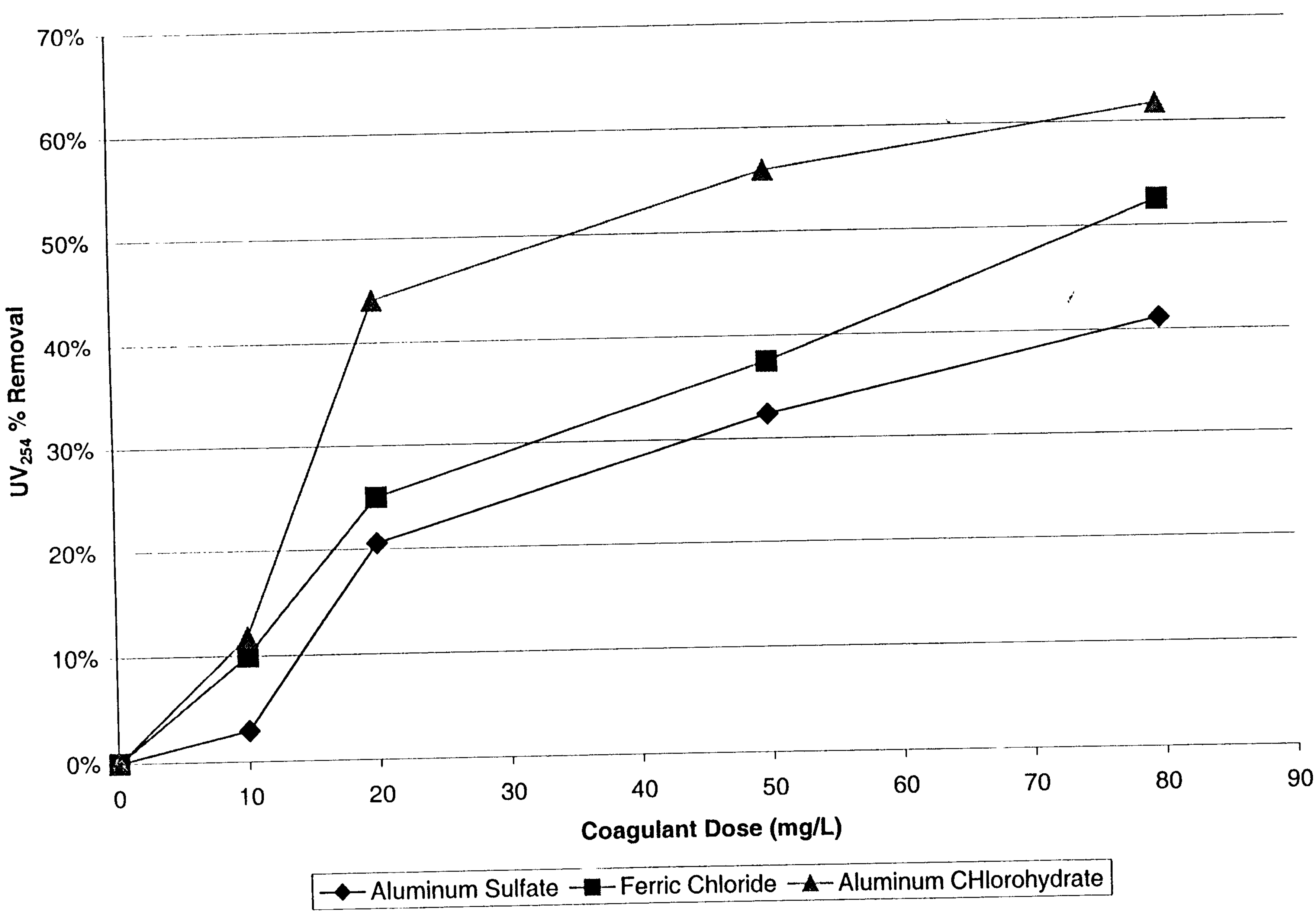


UV_{254}

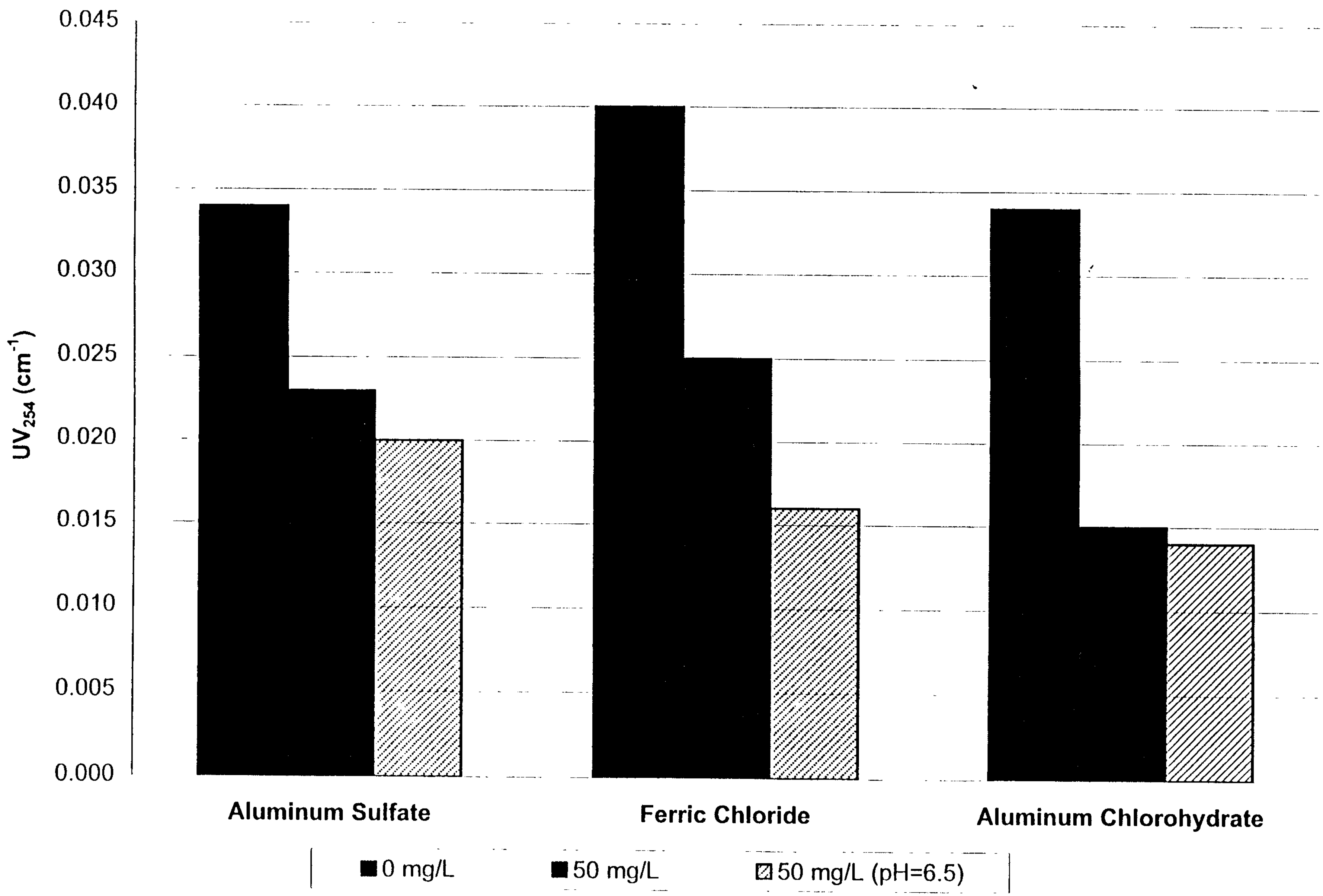


0.045
0.040
0.035
0.030
0.025
0.020
0.015
0.010
0.005
0.000
UV_{254} (cm^{-1})
0
10
20
30
40
50
60
70
80
90
Coagulant Dose (mg/L)
Aluminum Sulfate
Ferric Chloride
Aluminum Chlorohydrate

UV-% Removal

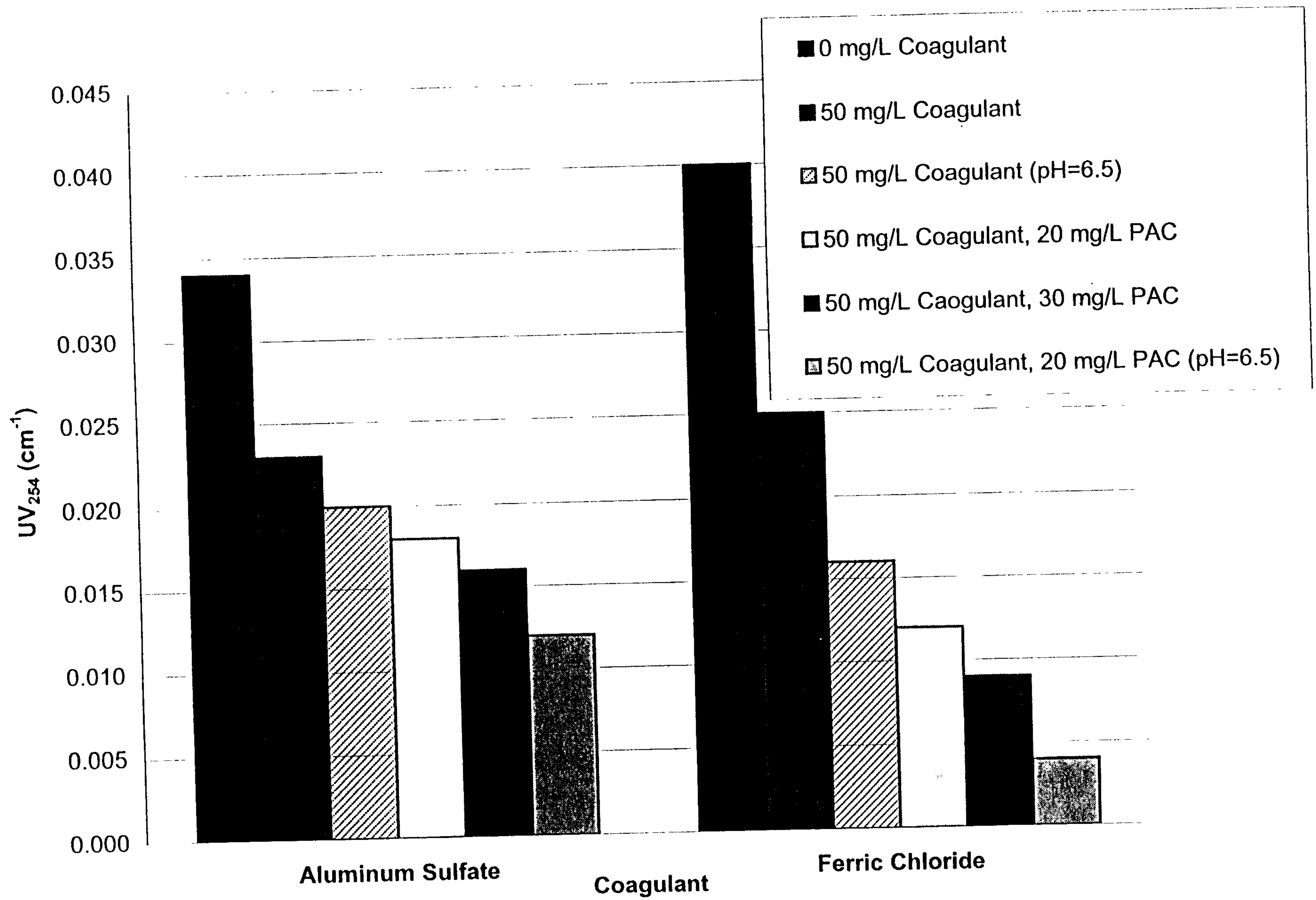

70%
60%
50%
40%
30%
20%
10%
0%
UV_{254} % Removal
0
10
20
30
40
50
60
70
80
90
Coagulant Dose (mg/L)
Aluminum Sulfate
Ferric Chloride
Aluminum CHlorohydrate

UV-pH



Western Comal Regional Water System

UV-PAC



0 mg/L Coagulant
50 mg/L Coagulant
50 mg/L Coagulant (pH=6.5)
50 mg/L Coagulant, 20 mg/L PAC
50 mg/L Caogulant, 30 mg/L PAC
50 mg/L Coagulant, 20 mg/L PAC (pH=6.5)
0.045
0.040
0.035
0.030
0.025
0.020
0.015
0.010
0.005
0.000
UV_{254} (cm^{-1})
Aluminum Sulfate
Ferric Chloride
Coagulant

WESTERN COMAL REGIONAL WATER SYSTEM
ENHANCED COAGULATION JAR TEST
Test 1 - ALUM

Date: 3/2/2000
Sampler Name: Tara Hickey/Karen Pappas
Concentration of Stock Chlorine Solution: 1100 mg/L as Cl_2
Concentration of Stock Aluminum Sulfate Solution: 10 mg/L as $Al_2(SO_4)_3 \cdot 14H_20$

Coagulation Conditions
pH, Turbidity, etc = Same as Raw Water Conditions

Settled Water Conditions

Sulfuric acid dose to lower pH to 6.5 = 35 drops of 2.0 N H_2SO_4

Parameter	Units	Sample ID	Coagulant Doses					
			0 mg/L	10 mg/L	20 mg/L	50 mg/L	80 mg/L	50 mg/L (pH=6.5)
pH		NA	8.34	8.07	7.83	7.57	7.20	6.62
Turbidity(a)	NTU	NA	1.33	0.78	0.43	0.58	0.48	0.53
UV-254	cm^{-1}	NA	0.034	0.033	0.027	0.023	0.020	0.020
TOC (GBRA)	mg/L	STOCA-0,10,...	2.56	2.87	2.82	1.48	0.92	1.04
TOC (MW lab - 3/29)			1.7	1.8	1.7	1.2	1.4	1.3
TTHM-2.5 (72)	µg/L	T1A2.5-50, (pH=6.5)	NA	NA	NA	42	NA	28
HAA5-2.5 (72)	µg/L	H1A2.5-50, (pH=6.5)	NA	NA	NA	20	NA	13

(a) Turbidimeter measured 0.025 on blank. Could not be standardized at zero. A value of 0 025 was subtracted from each turbidity measurement prior to documenting above.

Chlorine Residual at 72 Hours

Begin Contact Time: 10:00 AM | Quench Time: 3/5/00 8:45 AM

Chlorine Dose	Units	Sample ID	Coagulant Doses					
			0 mg/L	10 mg/L	20 mg/L	50 mg/L	80 mg/L	50 mg/L (pH=6.5)
1.5	mg/L	Cl1.5A-0,10,...	1.07	1.09	1.20	1.38	1.29	1.35
2.5	mg/L	Cl2.5A-0,10,...	1.78	1.95	1.98	>2.20	2.15	>2.20
3.5	mg/L	Cl3.5A-0,10,...	>2.20	>2.20	>2.20	>2.20	>2.20	>2.20

WESTERN COMAL REGIONAL WATER SYSTEM
ENHANCED COAGULATION JAR TEST
Test 2 - FERRIC CHLORIDE

Date:3/2/2000
Sampler Name:Karen Pappas/Tara Hickey
Concentration of Stock Chlorine Solution: 1100 mg/L as Cl_2
Concentration of Stock Ferric Chloride Solution:10 mg/L as $FeCl_3$

Coagulation Conditions
pH, Turbidity, etc = Same as Raw Water Conditions

Settled Water Conditions

Sulfuric acid dose to lower pH to 6.5 = 32 drops of 2.0 N H_2SO_4

Parameter	Units	Sample ID	Coagulant Doses					
			0 mg/L	10 mg/L	20 mg/L	50 mg/L	80 mg/L	50 mg/L (pH=6.5)
pH		NA	8.32	7.82	7.42	7.12	6.73	6.23
Turbidity (a)	NTU	NA	1.28	0.43	0.43	0.47	0.43	0.68
UV-254	cm^{-1}	NA	0.040	0.036	0.030	0.025	0.019	0.016
TOC (GBRA)	mg/L	STOCF-0,10,...	1.96	1.31	1.30	0.84	0.57	0.41
TOC (MW lab - 3/30)		STOCF-0,10,...	1.7	1.6	1.60	1.1	1.1	1.0
TTHM-2 5 (72)	µg/L	T2F2.5-50, (pH=6.5)	NA	NA	NA	28	NA	14
HAA5-2.5 (72)	µg/L	H2F2.5-50, (pH=6.5)	NA	NA	NA	12	NA	7

(a) Turbidimeter measured 0.025 on blank. Could not be standardized at zero. A value of 0.025 was subtracted from each turbidity measurement prior to documenting above.

Chlorine Residual at 72 Hours

Begin Contact Time:1:00 PM Quench Time: 3/5/00 11:30 AM

Chlorine Dose	Units	Sample ID	Coagulant Doses					
			0 mg/L	10 mg/L	20 mg/L	50 mg/L	80 mg/L	50 mg/L (pH=6.5)
1 5	mg/L	Cl1.5F-0,10,...	1.07	1.14	1.32	1.39	1.42	1.48
	mg/L	Cl2.5F-0,10,...	1.99	2.09	2.13	2.20	>2.20	>2.20
3.5	mg/L	Cl3.5F-0,10,...	>2.20	>2.20	>2.20	>2.20	>2.20	>2.20

WESTERN COMAL REGIONAL WATER SYSTEM
ENHANCED COAGULATION JAR TEST
Test 3 - ALUMINUM CHLOROHYDRATE

Date:3/2
Sampler Name:Tara Hickey/Karen Pappas
Concentration of Stock Chlorine Solution: 1100 mg/L as Cl_2
Concentration of Stock Aluminum Chlorohydrate Solution: 10 mg/L as $Al_3ClH_5O_5$-$2H_20$

Coagulation Conditions
pH, Turbidity, etc = Same as Raw Water Conditions

Settled Water Conditions

Sulfuric acid dose to lower pH to 6.5 = 25-30 drops of 2.0 N H_2SO_4

Parameter	Units	Sample ID	Coagulant Doses					
			0 mg/L	10 mg/L	20 mg/L	50 mg/L	80 mg/L	50 mg/L (pH=6.5)
pH		NA	8.35	8.18	8.04	7.81	7.70	6.90
Turbidity	NTU	NA	1.18	0.47	0.28	0.28	0.23	0.73(b)
UV-254	cm^{-1}	NA	0.034	0.030	0.019	0.015	0.013	0.014
TOC (GBRA)	mg/L	STOCAC-0,10,...	1.75	1.14	1.06	0.53	0.43	0.96
TOC (MW lab - 3/30)		STOCAC-0,10,...	1.6	1.4	1.4	1.1	1.1	1.3
TTHM-2.5 (72)	µg/L	T3AC2.5-50, (pH=6.5)	NA	NA	NA	40	NA	28
HAA5-2.5 (72)	µg/L	H3AC2.5-50, (pH=6.5)	NA	NA	NA	11	NA	17

(a) Turbidimeter measured 0.02 on blank. Could not be standardized at zero. A value of 0.02 was subtracted from each turbidity measurement prior to documenting above.
(b) Sample may have been agitated (introducing portions of settled floc) prior to testing turbidity.

Chlorine Residual at 72 Hours

Begin Contact Time: 4:30 PM — Quench Time: 3/5/00 5:45 PM

Chlorine Dose	Units	Sample ID	Coagulant Doses					
			0 mg/L	10 mg/L	20 mg/L	50 mg/L	80 mg/L	50 mg/L (pH=6.5)
1.5	mg/L	Cl2.5AC-0,10,...	1.24	1.27	1.31	1.33	1.35	1.13
2.5	mg/L	Cl3.5AC-0,10,...	1.83	>2.20	1.99	>2.20	>2.20	2.14
3.5	mg/L	Cl5.0AC-0,10,...	>2.20	>2.20	>2.20	>2.20	>2.20	>2.20

WESTERN COMAL REGIONAL WATER SYSTEM
ENHANCED COAGULATION JAR TEST
Test 4a - PAC (20 mg/L)

Date: 3/3/00
Sampler Name: Karen Pappas/Tara Hickey

Concentration of Stock Chlorine Solution: 1100 mg/L as Cl_2
Concentration of Stock WPH Calgon Powdered Activated Carbon Solution: 4.0 g/L
Concentration of Stock Aluminum Sulfate Solution: 10 mg/L as $Al_2(SO_4)_3\text{-}14H_20$
Concentration of Stock Ferric Chloride Solution: 10 mg/L as $FeCl_3$

Coagulation Conditions
pH, Turbidity, etc = Same as Raw Water Conditions

Settled Water Conditions

Sulfuric acid dose to lower pH to 6.5 = 25-30 drops of 2.0 N H_2SO_4

Parameter	Units	Sample ID	Coagulant Doses: Alum (50 mg/L)	Ferric (50 mg/L)	Alum (50 mg/L) pH=6.5	Ferric (50 mg/L) pH=6.5
pH		NA	7.6	7.16	6.8	6.75
Turbidity	NTU	NA	0.07	0.05	0.08	0.055
UV-254	cm^{-1}	NA	0 018	0.012	0.012	0.004
TOC (GBRA)	mg/L	SPACTOCA,F-50	1.17	0.75	0.86	0 30
TOC (MW - 3/27)	mg/L	SPACTOCA,F-50	1.4	1.1	1.2	0.90
TTHM (72)	µg/L		NA	NA	NA	NA
HAA5 (72)	µg/L		NA	NA	NA	NA

Chlorine Residual at 72 Hours

Begin Contact Time: 3/3/00 1:15 PM Quench Time: 3/6/00 12:00 PM

Chlorine Dose	Units	Sample ID	Coagulant Doses: Alum (50 mg/L)	Ferric (50 mg/L)	Alum (50 mg/L) pH=6.5	Ferric (50 mg/L) pH=6.5
1.5	mg/L	Cl1.5PACA,F-20	1.15	1.32	1.04	1.33
2.5	mg/L	Cl2.5PACA,F-20	1.99	>2.20	2.19	2.14
3.5	mg/L	Cl3.5PACA,F-20	>2.20	>2.20	>2.20	>2.20

WESTERN COMAL REGIONAL WATER SYSTEM
ENHANCED COAGULATION JAR TEST
Test 4b - PAC (30 mg/L)

Date: 3/3/00
Sampler Name: Karen Pappas/Tara Hickey

Concentration of Stock Chlorine Solution: 1100 mg/L as Cl_2
Concentration of Stock WPH Calgon Powdered Activated Carbon Solution: 4.0 g/L
Concentration of Stock Aluminum Sulfate Solution: 10 mg/L as $Al_2(SO_4)_3$-$14H_20$
Concentration of Stock Ferric Chloride Solution: 10 mg/L as $FeCl_3$

Coagulation Conditions
pH, Turbidity, etc = Same as Raw Water Conditions

Settled Water Conditions

Parameter	Units	Sample ID	Value for Various Coagulant Doses	
			Alum (50 mg/L)	Ferric (50 mg/L)
pH		NA	7.41	7.12
Turbidity	NTU	NA	0.05	0.02
UV-254	cm^{-1}	NA	0.016	0.009
TOC (GBRA)	mg/L	SPACTOCA,F-50	1.02	0.72
TOC (MW - 3/27)	mg/L	SPACTOCA,F-50	1.2	1.1
TTHM-2.5 (72)	µg/L	T4bPAC2.5-30F,A	29	18
HAA5-2.5 (72)	µg/L	H4bPAC2.5-30F,A	11	7.7

Chlorine Residual at 72 Hours

Begin Contact Time: 3/3/00 1:15 PM   Quench Time: 3/6/00 12:00 PM

Chlorine Dose	Units	Sample ID	Coagulant Doses	
			Alum (50 mg/L)	Ferric (50 mg/L)
1.5	mg/L	Cl1.5PACA,F-30	1.06	1.57
2.5	mg/L	Cl2.5PACA,F-30	>2.20	>2.20
3.5	mg/L	Cl3.5PACA,F-30	>2.20	>2.20

Regional Water Supply Project for Portions of Comal, Kendall, and Bexar Counties

Basis of Design Report
Contract B – Treated Water Delivery System Design

Prepared for

GBRA

GUADALUPE-BLANCO RIVER AUTHORITY

Prepared by

HDR

HDR Engineering, Inc.

June 2000

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June 2000

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Appendices

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Section 1
Introduction

1.1 Background

The Guadalupe-Blanco River Authority (GBRA) is developing this project to divert water from Canyon Lake and construct treatment and delivery facilities to provide treated water to customers in Comal, Kendall, and Bexar Counties. Engineering design services for the project are divided into two contracts. Contract A is for design of the raw water intake and pump station, treatment plant, and treated water pump station. Contract B is for permitting, pipeline route selection for the raw water and treated water pipelines, and hydraulic design of the treated water pump station, treated water pipeline, and customer delivery facilities.

1.2 Project Concept

The basic concept of this project is to fully utilize project facilities throughout the year and thereby reduce the cost of produced water to the lowest possible unit cost. Water not needed by In-District participants, either on an annual or day-by-day basis, will be purchased by either the San Antonio Water System (SAWS) or Bexar Metropolitan Water District (BMWD). With the Bexar County participants purchasing any unused water, the project will be able to sell all water that can be produced throughout the year. To do this, the project concept is as follows:

- Participants will contract for an annual delivery of water. The project will deliver that water at a uniform rate throughout the year in order to maximize use of all project facilities. In order for the participants to utilize water from the project throughout the year, they must continue to use their present water supply system to meet peak daily water needs and meet peak hour demands from their own storage facilities.
- The Base Project capacity will be 10,500 acft/yr. A Phased Project Option is possible to deliver additional quantities of water.
- The system will be sized to deliver about 7,900 acft/yr to Bexar County entities in the initial year and 2,600 acft/yr to In-District participants. As In-District water needs increase, the amount delivered to Bexar County will decrease, but will not be reduced below 4,000 acft/yr.
- All water will be sold on a wholesale basis with a uniform system rate schedule for each cost component.

1.3 Operational Philosophy

In the early years of project operation, water not used by In-District customers will be delivered to Bexar County entities (SAWS, BMWD, and the San Antonio River Authority [SARA]). Therefore, the design condition governing for most of the system, and particularly for the Bexar County entities, is to be able to meet the Initial Year Conditions. As water demands increase during the next 10 to 15 years, the In-District customers will increase their firm annual commitments up to their contracted amounts, thereby reducing deliveries to Bexar County. For specific facilities, such as the short transmission pipeline serving only the City of Boerne and the approach main to City of Fair Oaks, the design condition will be the Ultimate Year Condition.

1.4 Report Purpose

The purpose of this *Basis of Design Report* is to define each of the project facilities in sufficient detail to enable the final design process to proceed expeditiously. Requirements for final design of the treated water pump station, hydraulic requirements of the pipeline, and customer delivery facilities are defined. This information is used in development of updated project cost estimates. This Design Report also presents a recommended pipeline route and identifies permitting issues as currently understood.

Section 2
Delivery System Capacity and Design Conditions

2.1 Delivery Quantities

Table 2-1 lists the potential project participants that are in the project area. These entities have expressed interest in purchasing water from the project and can potentially meet the connection criteria described in Section 6.1. Table 2-1 also lists the "total water commitment" annual delivery quantities requested by the potential project participants. This quantity is referred to in this report as the "ultimate year" condition or capacity and is expected to be achieved by 2012.

Table 2-1.
Potential Project Participants and Water Delivery Requests

Potential Project Participant	Requested Total Water Commitment (acft)
City of Boerne	1,861
City of Fair Oaks	1,400
San Antonio Water System[1]	1,812.5
Bexar Metropolitan Water District[1]	2,137.5
San Antonio River Authority[1]	50
Comal Independent School District	150
Other Potential In-District Entities (Apex Water Co., Bulverde Hills, Murcia Development Co, Clyde Johnson, Double J Ranch, Cordillera Development, Tapatio Springs Development, and others)	3,116
Total	**10,527**

[1] The GBRA Board of Directors has authorized up to 4,000 acft/yr of Canyon Lake water be available to Bexar County entities. In the original allocation, SARA requested 375 acft/yr for water supply entities in north central Bexar County; SAWS and BMWD split the remaining water evenly, each receiving 1,812.5 acft/yr. Since then, BMWD has purchased some of the entities represented by SARA and 325 acft/yr of those requests are now assigned to BMWD, resulting in an allocation 2,137.5 acft/yr to BMWD, 50 acft/yr to SARA, and SAWS allocation remains at 1,812.5 acft/yr.

As shown in Table 2-1, the total annual water delivery will be 10,527 acft/yr, or about 9.4 MGD. In order to meet this annual delivery commitment, design flow rates must be somewhat higher to allow for scheduled outages resulting from necessary maintenance of system facilities and from unscheduled outages due to power failure, equipment malfunction, and leakage. To meet these requirements, the system design flow rates will be set to deliver the total annual water delivery in 95 percent of a year (i.e., flow rates will be such that the total annual delivery can be made in 347 days [0.95 x 365 days = 347 days]). Applying this factor creates a daily system delivery capacity of at least 9.9 MGD. For reference purposes through this report, the delivery capacity is rounded to 10 MGD, however, hydraulic calculations were made using 9.9 MGD. The locations of the potential project participants, along with the transmission pipeline route alternatives, are discussed later in Section 3.

2.1.1 Delivery Quantities – Initial Year Conditions

As briefly mentioned in Section 1.3, water not used by In-District customers in the early years of project operation will be delivered to Bexar County entities. Therefore, the design condition governing for most of the system, and particularly for the Bexar County entities, is to be able to meet the Initial Year Conditions. Each of the potential project participants was asked to provide expected Initial Year water demand in order to estimate the quantity available to be delivered to Bexar County entities. Table 2-1 summarizes the estimated annual deliveries to each entity and the corresponding daily flow rate.

Table 2-2.
Requested Initial Year Water Demand and Delivery Rates

Potential Project Participant	Requested Initial Year Delivery Quantity (acft)	Initial Year Delivery Rate[1] (MGD)
City of Boerne	500	0.47
City of Fair Oaks	800	0.75
Bexar Metropolitan Water District[2]	3,215	3.02
San Antonio River Authority	50	0.05
Comal Independent School District	150	0.14
Other In-District Entities	1,157	1.09
Subtotal	5,872	5.52
San Antonio Water System[3]	4,655	4.38
Total	10,527	9.90

1 Delivery rate is based on meeting annual delivery quantity in 347 days (95% of the year).
2 After consideration of hydraulic constraints and water needs in their service area, BMWD has requested that their water delivery be no more than 3,215 acft/yr.
3 SAWS delivery quantity is the balance left after other project participant's initial year needs have been met.

2.1.2 Delivery Quantities – Minimum Day Conditions

At times, In-District participants may not be able to take delivery of their contracted water supply due to low demand (wet weather or low winter demand) or due to a pipeline break or other outage in their system. During these times, additional water would be available for delivery to Bexar County entities. Because BMWD has capped its delivery to 3.0 MGD (3,215 acft/yr) and SARA will not be able to receive any significant additional quantity, all additional water available would have to be taken by SAWS. In order to estimate what the quantity of additional water may be, the following minimum day factors have been applied to the known average-year demands of In-District participants:

- A minimum day factor of 0.5, where peaking needs are met from available on-site storage.
- A minimum day factor of 0.25, where peaking needs are to be met by the system.

The initial year demand has been used where an average year demand was not known or available. The lesser value of the minimum daily demand and initial year demand has been evaluated as the minimum day delivery for the system. The resulting water availability to SAWS for minimum day conditions is summarized in Table 2-3.

Table 2-3.
Estimated Minimum Day Deliveries – Initial Year Conditions

Potential Project Participant	Initial Year Delivery Rates[1] (MGD)	Minimum Day Deliveries[2] (MGD)
City of Boerne	0.47	0.47
City of Fair Oaks	0.75	0.50
Bexar Metropolitan Water District[3]	3.02	3.02
San Antonio River Authority	0.05	0.05
Comal Independent School District[4]	0.14	0.00
Other In-District Entities	1.09	0.43
Subtotal	5.52	4.47
San Antonio Water System	4.38	5.43
Total	9.90	9.90

1 Delivery rate is based on meeting annual delivery quantity in 347 days (95% of the year).
2 Using minimum day factor of 0.5 times annual delivery quantity for entities with on-site storage available for peaking needs; 0.25 for entities using the system for peaking.
3 BMWD and SARA delivery rates held constant at annual delivery rate of 3.0 MGD and 0.05, respectively.
4 CISD delivery set to zero during low demand conditions.

2.2 Capacity Phasing Alternatives

2.2.1 Base Project (10 MGD)

The base project has the capacity to deliver 10,527 acft (9.9 MGD), as defined in Table 2-1 (Ultimate Year Condition), Table 2-2 (Initial Year Condition), and Table 2-3 (Minimum Day – Initial Year Condition). For reference purposes through this report, the delivery capacity is rounded to 10 MGD, however, hydraulic calculations were made using 9.9 MGD.

2.2.2 Phased Project (15 MGD)

In future years, the need may arise to be able to deliver water in excess of the Base Project capacities to meet area needs. To meet possible future demands and to give the operational flexibility to the GBRA, a phased project expansion has been developed. The Phased Project capacity is based on the possible availability of about 5,470 acft above the Base Project capacity for In-District use. Therefore, the Phased Project total delivery capacity would be about 16,000 acft/yr. Considering additional allowance for outages due to power failure, equipment malfunction, and leakage as described previously, this total capacity value equates to an approximate production capacity of 16,800 acft/yr. To provide the greatest flexibility to deliver this water throughout the system, the system hydraulic design provides for delivery of 75 percent of the additional water to In-District customers west of Bulverde. The remaining 25 percent of the additional water would be consumed by In-District customers east of Bulverde. Table 2-4 lists the potential project participants and delivery rates with total annual production of 16,800 acft/yr (phased annual delivery commitment of 16,000 acft/yr). The sum of delivery rates to all customers is 15.0 MGD for the phased project as shown in Table 2-4.

Table 2-4.
Possible Ultimate Year Water Delivery with Phased Project (16,000 acft/yr Total Delivery Capacity)

Potential Project Participant	***Requested Ultimate Year Delivery Quantity (acft)***	***Ultimate Year Delivery Rate[1] (MGD)***
City of Boerne	1,861	1.74
City of Fair Oaks	1,400	1.31
Bexar Metropolitan Water District	2,138	2.00
San Antonio River Authority	50	0.05
Comal Independent School District	150	0.14
San Antonio Water System	1,813	1.70
Subtotal	7,412	6.94
Other In-District Entities[2]	8,588	8.06
Total	16,000	15.00

[1] Delivery rate is based on meeting annual delivery quantity in 347 days (95% of the year).
[2] Other In-District deliveries have been increased above Base Project to account for the available additional phased project capacity.

Section 3
Pipeline Route

3.1 Preliminary Route Alternatives

Selection of a pipeline route is influenced by many factors. The pipeline route must be configured so that each of the water customers can connect to the line. However, the length of the route directly determines capital cost of the pipeline. In addition, land acquisition costs and the length of time required to secure the easements are determined by the value of the land located along the route as well as the number of tracts crossed. Many other factors, such as existing land development, environmental concerns, and physical features, influence both the pipeline cost and construction schedule.

As part of the preliminary engineering effort, Table 3-1 was developed to identify the key criteria to be considered in the route selection process. These criteria were presented to the GBRA and the customers for evaluation and comment. These criteria were then refined and categorized as "high," "medium," and "low" significance. It should be noted that designation as "low" does not indicate that the issue is not important. Each item on the criteria list was considered as an important consideration. The ranking designation was meant solely to provide a screening level for evaluation of the various alternatives.

The initial pipeline route alternatives were developed with attention to the all of the "high" importance criteria expect for the hydraulic requirements. The hydraulic analyses were then completed for each of the principal route alternatives. These analyses eliminated some of the alternatives from further consideration due to the limit selected for maximum working pressure in the pipeline. The remaining alternatives were then discussed with the GBRA and refined by closer examination of the "medium" and "low" importance criteria.

3.2 Recommended Route

After all criteria were examined, two significant route alternatives remained along with several minor route possibilities. Figure 3-1 presents the recommended pipeline route along with the remaining alternatives. The most significant route decision was between the route following the LCRA power transmission line or along Amman Road. The power line route is slightly longer than the Amman Road alternative, but the difference in capital cost was found to

Table 3-1.
Pipeline Route Selection Criteria

Importance	Selection Criteria	Project Impacts
High	Meet Customer Connection Requirements	Capital Cost
High	Minimize Pipeline Length	Capital Cost Maintenance Cost
High	Avoid Existing Development	Land Acquisition Cost Land Acquisition Schedule Construction Cost
High	Follow Existing Physical Features and Avoid Unusual Topography	Capital Cost For Pipeline Construction Capital Cost for Pumping Construction Operational Costs
High	Avoid Significant Environmental and Archaeological Features	Permitting Cost Permitting Schedule Mitigation/Recovery Costs Mitigation/Recovery Schedule
High	Meet Hydraulic Requirements	Capital Cost Operational Costs
High	Minimize Road and Railroad Crossings	Capital Cost
Medium	Evaluate Landowners	Land Acquisition Cost Land Acquisition Schedule Permitting Costs Permitting Schedule
Medium	Minimize Number of Tracts Crossed	Land Acquisition Cost Land Acquisition Schedule
Medium	Minimize Number of Driveway Cuts	Capital Cost Land Acquisition Cost Land Acquisition Schedule
Medium	Accessibility for Maintenance	Maintenance Costs
Low	Avoid Pipeline Bends	Capital Costs Operation Cost

be about 1 percent of the expected construction cost. Since the cost of the two alternatives was almost insignificant, the LCRA power line route was recommended for the following reasons:

- The power line is an established, straight easement that can be paralleled.
- There is the possibility to overlap the pipeline construction easements with the existing power line easements, which may make easement acquisition less difficult.
- The power line route has less development along the route resulting in lower land prices and fewer driveway crossings.
- The power line corridor has already been disturbed and may present fewer environmental challenges.

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Section 4
System Hydraulics

4.1 Delivery System Hydraulic Alternatives

The delivery system hydraulics for the preliminary pipeline route described in Section 3 have been evaluated and detailed in Technical Memoranda (TMs) Nos. 1-4, included as Appendices A-D of this report. The hydraulic analyses are based on preliminary routes established during an earlier phase of the current study. Although minor modifications to the preliminary route have been made throughout the hydraulic evaluation, the basic results of the analysis remain unchanged. A brief overview of each evaluated alternative and development of the recommended system design are provided below. Refer to the TMs for more detailed information.

4.1.1 Alternative 1 and Variations

Alternative 1 provides pumped delivery to all customers. Originally, Alternative 1 was limited to a single pump station located at the proposed water treatment plant (WTP), requiring maximum piping pressures up to 250 psi for the base project (10 MGD) and 300 psi for the phased project maximum system capacity. Analysis of this Alternative 1 variation is detailed in TM No. 1, *Water Delivery System Hydraulic Design and Expansion Alternatives*.

Concerns about such high operating pressures led to subsequent analyses of Alternative 1 to develop a recommendation for a system with a reduced maximum pressure. The optimal maximum operating pressure was found to be 200 psi, which would require a remote pump station in addition to the pump station located at the WTP. The remote pump station was located near the Fair Oaks Ranch delivery point within Comal County, approximately 26 miles downstream of the proposed Startz Hill Reservoir. These subsequent hydraulic analyses are detailed in TM No. 2, *Determination of Maximum Pipe Pressure Class*, and TM No. 3, *Hydraulic Design Alternatives for Reduced Maximum Pressure Class*.

Pumped delivery to all customers provided by Alternative 1 not only allows for flexibility in location of the water treatment plant, but is also an advantage to the overall system control strategy. Location of the pump station at the WTP permits the WTP to be located within a depressed area, allowing flow from the proposed raw water reservoir at Startz Hill to be conveyed by gravity.

4.1.2 Alternative 2 and Variations

Alternative 2 provides for pressurized gravity flow from the WTP to a remote pump station near US Highway 281 from which flow would be pumped to all downstream delivery points. As with the initial Alternative 1, a maximum pressure exceeding 200 psi was evaluated and detailed in TM No. 1, *Water Delivery System Hydraulic Design and Expansion Alternatives*. Further development of Alternative 2 led to modification of the pressurized gravity/remote pump system to allow for a maximum operating pressure of 200 psi within the pipeline (TM No. 2, *Determination of Maximum Pressure Class*).

The main advantage of Alternative 2 is the energy cost savings and lower pressures occurring within the gravity fed portion of the system. However, this alternative would require larger diameter lines (balancing the economic savings of lower pressure pipelines with increased costs for larger diameter piping), restrict selection for the WTP location, and limit the potential for future capacity expansion of the system.

4.1.3 Alternative 3

Alternative 3 involves gravity delivery to Bexar Metropolitan Water District (BMWD) in Bexar County, requiring the pipeline route to be lengthened and reconfigured along Smithson Valley Road. A remote pump station located in the vicinity of the Ammann Road and Bulverde Road intersection would be required to pump flows west toward Fair Oaks Ranch, Boerne, and the San Antonio Water System.

The advantage of this alternative is that less energy would be required for the system since water is conveyed by gravity for areas east of the Ammann Road and Bulverde Road intersection. System pipeline pressures would also be relatively low for these areas. However, the disadvantages of this alternative were found to outweigh the advantages, thus eliminating the alternative from further consideration. Hydraulic evaluation of this alternative found that about 13 miles of the piping system would violate the minimum velocity criteria of 2 feet per second to accomplish gravity delivery to BMWD. Other disadvantages include higher cost for piping due to longer length and larger diameter, less flexibility in WTP location, and less than desirable delivery pressures available at the BMWD delivery point.

4.2 Recommended Hydraulic Alternative

Based on analyses presented in TM Nos. 1-3, the recommended delivery system for the project is Alternative 1, providing pumped delivery to all customers from a pump station located at the WTP in conjunction with a remote pump station. The maximum pressure within the greater part of the delivery system should be 200 psi. This recommendation is based on evaluation of the advantages and disadvantages of the system hydraulics and economic and non-economic concerns presented in the hydraulic TMs.

Locating a pump station at the plant appears to offer substantial economic benefits for the treated water delivery system. An additional benefit includes the ability to locate the WTP at a lower elevation, permitting gravity feed from a reservoir on Startz Hill to the WTP. This benefit is pronounced for system control, as described further in Section 4.3 and the Contract A Basis of Design Report.

Hydraulic modeling of Alternative 1, based on the original Ammann Road alignment, was performed using Pipe 2000, an updated version of KYPIPE, to define a delivery system configuration capable of satisfying the system delivery and control requirements. Although the recommended alignment along the power transmission line varies somewhat from the alignment originally modeled, the general hydraulic evaluation of the system remains the same and is appropriate for the level of detail required for determination of the major general piping and pumping system requirements. The recommended alignment was evaluated for major topographic features that may vary the hydraulics of the system design. These features of the recommended alignment were considered in the hydraulic evaluation and incorporated into development of the final recommended hydraulic design alternative, as described in TM No. 3, Appendix C. Resulting piping and pumping requirements for the recommended alignment are expected to be comparable for use in design and project cost estimation. The general hydraulic criteria and constraints applied to the hydraulic modeling include the following:

- A minimum pipeline working pressure of 10 psig within the system during normal conditions. This value was developed with consideration for Texas Natural Resource Conservation Commission (TNRCC) guidelines for potable water transfer lines.
- An optimal maximum working pressure of 200 psi in the treated water pipeline, based on hydraulic evaluation (TM No. 2, *Determination of Maximum Pipe Pressure Class*).
- The minimum velocity guideline of approximately 2 feet per second (fps) required for maintaining adequate scour of potential filter sediment entering from the WTP.

Hydraulic evaluation of the delivery system capacities presented in Section 2 resulted in the hydraulic grade line (HGL) profiles shown in Figure 4-1. The system capacities include conditions expected for the base project initially in 2002, ultimately in 2012, and phased project demands once the system is expanded. The HGL profiles are indicated along with the corresponding ground elevations along the pipeline route between Canyon Lake and the furthest delivery point at San Antonio Water System (SAWS).

The resulting pipeline sizes range from 30-inch diameter for the main pipeline from the treatment plant to 14-inch diameter for the Bexar Metropolitan Water District delivery point south of Bulverde, 18-inch diameter for the SAWS delivery, and 12-inch diameter to the City of Boerne. The pipeline diameters, pipeline pressure class, and length of pipeline within each pressure class for the treated water delivery system are indicated in Table 4-1. All transfer piping required for conveyance of the base project capacity is less than the optimal maximum 200-psi pressure value. Only a limited area of piping is subject to pressures just exceeding 200 psi under the phased project capacity. The estimation for length of this limited area is conservative and will likely be reduced with detailed topographic survey in design.

The pumping requirements for the pump station at the WTP and remote pumping stations are indicated in Table 4-2 for both base project (10 MGD) and phased project (15 MGD) demands. The minimum and maximum requirement range is based on the anticipated hydraulic conditions for the system under the range of likely operating scenarios, as detailed in TM No. 4, *Pump Station Design.*

4.3 System Storage Requirements

A raw water reservoir located on Startz Hill is proposed to serve as a head tank to would provide water to the treatment plant under a relatively constant flow and pressure, thereby eliminating head and flow fluctuations at the plant's inlet and ensuring hydraulic transients from the starting and stopping of pump equipment are not transmitted to the WTP. The Startz Hill reservoir is described in more detail in the Basis of Design Report for Contract A.

Storage within the treated water delivery system should be provided both at the WTP and the remote pump station. Storage provided at the remote pump station eliminates the need for constructing the remote pump station as an in-line booster pump station, and the reservoir level provides a reliable control input for the pumps located at the remote pump station and WTP.

Figure 4-1. System Hydraulic Grade Line Profiles

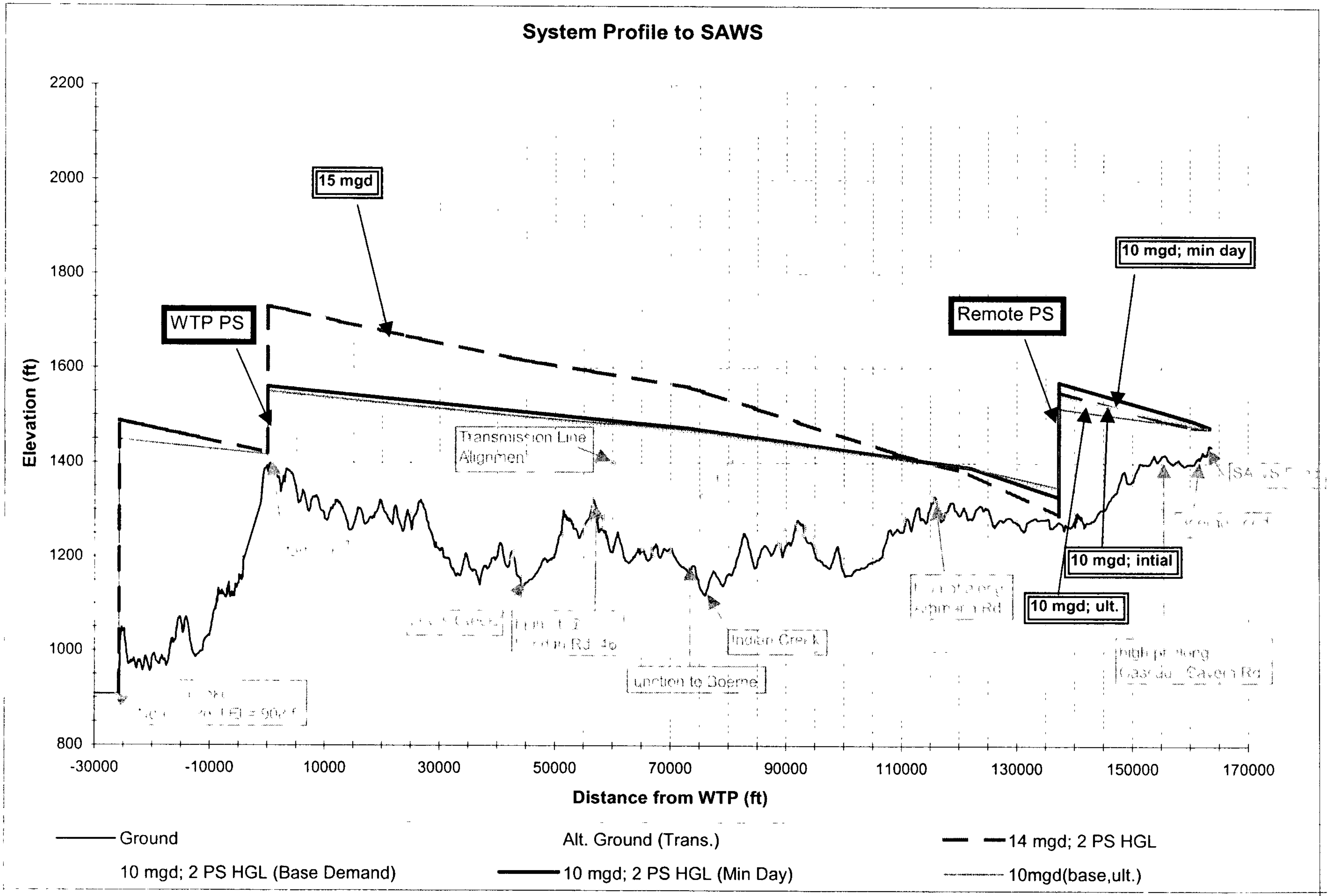

Table 4-1.
Summary of Approximate Treated Water Piping Lengths by Pipe Class

Pipe Diameter (inches)	Pressure Class (psi)	Pipe Length (feet)
30	250	2,060
30	200	68,288
30	150	2,805
24	200	11,407
24	150	7,488
24	100	29,772
20	100	27,138
18	150	11,790
14	250	4,853
14	200	22,572
14	150	3,018
12	150	22,340

Table 4-2.
Summary of Pumping Requirements

	Base Project[1]	Phased Project
Number of Pumps Operating	2	3
WTP Pump Station		
Total Flow (MGD)	9.9	15.0
Each Pump Flow (gpm)	3,430	3,470
Required Head (feet)	169	319
Remote Pump Station		
Minimum Flow (MGD)	5.3	7.9
Each Pump Flow (gpm)	1,850	1,820
Minimum Head (feet)	255	285
Maximum Flow (MGD)	6.5	7.9
Each Pump Flow (gpm)	2,270	1,820
Maximum Head (feet)	303	333

[1] Base project is referred to as 10 MGD for convenience of reference. Hydraulic calculations are based on requested delivery quantity of 9.9 MGD.

The volume of the reservoir at the WTP will depend largely on the treatment process selected (refer to the Contract A Basis of Design report for more information on the WTP reservoir). The estimated volume of the reservoir located at the remote pump station is approximately 250,000 gallons.

4.4 Surge-Transient Analysis

Transient pressures are caused by sudden changes in the velocity of the water column being conveyed by the pipeline. Causes of such sudden changes include power loss at the pump station, valve movements, controlled pump shutdown, pump failure, pump start-up, air venting from the lines, failure of flow or pressure regulators, or pipe rupture. Transients create widely varying changes in pressure, or surges, in the pipeline and could cause damage to the pipeline system via weakened pipe material, dislodged joints, ruptured pipe, or damage to pumping equipment.

Pipeline systems must be protected from the effects of transient flows by relief of the surge pressures to within acceptable pipeline pressure ratings. This relief can be accomplished with the use of surge control equipment. Design of surge control equipment requires analysis of the specific unsteady flow conditions expected to occur within the system. Modeling of the system's transient pressures will be performed once a final pipeline route is selected. It is anticipated that the surge control equipment will consist of pump control valves on the discharge of each pump as well as pressure relief and surge anticipator valves on the pump discharge header, as deemed necessary.

4.5 Treated Water Pump Stations

Two main treated water pump stations are proposed for the system to maintain a maximum pressure of 200 psi within the system and provide pumped delivery to all GBRA system customers. For construction cost savings and improved operations, one pump station is proposed at the WTP site. The other, a remote pump station, is recommended near the proposed Fair Oaks Ranch delivery point. Location of the remote pump station is based on review of the calculated system hydraulic grade line and existing topography, as detailed in TM No. 4, *Pump Station Design*. In addition to these two main treated water pump stations, a small booster pump is proposed for the system for conveyance of water supply to the Bergheim (Cordillera Ranch) area. A minimum of 10 psi, per early criteria developed in TM No. 1, is maintained within the system piping at all times.

4.5.1 Pumping Alternatives

The most significant options for design of a treated water pumping station are related to the type and required number of pumps, their relative efficiency, and their maintenance requirements. The most commonly used pumps are horizontal split-case pumps and vertical turbine pumps. The general advantages and disadvantages of each type of pump are listed below.

Horizontal Split Case (HSC) Pumps

- Advantages:
  - Pumps and motors can be removed separately for repair.
  - Pump repair can usually be performed in place without removing the entire pump.
  - Piping may be installed exposed.
- Disadvantages:
  - Pumps require more floor space and a larger building.

Vertical Turbine (VT) Pumps

- Advantages:
  - Pumps are more easily modified for future pumping conditions.
  - Small footprint, requiring less floor space.
- Disadvantages:
  - Removing a pump for repair requires more labor.
  - Suction piping is usually buried under the floor slab.

Each treated water pump station was evaluated for use of both horizontal split case and vertical turbine pumps as it relates to the layout, capital and operating cost, maintainability, and expandability of the pumping station. Pump selections for both types of pumps were made for each treated water pump station, based on developed hydraulic system curves. The system curves are based on anticipated hydraulic conditions for the pumps under the range of likely operating scenarios. The pump selection evaluation is detailed in TM No. 4, *Pump Station Design*, and only the recommendations are reiterated herein.

Based on the system curves, use of vertical turbine pumps is preferable for the pump stations. Vertical turbine pumps inherently produce a steeper pump curve, which is conducive to the larger increase in head with relatively low increase in flow per pump expected to occur within the system. In addition, the vertical turbine pumps require minimal layout area and are more easily modified to provide for future expansion of the system, requiring overall less construction costs for the required facilities. Variable frequency drives (VFDs) are recommended for both the WTP and remote pump stations, as described in detail in TM No. 4, Appendix D.

For operational flexibility, the WTP and remote pump stations will require 3 pumps initially (1 serving as standby) for the 10 MGD base project demand. The vertical turbine pumps for base project demand conditions will require 200 hp motors at the WTP and 250 hp motors at the remote pump station. Future expansion of the system capacity to 15 MGD may occur by adding stages to the existing WTP pumps and one additional pump at each pump station. The pump motors at the WTP will also need to be upgraded to 400 hp motors for the phased project expansion. For the small booster pump required to serve the Bergheim (Cordillera Ranch) area, 2 vertical turbine pumps with 30 hp motors (1 serving as standby) would be required for the initial base project capacity of 0.9 mgd. An additional pump would be required for the phased project expansion capacity of 1.5 mgd. Details for the pump station designs are further discussed in Section 5.2, Pump Station Recommendations.

4.5.2 Recommendations

Of the several alternative pumping scenarios and operating pressures investigated, the recommended delivery system uses two main treated water pump stations with a maximum pressure of 200 psi. The maximum pressure criterion is only compromised in a limited area of the system and only during phased expansion conditions to 15 MGD. One pump station should be located at the WTP, and the other is to be a remote pump station located along the route west to Boerne near the Fair Oaks Ranch delivery point. An additional 30 hp booster pump station is proposed along the 12-inch diameter pipeline to the Bergheim (Cordillera Ranch) area. Vertical turbine pumps are recommended for all pump stations based on economic, layout, and expandability considerations. The proposed piping would be capable of delivering 15 MGD under the phased expansion scenario, and expansion of the system's capacity would only require adding an additional pump at each pump station, and upgrading the pumps and motors at the WTP.