LIST OF TABLES

LIST OF FIGURES

TABLE OF CONTENTS (Continued)

Page

1.0 Introduction

1.1 Purpose

The Village of Wimberley currently is provided water through a local water service corporation and wastewater is treated through individual septic systems. The Village has applied for a Certificate of Convenience and Necessity (CCN) to serve a portion of the Village with centralized wastewater collection and treatment. The vision for the system is centralized wastewater treatment with no discharge to local streams. The purpose of this study is to determine a cost effective method to provide wastewater services initially and as the Village grows and the system expands. One primary function of the study is to provide documentation to provide engineering support to the application for the CCN. The project development will build on the "Wimberley Regional Wastewater Planning Study" completed in 1996. Recommended facility improvements are also identified for the time period commencing with the present and extending through 2020, in three major phases.

1.2 Scope

The planning period for this report extends from the present to the year 2020. Principal elements include the following:

- Review and summary of historical population and water quality data
- Review and summary of wastewater quantities
- Determination of treated effluent quality
- Determination of future population projections and waste stream capacities
- Recommendation for treatment facilities
- Recommendation for collection system
- WWTP site location evaluation
- Cost estimation for the recommended facilities
- A basic rate determination for the new facilities
- Public presentation of findings and recommendations
- Development of a report for support of the CCN application

1.3 Abbreviations

Abbreviations used in this report are as follows:

ac	Acre
avg.	Average

BOD	Biochemical Oxygen Demand (5-day)
BVSS	Biodegradable volatile suspended solids
C	Centigrade
cfm/ft	Cubic feet per minute per foot
cfs	Cubic feet per second
CFR	Code of Federal Regulations
CMAS	Complete mix activated sludge
cu ft	Cubic feet
DO	Dissolved oxygen
Eff.	Effluent
EPA	Environmental Protection Agency
ft	Feet
ft^2	Square feet
gpd	Gallons per day
gpm	Gallons per minute
hp	Horsepower
HRT	Hydraulic retention time
I/I	Infiltration/inflow
k	Thousand units
km	kilometer
lb	Pounds
lbs/day	Pounds per day
Max.	Maximum
µg/l	Micrograms per liter
MGD	Million gallons per day
mg/g	Milligrams per gram
mg/l	Milligrams per liter
MLSS	Mixed liquor suspended solids
MM	Maximum month
MPN	Most Probable Number
NH_3-N	Ammonia-nitrogen
NPDES	National Pollutant Discharge Elimination System
O&M	Operation and Maintenance
OSHA	Occupational Safety and Health Act
O_2	Oxygen
P	Phosphorus

pH	Hydrogen ion concentration
ppd	Pounds per day
pcd	Pounds per capita per day
RAS	Return Activated Sludge
SCADA	Supervisory Control and Data Acquisition
SCFM	Standard cubic feet per minute
SLR	Solids loading rate
std.	Standard
SOD	Sediment oxygen demand
SOTR	Standard oxygen transfer rate
SOUR	Specific oxygen uptake rate
SRT	Solids retention time
SWD	Sidewater depth
TDH	Total dynamic head
TSS	Total suspended solids
UBC	Uniform Building Code
USGS	United States Geological Survey
VSS	Volatile suspended solids
WAS	Waste activated sludge
WVS	Waste volatile solids
wt	Weight
WW	Wastewater
WWTP	Wastewater Treatment Plant

2.0 Summary of Findings and Recommendations

2.1 Findings

1. The current city limits of Wimberley plus a 1 mile extension defines the current Extra Territorial Jurisdiction (ETJ) and the limits of the proposed Certificate for Convenience and Necessity (CCN) for the new wastewater sewer collection and treatment system. Wimberley's proposed CCN excludes Woodcreek's ETJ and CCNs held by Aquasource and Blue Hole. However, Blue Hole and Wimberley have entered into a Memorandum of Understanding wherein the Blue Hole will be included within Wimberley's CCN.

2. The historical growth in the County, surrounding cities, and Wimberley has been significant in recent years with growth rates up to 4.7 percent per year. Investigations of multiple population studies indicate expected growth rates from 1.5 percent to 4 percent.

3. Testing on selected septic tanks has indicated that failed septic systems can have a direct impact on stream water quality. Water quality testing on Cypress Creek shows a general trend of increasing coliform measurements over the last 18 years.

4. Regulations on the water quality in Texas and the Wimberley area have changed since the Wimberley Regional Wastewater Planning Study was completed in 1996. Cypress Creek is currently listed as an impaired water body on the Texas Clean Water Act Section 303(d) List and a Total Maximum Daily Load study is being conducted by Texas A&M University Kingsville. Stringent wastewater effluent discharge limits on the Blanco River upstream of the Edwards Aquifer Recharge Zone have been established.

5. Water supply and water quality are issues critical to the residents and economy of Wimberley. A water supply masterplan should be completed for the Wimberley area to identify a viable long-term water supply.

6. Wastewater quality and quantities were collected and summarized. Good correlation exists between local WWTPs, TNRCC standards and national standards.

7. Collection system costs to serve every resident in the CCN in the near future are cost prohibitive. Therefore, areas of dense development close to the streams were selected for the implementation of an initial sewage collection and treatment system.

2.2 Recommendations

1. An estimated annual growth rate of 3.0 percent is recommended for wastewater system planning through the year 2020. With an initial population inside the city limits of 5,125 and a population outside the city limits but inside of the CCN boundaries of 2,074, the estimated population for planning is summarized as follows:

Table 2-1 Population Projections	
Year	Wimberley CCN
2000	7,199
2005	8,346
2010	9,675
2015	11,216
2020	13,002

2. Wastewater improvements should be provided in three phases. Phase 1 includes the down town area, the Blue Hole Development and the Lumberyard. Phase 2 includes the remainder of the densely populated areas plus an allowance for growth until 2010. And the ultimate plant buildout will include growth through 2020. Plant size recommended is shown on the following table:

Table 2-2 Wimberley Project Design Flows			
Service Area	Annual Average Flow (gpd)	Maximum Month Flow (gpd)	Peak 2-Hour Flow (gpd)
Phase 1	235,300	305,900	611,800
Phase 2	1,175,785	1,528,520	3,057,041
Ultimate	1,545,726	2,009,444	4,018,887

Plant capacity will be the maximum month flow for each phase.

3. A decentralized system is not recommended due to extensive operational costs, permitting requirements, discharge controls and operational complexity.

4. Eight treatment alternatives for primary treatment were investigated for the WWTP liquid phase. The recommended treatment is either conventional activated sludge or a biological

membrane reactor. Final process selection to be made with input from operations staff. Solids treatment recommendation is aerobic digestion with drying beds. Effluent disposal will be land application and crop growth.

5. Capital costs are summarized in the following table:

Table 2-3

Capital Costs for Phases 1 and 2

Phase 1	
Treatment	$1,160,300
Collection System	$1,712,000
Pump Station	$60,000
Force Main	$110,000
Engineering, Legal, Administrative (15%)	$456,345
Contingency (10%)	$349,865
	$3,848,510
Phase 2	
Treatment	$5,807,700
Collection System	$14,393,000
Pump Station (3)	$180,000
Force Main	$220,000
Engineering, Legal, Administrative (15%)	$3,090,105
Contingency (10%)	$2,369,081
	$26,059,886
Total cost	$29,908,396

6. WWTP sites are limited by available size, topography, location of flood planes, location of archaeological resources, proximity to the Edward Aquifer Recharge Zone, and Geology. Three sites are identified for further investigation during final design.

7. Residential monthly bills are estimated to be $65 for Phase 1 and $68 for Phase 2 if capital costs are bond financed without additional stating capital. Commercial monthly bills will be approximately twice those amounts. Providing starting capital through low or not interest loans, fees, taxes and grants would reduce the monthly bills. As an example, the implementation of a connection fee of $2,700 for Phase 1 and $3,000 for Phase 2 would reduce residential monthly bills to $50.

3.0 Service Area Definition and Population Growth

3.1 Overview

This section defines the area that will be provided with wastewater collection and treatment services and the population projections for the defined area.

3.2 Determination of Service Area

The service area, henceforth known as the Wimberley Certificate of Convenience and Necessity (CCN), includes the corporate limits of the Village of Wimberley plus an additional one-mile Extra Territorial Jurisdiction (ETJ) outside the current limits. The Wimberley CCN is shown on Figure 3-1. The abbreviation "CCN" shall be used in this report to mean the intended service area and current ETJ. The CCN does not extend into the current ETJ of nearby Woodcreek, and extends only to the boundaries of Aquasource's CCNs. Based on a current Memorandum of Understanding, the Wimberley CCN includes the Blue Hole CCN in its service area.

3.3 Projected Population

The population projections used for this study are based on the review of population data from a variety of sources. This study encompasses a 20-year planning horizon.

3.3.1 Data Sources

The following sources were reviewed to project the future population for the Wimberley CCN.

3.3.1.1 Texas State Data Center (TXSDC). The Texas State Data Center provides population breakdowns for the 2000 Census by county, tract, and block. The center also provides population projections for each county in Texas up to the year 2040. Projections are made based on three different scenarios. These scenarios are [1] the zero migration (0.0) scenario, [2] the one-half 1990-2000 (0.5) scenario, and [3] the 1990-2000 scenario (1.0). The 0.0 scenario assumes that the net migration is zero, so any change in population is based on the differences in births and deaths in the county. The 0.5 scenario assumes a net migration rate of one-half of that experienced in the 1990s. The 1.0 scenario assumes that the net migration in the future will be equal to that experienced in the 1990s. The Texas State Data Center suggests that the 0.5 scenario is the most appropriate scenario to use for planning purposes.

3.3.1.2 Texas Water Development Board (TWDB). The Texas Water Development Board provides population projections up to the year 2050 by county and by city and Census Designated Place (CDP). The U.S. Census Bureau defines a CDP as a statistical area that is a densely settled concentration of population that is not incorporated but which resembles an incorporated place in that it can be identified with a name. The most recent projections from the Texas Water Development Board are still based on the 1990 Census and have not been updated since the 2000 Census.

3.3.1.3 U.S. Census 1990 and 2000. Census data for Hays County are available back to the 1960 Census. Census data for 1990 and 2000 are available for the Wimberley CDP. There is no census data for the Wimberley CDP prior to the 1990 Census. The Wimberley CDP represents an area that is smaller than that of the Wimberley CCN.

3.3.1.4 Wimberley Independent School District. Wimberley ISD had DeskMap Systems, Inc. prepared the Wimberley ISD Demographic Study in January of 1999. Wimberley ISD represents an area larger than that of the Wimberley CCN. Population projections for Wimberley ISD were estimated up to the year 2007 using the Cohort-Survival Method, which is a forecasting tool used by school districts to determine future student enrollments. Wimberley ISD is currently experiencing an annual growth rate of approximately 3%.

3.3.1.5 Wimberley Regional Wastewater Planning Study. This study was completed in March of 1996. It includes population projections for a service area that is larger than the Wimberley CCN. This study utilized the TWDB population projections of Buda, Kyle, and Dripping Springs because at the time Wimberley was not incorporated and there were no TWDB projections available. These three cities were thought to be representative of growth patterns similar to Wimberley. This study assumed an annual growth rate of 4% for the entire service area.

3.3.1.6 Wimberley Water Supply Corporation. The Wimberley Water Supply Corporation indicated that they have 1,254 meters within the city limits and 389 meters outside the city limits, but within the CCN. They do not have any population data or projections available. However, water pumpage records from 1992 to 2001 are available and are useful as an indicator of growth. The average annual increase in pumpage from 1992 to 2001 is 3.32%. This data is summarized in Table 3-1.

Table 3-1 Wimberley Water Supply Corporation Yearly Water Pumpage	
Year	**Water Pumpage**
1992	161,100,000
1993	162,757,000
1994	165,847,000
1995	174,469,000
1996	187,741,000
1997	170,456,000
1998	204,665,000
1999	218,852,000
2000	212,164,000
2001	216,152,000

The population projections from the sources discussed in this section are summarized in Table 3-2.

Table 3-2
Population Data Summary

Year	Hays County (TXSDC)	Avg. Annual Growth (%)	Wimberley CDP (TXWDB)	Avg. Annual Growth (%)	Wimberley CDP (Census)	Avg. Annual Growth (%)	Wimberley ISD Demographic Study (1999)	Avg. Annual Growth (%)	Wimberley Regional Wastewater Plan (1996)	Avg. Annual Growth (%)
1980	40,594	----								
1990	65,614	4.92	2,403	----	2,403	----	6,324	----		
1995									6,012	----
1997							8,077	3.56		
2000	97,589	4.05	3,325	3.30	3,797	4.68				
2002							9,334	3.11		
2005	115,595	3.44								
2007							10,591	2.56		
2010	135,450	3.22	4,301	2.61						
2015	157,115	3.01							13,173	4.00
2020	178,784	2.62	5,001	1.52						

3.3.2 Historical Population

As stated earlier, population data for the Wimberley CDP prior to 1990 is unavailable and the Wimberley CCN represents an area larger than the Wimberley CDP. The 2000 estimated population for the Wimberley CCN was determined by assuming a base population of 5,125 within the city limits, and then adding the estimated population inside of the Wimberley CCN, but outside the city limits.

The base population within the city limits was estimated n 2000 by a committee of local residents who were advocating incorporation of the village. Dr. Sally Caldwell, local resident and faculty member from Southwest Texas State Univeristy, reported that the estimate was based upon a variety of data sources, including:

- Data collected in a previous incorporation effort (1997)
- Information from the post office
- Information from PEC
- School district enrollment data
- Central Appraisal District

Additionally, a substantial amount of ground truthing was undertaken in an effort to verify information developed from other sources.

The population inside the Wimberley CCN, but outside the city limits was determined by identifying the percentage of each census block located within the Wimberley CCN, but outside the city limits as shown in Figure 3-1. These percentages were then applied to the total census block population. The populations for all of the census blocks were then added together to determine the total estimated population for the Wimberley CCN outside of the city limits. A tabulation of the 2000 population for the census blocks within the Wimberley CCN, but outside the city limits is shown in Table 3-3.

TABLE 3-3 Year 2000 Population for Wimberley CCN Outside of City Limits				
Tract	Block	2000 Census Population	% of Block	2000 Population for CCN Outside of City Limits
107	2010	0	20%	0
107	2011	0	50%	0
107	2012	0	5%	0
108.02	1030	200	60%	120
108.02	1033	51	40%	20
108.02	1034	25	10%	3
108.02	1037	115	50%	58
108.02	1041	16	20%	3
108.02	1042	10	100%	10
108.02	1043	5	100%	5
108.02	1044	5	100%	5
108.02	1045	7	100%	7
108.02	4018	39	50%	20
108.02	4026	0	100%	0
108.02	4044	117	45%	53
108.02	4046	10	50%	5
108.02	4048	6	33%	2
108.02	4049	30	50%	15
108.02	4056	14	50%	7
108.02	4063	39	100%	39
108.02	6000	175	80%	140
108.02	6001	240	80%	192
108.02	6004	3	100%	3
108.02	6005	1	100%	1
108.02	6006	0	100%	0
108.02	6007	4	100%	4
108.02	6008	23	50%	12
108.02	6009	0	100%	0
108.02	6010	0	100%	0
108.02	6011	0	100%	0
108.02	6012	8	100%	8
108.02	6013	215	25%	54
108.02	6029	163	50%	82
108.02	6030	10	100%	10
108.02	6031	21	100%	21
108.02	6032	7	100%	7
108.02	6033	14	100%	14
108.02	6034	2	100%	2
108.02	6035	6	100%	6
108.02	6036	50	100%	50
108.02	6037	41	5%	2
108.02	6038	8	100%	8

TABLE 3-3 Year 2000 Population for Wimberley CCN Outside of City Limits				
Tract	**Block**	**2000 Census Population**	**% of Block**	**2000 Population for CCN Outside of City Limits**
108.02	6039	38	30%	11
108.02	6041	60	60%	36
108.02	6042	13	100%	13
108.02	6045	16	15%	2
108.02	7009	65	75%	49
108.02	7010	59	60%	35
108.02	7011	267	50%	134
108.02	7013	76	100%	76
108.02	7014	62	100%	62
108.02	7015	55	40%	22
108.02	7034	146	40%	58
108.02	7040	56	90%	50
108.02	7041	189	80%	151
108.02	7042	18	100%	18
108.02	7043	25	100%	25
108.02	7044	129	100%	129
108.02	7045	98	100%	98
108.02	7046	29	30%	9
108.02	7056	141	50%	71
108.02	7059	11	100%	11
108.02	7060	0	100%	0
109.04	3037	147	5%	7
109.04	3040	38	50%	19
109.04	3042	0	100%	0
109.04	3043	0	100%	0
TOTAL				2,074

As shown in Table 3-3, the total population within the Wimberley CCN outside the city limits in the year 2000 is 2,074. The total estimated population for the year 2000 is 7,199. This population will be used as the base year for all projections for years 2005, 2010, 2015, and 2020.

3.3.3 Projected Population

The most reliable source of data for future population data are the projections made by the Texas State Data Center (TXSDC). The population projections for the next twenty years for Hays County show a 3.1% average annual rate of growth. This growth rate is recommended by TXSDC, and is consistent with growth projections estimated by other independent sources. For purposes of this study, an average annual growth rate of 3% will be assumed for the Wimberley CCN. The population projections produced by using this approach are summarized in Table 3-4.

TABLE 3-4
Population Projections for Wimberley CCN

Year	Hays County (TXSDC)	Wimberley CCN
2000	97,589	7,199
2005	115,595	8,346
2010	135,450	9,675
2015	157,115	11,216
2020	178,784	13,002

OVERSIZED MAP(S)

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map Figure 3-1

4.0 Water Quality

4.1 Overview

The purpose of this section is to summarize existing surface water quality data, water quality regulations and activities which may impact water quality in the future. Cypress Creek and the Blanco River in Wimberley represent quality of life and livelihood for the residents of the Village. The Village is concerned about impacts on stream water quality resulting from development and continued use of septic tanks. In response they have enacted development regulations and is pursuing centralized wastewater treatment facilities in an effort to minimize impacts on the rivers and streams.

4.2 Septic Tank Testing

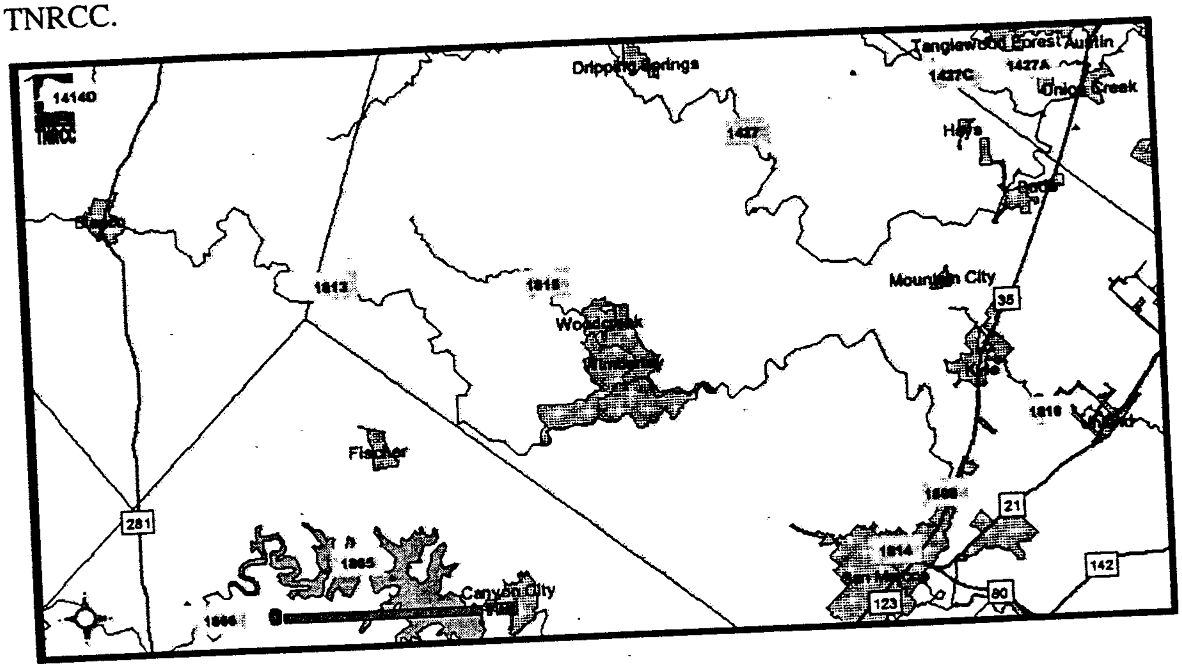
In 1983 it was believed that some septic tanks were not performing to standards and were impacting Cypress Creek. An informal dye test was completed on a number of septic systems in and around the downtown square area and other properties bordering the creek. Jean Williams, a resident of the Wimberley area and other volunteers flushed red dye down selected toilets in the area. The dye showed up below Sheffield's log cabins and west of the square. The Hays County Environmental Health Department was notified of the issue and appropriate septic tank systems were expanded or modified to address the issue.

The impact from septic tanks continues to be a concern for the quality of water in Cypress Creek. The previous testing demonstrates the potential for overloaded septic tanks to have a direct and identifiable impact on the local streams.

4.3 Texas Clean Water Act Section 303(d) List

Section 305(b) of the federal Clean Water Act (CWA) requires states to produce a periodic inventory comparing water quality conditions to established standards (Surface Water Quality Standards, 30 Tex. Admin. Code (TAC) Section 307, and Drinking Water Standards, 30 TAC Sections 290.101-121). The 305(b) Water Quality Inventory is an overview of the status of surface waters of the state, including concerns for public health, fitness for use by aquatic species and other wildlife, and specific pollutants and their possible sources. Section 303(d) of the CWA requires the state to develop a list of water bodies that do not meet established standards. These are referred to as "impaired waters." The state must take appropriate action to improve impaired water bodies, such as development of Total Maximum Daily Loads (TMDL). This program was not in place when the 1996 regional planning study was completed.

The 2000 and 2002 inventory both include Cypress Creek, Segment ID 1815 in the list of impaired waters. Currently, the reaches of the Blanco River upstream (Segment ID 1813) and downstream (Segment 1809) of Wimberley are not listed as being impaired by TNRCC.



The water quality goals for each of the segments is summarized as follows:

Table 4-1 Stream Water Quality Standards			
	Segment 1809 Lower Blanco	Segment 1813 Upper Blanco	Segment 1815 Cypress Creek
Chloride, mg/l	50	50	50
Sulfate, mg/l	50	50	50
Dissolved Solids, Min, mg/l	400	400	400
Dissolved Oxygen, mg/l	5	6	6
Min pH	6.5	6.5	6.5
Max pH	9.0	9.0	9.0
E.coli	126	126	126
Fecal Coliform	200	200	200
Temperature, °F	92	92	86

Using historical data, TNRCC assessed the stream segments in the Guadalupe River Basin and included Segment 1815 on the Water Quality Inventory and List of Impaired Waters. TNRCC has included Segment 1815 in a TMDL project being conducted by Texas A&M Kingsville. The designation "Impairment" causes a segment to be listed as the result of both point source and non-point sources. The Segment Summary on 303(d) list states for Segment 1815 that "Dissolved oxygen concentrations are occasionally lower than the criterion established to assure optimum conditions for aquatic life. Bacteria levels are sometimes higher than the criterion established to assure the safety of contact recreation." Water quality testing that has been conducted on this segment is summarized in Table 4.2.

The TNRCC is attempting to address these issues through the Total Maximum Daily Load (TMDL) Program. The objective of the TMDL Program is to restore and maintain the beneficial uses of impaired or threatened water bodies in Texas. The program is authorized by and created to fulfill the requirements of the federal Clean Water Act. At this time there are only 11 stream segments that have an established TMDL implementation plan.

Table 4.2

TNRCC Segment 1803
TNRCC Station 12674
Station Number 22 Cypress Creek at FM 12 in Wimberley
Latitude 29/59/46 Longitude 98/05/48

Parameter	Parameter Code	Date and 24 hour time												
		3/17/98 1338	6/11/98 1112	9/16/98 1142	12/7/98 1220	4/13/99 1023	6/14/99 1244	9/14/99 1130	12/13/99 950	3/21/00 931	7/18/00 951	10/31/00 1003	1/22/01 951	3/19/01 1108
Flow (cfs)			9.21	12.3	27.7	7.72	3.89	0.96	2.54	3.87	0.33	1.94	43.9	20
Fecal Coliform(org/100mL)	31616	275	1225	662	66	64	336	400	232	60	70	96	76	20
Suspended Solids(mg/L)	530	3.4	4	3.2	1.6	<1.0	<1.0	1.7	<1.0	1.2	1.6	<1	1.8	1
Turbidity(NTU)	82079	4.1	3.6	3.3	1.4	2.1	0.8	2.1	0.85	1	1.4	1.5	1	2.2
pH	400	8.26		7.66	7.53	8.09	7.78	7.99	9.01	8.01	7.54	7.4	7.89	8.02
Temperature(C)	10	17 95	24.63	24.36	19.48	21.18	24.53	24.71	13.51	17.17	25.77	22.08	12.8	15.16
Dissolved Oxygen(mg/L)	300	10.19	7.22	6.98	9.16	8.16	8.11	8.59	7.68	8.77	6.8	7.52	11.28	11.3
Conductivity(umhos/cm)	94	495	512	514	545	534	507	515	534	549	563	586	570	559
Total Phosphorus(mg/L)	665	0.07	0.15	<0.01	<0.01	<0.01	0.2	<0.01	0.07	0.054	0.01	0.2	0.02	0.07
Nitrate-N(mg/L)	620	0.28	0.17	0.19	0.24	<0.02	<0.02	<0.02	0.052	<0.02	0.07	<0.02	0.09	0.31
Chloride(mg/L)	940	17.7	13	15.6	13.8	17.4	16.6	16.4	16.2	17.2	21.2	22.6	18.7	15.2
Sulfate(mg/L)	945	10	16.2	16.4	8.8	17.6	17.1	15.1	16.9	19.1	20.4	25.2	18.8	16.2
Total Hardness(mg/L)	900	256	229	228	144	270	290	225	268	191	266	293	286	265
Ammonia-N(mg/L)	610	0.05	0.1	0.09	0.05	0.07	0.05	0.92	0.11	0.11	0.11	<0.02	0.09	<0.02
E. coli(org/100mL)	31648	250	1175	112	60	18	200	88	2332	60	60	96	54	70
Chlorophyll a(mg/m^3)	32211	<1.0	1.1	<1.0	<1.0	<1.0	<1.0	<1.0	<1.0	<1.0	1.1	<1	<1	<1

4.4 Other Water Quality Sampling

Water quality and quantity sampling is routinely sampled and recorded at various points on Cypress Creek and the Blanco River by Guadalupe Blanco River Authority, TNRCC, the United State Geological Survey (USGS), and the Wimberley Valley Watershed Association (WVWA). Some of the data has been presented in the section above. Much of the data pertains to general information such as flow, level, and temperature. WVWA has sampled for fecal coliform since 1984. There are ten sampling points total, with 5 each on Cypress Creek and the Blanco River. The yearly mean generally shows that the fecal coliform measured in the stream is generally increasing. The numbers range from near zero to over 1000. These measurements are taken monthly for approximately 9 months of the year. The readings show general trends, but they are affected greatly by seasonal land use, stream flow and rainfall.

4.5 TNRCC Discharge Limits

TNRCC Chapter 213, Subchapter A was developed and made effective in 1999 to regulate activities having the potential for polluting the Edwards Aquifer and hydrologically connected surface streams in order to protect existing and potential uses of groundwater and maintain Texas Surface Water Quality Standards. The Edwards Aquifer extends into the southern and southeastern edges of the Village of Wimberley Extra Territorial Jurisdiction (ETJ) and across approximately one-half mile of the southern portion of the city limits. The Edwards Aquifer crosses the Blanco River just outside of the eastern edge of the ETJ as shown on Figure 4-1. The applicable requirements of Chapter 213, Subchapter "A" are as follows:

1. Any construction of any kind that is on the recharge zone must have an approved Water Pollution Abatement Plan. (213.5 (a) (1))
2. Sewer collection systems built in the recharge zone must meet the requirements of Chapter 317, and special requirements of 213.5 (c)
3. Land application of wastewater through evaporation or irrigation must meet requirements of Chapter 309 and will be approved on a case by case basis. (213.6 (b))
4. Discharge within the first five miles upstream from the recharge zone must achieve the following effluent treatment:
   - Five milligrams per liter of carbonaceous biochemical oxygen demand, based on 30-day average.
   - Five milligrams per liter of total suspended solids, based on a 30-day average.
   - Two milligrams per liter of ammonia nitrogen, based on a 30-day average
   - One milligram per liter of phosphorus, based on a 30-day average
5. Any discharge more than 5 miles but less than 10 miles upstream of the recharge zone must, as a minimum, achieve a level of effluent treatment for system 2N based on a 30-day average as set out in Table 1 of Chapter 309, but more stringent treatment and monitoring may be required on a case-by-case basis.

4.6 Water Reuse and Disposal

Reuse water quality standards may impact the treatment requirements for a new wastewater treatment facility. The goal of the treatment facility being considered by the Village is zero discharge into the rivers and streams in the area. Water will be disposed of through irrigation, reuse, evaporation, or percolation. The water quality requirements for these disposal options will be summarized and discussed in more detail in the Wastewater Treatment section of this report.

4.7 Groundwater Quality

Groundwater is the source of drinking water for the Village of Wimberley and surrounding area. Most of the ETJ lies within the Trinity Aquifer. Through population growth and subsequent pumping, the aquifer levels have dropped as much as 550 feet. Jacob's Well, a large artesian spring which feeds Cypress Creek ceased to flow in 2000, the first time in recorded history. The need for water supply for the area, the reduction in aquifer level, water quality, and stream flow are all important issues that must be addressed for Wimberley and the surrounding area. These tasks have been addressed in part by the *Hydrogeologic Assessment of the Trinity Aquifer in Parts of Comal and Hays Counties*.

OVERSIZED MAP(S)

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map Figure 4-1

5.0 Wastewater Quality and Quantity

5.1 Overview

In order to correctly size the wastewater treatment system for Wimberley it is necessary to estimate both the quantity of wastewater to be treated and the likely strength of the wastewater. A number of references and guidelines are available to enable an accurate estimate to be made of probable wastewater quantities and quality, including the TNRCC Rules Chapter 317, the Ten State standards, and the Wastewater Engineering textbook distributed by Metcalf & Eddy Inc. In addition to these references, data from three wastewater facilities located near Wimberley were analyzed to determine the parameters appropriate for use in this area. Flow and raw influent concentration data were obtained from the San Marcos wastewater treatment plant, the Springs Hill plant and the North Cliff facilities. This information was used to select suitable flows and concentrations for the raw wastewater expected at Wimberley. Development of flows and loads for the different scenarios was based on the approach used by GBRA in the Wimberley Regional Wastewater Planning Study (March 1996).

Two scenarios were considered:
Phase 1 – Initial wastewater treatment facility for the downtown area, including the Lumber Yard development and the Blue Hole Development.
Phase 2 – Final wastewater treatment facility covering all of the CCN for 2010.
Ultimate – Final wastewater treatment facility designed to 2020 projections.

5.2 Wastewater Quantities

Wastewater flows for residential communities are often estimated by assigning an average per capita flow rate and then multiplying this by the estimated population. Flows from commercial sources and other known wastewater flows, such as schools and retirement homes, are then added to the residential flow to give the total expected flow. The Wimberley Regional Wastewater Planning Study included a detailed analysis of the different wastewater sources in the Wimberley area.

The Wimberley Regional Wastewater Planning Study used a per capita wastewater flow of 100 gpcd. This is the value recommended by Chapter 317 of the TNRCC Rules and by the Ten States Standard. The Wastewater Engineering textbook suggests that a per capita flow between 40 and 130 gpcd may be appropriate. These values include an allowance for some Infiltration and Inflow (I&I) in the sewer system.

Analysis of the San Marcos flow data from 1980 to 1993 showed that their per capita flow rate varied between 94 and 138 gpcd over that period and had an overall average of 113 gpcd. It is noted that San Marcos had a significant flow contribution from I&I over this period. This explains why the per capita flow was slightly higher than the value recommended by TNRCC and Ten State Standards.

Based on the recommendations of the references and based on Black & Veatch's own experience, a value of 100 gpcd was used to develop design flows. This is the same flow value used in the 1996 Wimberley Regional Wastewater Planning Study.

Table 5-1 shows the projected flows for the different proposed phases in the Wimberley treatment facility design. Residential flows for phase 1 are based on the number of sewer connections counted for the downtown area and the expected connections in the Blue Hole development. Residential flows for phase 2 and the ultimate design are based on population projections for the Wimberley CCN for 2010 and 2020 respectively. The commercial flow data is based on the information provided in the 1996 Planning Study, with the exception of the flow data for the "Lumber Yard" development, which was provided separately.

Table 5-1
Elements for Flow Calculations

Service Area	Connections	Population	Flow (gpd)
Phase 1			
Initial Residential	160	480	48,000
Initial Commercial			24,000
Nursing Home			50,000
Blue Hole Residential	206	618	61,800
Lumber Yard			1,500
Phase 2			
2010 Total CCN, Residential	3,225	9,675	967,500
2010 Total CCN Commercial*			158,285
Ultimate			
Ultimate CCN Residential (2020)	4,334	13,002	1,300,200
Ultimate CCN Commercial (2020)*			195,526
Seasonal Summer Load Increase			50,000

Assumptions:

Per Capita Flow	*100*
Capita Per Connection	*3*
Annual Residential Growth	*3.0%*
Annual Commercial Growth	*3.0%*

**Commercial Flows based on 1995 total commercial flow 69,504 gpd*

A seasonal load increase has been included as a separate entry as a provision for the high influx of visitors to the area during the summer months.

Based on these individual flows, the annual average (AA) flow was calculated for each scenario. The Plant designs are usually based on a maximum month (MM) flow. The ratio of MM/AA is typically 1.1 to 1.5, depending on the amount of I&I in the sewer system. At San Marcos the MM/AA ratio was 1.29 which includes some I&I. Based on the assumption that Wimberley may have a similar degree of I&I, a MM/AA of 1.3 was used for design. The peak 2-hour flow was assumed to be 2.0 times the maximum month flow.

Table 5-2 summarizes the calculated design flows for the three treatment plant phases.

Table 5-2
Wimberley Project Design Flows

Service Area	Annual Average Flow (gpd)	Maximum Month Flow (gpd)	Peak 2-Hour Flow (gpd)
Phase 1	235,300	305,900	611,800
Phase 2	1,175,785	1,528,520	3,057,041
Ultimate	1,545,726	2,009,444	4,018,887

5.3 Wastewater Quality

Two approaches can be used to calculate the estimated wastewater quality and the design loadings on the treatment plant. The water quality can be expressed as a concentration that is then multiplied by the design flow to give the design loading. Per capita loading can also be specified. This value is then multiplied by the projected population to give the design loading. In the following discussion, some references express estimated values in concentration units and some use per capita loadings. The calculations in this report are based on assumed concentrations based on recommended typical design values. These estimates are then checked against data obtained from local wastewater treatment facilities.

The Ten State Standards suggests that a typical domestic wastewater has a BOD concentration of 203 to 240 mg/L and a TSS concentration of 240 to 276 mg/L. Wastewater Engineering suggests a wider range of influent waste concentrations at 110 to

400 mg/L for BOD, 100 to 350 mg/L for TSS and 12 to 50 mg/L for ammonia. The TNRCC Rules Chapter 317 suggests an influent BOD concentration of 200 mg/L for residential areas and 300 mg/L for commercial areas, schools and nursing homes.

Table 5-3 summarizes the data obtained from three local wastewater treatment facilities.

Table 5-3
Local Raw Wastewater Concentrations

	Springs Hill	North Cliff	San Marcos
BOD (mg/L)			
AA	218	258	170
MM	290	373	269
MM/AA	1.33	1.44	1.58
TSS (mg/L)			
AA	151	213	184
MM	248	381	254
MM/AA	1.64	1.79	1.38
Ammonia (mg/L)			
AA	n/a	n/a	16.0
MM	n/a	n/a	21.0
MM/AA			1.31

The loads for the Wimberley Plant and the raw influent concentrations were calculated by assuming different concentrations for the residential areas than for the commercial areas. The residential wastewater quality was based on a BOD concentration of 220mg/l, TSS of 230mg/L, ammonia of 21mg/L and a total-phosphorus of 8mg/L. The commercial wastewater strength was based on a BOD of 300mg/L, TSS of 350mg/L, ammonia of 25mg/L, and Total-phosphorus of 10mg/L. The wastewater strength for the Lumber Yard waste is based on a BOD of 1500mg/L. The ratio of ammonia to Total Kjeldahl Nitrogen (TKN) is assumed 0.7.

Using these values and the flows presented in Table 5-1, the annual average loadings were calculated. MM/AA ratios of 1.4 for BOD and TSS, 1.3 for ammonia and 1.2 for phosphorus were applied to give the design maximum month loadings. The design loadings for designing the Wimberley WWTP are summarized in Table 5-4.

Table 5-4 Wimberley Design Loadings					
	BOD Load (ppd)	TSS Load (ppd)	Ammonia Load (ppd)	TKN Load (ppd)	Total-P Load (ppd)
Phase 1					
AA	530	594	46.6	66.6	18.3
MM	743	832	60.6	86.6	22.0
Phase 2					
AA	2,296	2,464	212.9	304.1	81.9
MM	3,215	3,449	276.7	395.3	98.3
Ultimate					
AA	3,000	3,211	278.9	398.4	107.2
MM	4,200	4,495	362.6	518.0	128.7

The loadings presented in Table 5-4 were used along with the projected design annual average and maximum month flows to derive the influent waste concentrations, presented in Table 5-5. The maximum month concentration were calculated from the maximum month load and maximum month flow.

Table 5-5 Wimberley Design Raw Influent Concentrations					
	BOD Concentration (mg/L)	TSS Concentration (mg/L)	Ammonia Concentration (mg/L)	TKN Concentration (mg/L)	Total-P Concentration (mg/L)
Phase 1					
AA	270	303	23.8	34.0	9.3
MM	291	326	23.8	34.0	8.6
Phase 2					
AA	234	251	21.7	31.0	8.4
MM	252	271	21.7	31.0	7.7
Ultimate					
AA	233	249	21.6	30.9	8.3
MM	251	268	21.6	30.9	7.7

6.0 Wastewater Treatment Selection

6.1 Overview

A preliminary screening of wastewater treatment technologies was conducted to identify viable treatment alternatives for the proposed Wimberley, Texas wastewater treatment plant. The goal of the preliminary screening was to eliminate technologies that did not meet the goals and objectives established by Wimberley, resulting in a reduced number of alternatives that would need to be examined.

To meet the reuse treatment objectives, the plant at a minimum will have to consist of preliminary treatment, secondary biological treatment, tertiary filtration, and disinfection. Primary treatment would be optional to reduce the load on the more expensive secondary process. Included as part of the prescreening process was the development of a subjective comparison of O&M costs and a comparison of non-economic factors. The O&M cost comparison was subjective based on a weighted relative scale; therefore, the costs were not quantified and discretion must be used when drawing conclusions from this information. While O&M costs for this comparison were used in this manner, it has the same effect on the screening analysis as detailed costs would have and further has no effect on actual O&M costs which will be used for final alternative comparison in later chapters.

The results of the preliminary screening process are summarized in this chapter.

6.2 Background

It is proposed that the wastewater treatment facility at Wimberley should have zero discharge, meaning no discharge to the receiving waters of the State of Texas. The water produced by the plant will be reclaimed and used for direct irrigation, gray water reuse, or disposal via land application at a Village owned facility.

The TNRCC regulations, Chapter 210.33 for Type I reclaimed waters requires the water to meet the following 30-day average criteria:

BOD	5 mg/L
Turbidity	3 NTU
Fecal Coliform	20 CFU/100mL (geometric mean)
Max Fecal Coliform	75 CFU/100mL (single grab)

Type I reclaimed water can be used for a wide range of applications, which are listed in the TNRCC rules, Chapter 210.32 as follows:

> *"(1) Type I Reclaimed Water Use. This type of use includes irrigation or other uses in areas*
> *where the public may be present during the time when irrigation takes place or other uses where the public may come in contact with the reclaimed water. The following types of uses would be considered Type I uses:*
> *(A) Residential irrigation, including landscape irrigation at individual homes.*
> *(B) Urban uses, including irrigation of public parks, golf courses with unrestricted public access, school yards, or athletic fields.*
> *(C) Use of reclaimed water for fire protection, either in internal sprinkler systems or external fire hydrants.*
> *(D) Irrigation of food crops where the applied reclaimed water may have direct contact with the edible part of the crop, unless the food crop undergoes a pasteurization process.*
> *(E) Irrigation of pastures for milking animals.*
> *(F) Maintenance of impoundments or natural water bodies where recreational activities, such as wading or fishing, are anticipated even though the water body was not specifically designed for such a use.*
> *(G) Toilet or urinal flush water.*
> *(H) Other similar activities where the potential for unintentional human exposure may occur"*

With land application as the anticipated primary disposal option, it will be necessary for the treatment process to provide nitrification and denitrification to ensure that the nitrogen load does not limit the application rate. Therefore, this evaluation requires the secondary treatment process to be designed not only for the removal of soluble organics, and suspended solids but also for total-nitrogen. For the purpose of this study, the target process effluent total-nitrogen concentration has been set at 10 mg/L. This concentration should be re-evaluated at the design stage.

6.2.1 Centralized Treatment

Currently wastewater treatment in the Village of Wimberley consists of individual septic tanks and small treatment systems at each residence and commercial unit. This approach to wastewater management is termed a "decentralized system". While a decentralized system may be acceptable for remote residences and very small communities, it is inappropriate for communities that have a significant population density and for areas where there are concerns about the quality of local streams and water courses. The population of Wimberley is now sufficiently large to give concern over the use of decentralized treatment.

The EPA publication " DRAFT - EPA Guidelines for Management of Onsite/Decentralized Wastewater Systems September 26, 2000" states:

> *"**The performance of onsite/decentralized wastewater systems is a national issue of great concern to EPA**...Unfortunately, many of the systems currently in use do not provide the level of treatment necessary to adequately protect public health and surface and ground water quality. More than half of the existing systems are over 30 years old, and homeowners indicate that at least 10 percent of all systems are not working at all at any given time. Other data has shown that at least 25 percent of systems are malfunctioning to some degree. In a majority of cases, the homeowner is not aware of a system failure until it backs up in the home or breaks out on the ground surface. In many areas of the country, the local authority lacks records of all the systems within the service area. State agencies report that septic systems constitute the third most common source of ground water contamination and that these systems have failed because of inappropriate siting or design or inadequate long-term maintenance."*

The performance of many of the septic tank systems in the Village is suspected to be inadequate and there are concerns that the Creek may be becoming contaminated with fecal material and other pollutants. In the EPA guidelines, this concern is expressed with regard to drinking water, but the observations will also apply to local streams and rivers.

> *"Septic systems also contribute to contamination of drinking water sources. EPA estimates that an estimated 168,000 viral and 34,000 bacterial illnesses each year occur as a result of consumption of drinking water from systems which rely on improperly treated ground water. Malfunctioning septic systems are identified as one potential source of this contamination. A recent example of contamination involved a substantial number of visitors to the New York State Fair in 1999, who became ill after consuming water from a well source, which was likely contaminated by septic systems at an adjacent university. Other examples in which septic systems were attributed to be the pollution source include 82 cases of shigellosis resulting from a contaminated well in Island Park, Idaho in 1995,*

46 cases of hepatitis A at a non-community water supply in Racine, Missouri, and 49 cases of hepatitis A in Lancaster, Pennsylvania in 1980."

Finally, the guidelines state,

> *"While it is difficult to measure and document specific cause-and-effect relationships between onsite systems and the quality of our water resources, it is widely accepted that improperly operating systems are contributors to major water quality problems."*

Based on the observations of the EPA guidelines and the probable poor performance of a significant proportion of the septic tanks in the Village, it is recommended that a centralized treatment system should be used at Wimberley. This will enable the Village to manage wastewater more effectively, to ensure that wastewater is collected, treated, and disposed of in an appropriate manner that will protect the Creek and groundwater sources. In addition, the proposed reclamation of water will provide a valuable water resource for reuse that will help to protect groundwater resources from over-abstraction.

6.3 Treatment Technologies

There are numerous configurations capable of providing the level of treatment defined in Section 6.2. Secondary treatment technologies are typically categorized as either suspended growth or attached growth processes. The following is a list of the secondary treatment processes that were identified as treatment alternatives for the Wimberley:

<u>Suspended Growth Technologies</u>
Conventional Activated Sludge
Membrane Biological Reactors (MBR)
Oxidation Ditch
Sequential Batch Reactors
Aerated and Facultative Lagoons

<u>Attached Growth Technologies</u>
Biological Aerated Filters (BAF)
Trickling Filters
Rotating Biological Contactors (RBC)

These eight technologies were initially considered as possible secondary treatment alternatives for the satellite plants. In order to reduce the number of process options to just five processes, a cursory review was carried out. The review was based on Black & Veatch's experience and included the following objectives: (1) ability to meet effluent criteria, (2) high degree of process reliability, (3) low level of operator attention, and (4) low maintenance.

6.3.1 Cursory Review of Secondary Treatment Technologies

This section identifies the technologies that were eliminated from further consideration through a cursory review of the processes.

Aerated and Facultative Lagoons. An aerated lagoon process consists of a partially aerated 24-hour lagoon followed by several polishing cells. Soluble BOD is converted to bio-mass in the aerated lagoon and the polishing cells are used for nitrification, solids separation and further stabilization of the bio-mass. The polishing cells are dredged every few years to remove the settled solids from the lagoon. This process requires a very low level of operator attention but process reliability due to algae growth can be a problem. Algae blooms are also difficult to remove through sand filtration and may cause problems with UV disinfection. Total nitrogen removal in lagoon systems is typically unreliable, especially when algae growth is dominant. For these reasons, aerated lagoons were eliminated as a possible treatment technology for Wimberley.

Trickling Filters. Trickling filters are an attached growth activated sludge process which general use a rock, plastic, or wood media for the bio-mass to attach to and grow. A circular rotary distributor mechanism is used to apply the wastewater to the media surface. A recycle line is often used to increase the wetting rate to optimize performance of the process. Forced ventilation is also often used to increase the efficiency of a trickling filter. Although there has been operational improvements of this technology over the years, odors are still a chronic problem with this technology. There is an increased potential of odors in warmer climates; therefore, this technology may not be a good choice for Wimberley. In addition, trickling filters would not easily accommodate more stringent effluent criteria. It is unlikely that this process could achieve sufficient nitrogen removal for land disposal of the effluent. For these reasons, trickling filters were eliminated as a possible treatment technology.

Rotating Biological Contactors. Rotating biological reactors have a fixed film bio-mass on a rotating media which is partially submerged in the waste stream. High density plastic in a shape of a disc is typically used for the rotating media. The media provides a surface for the bio-mass to grow. The bio-mass adsorbs soluble organics as the discs rotate through the process water. The bio-mass then rotates up into the air above the process water which provides the oxygen for synthesis. Rotating biological contactor installations have had significant performance and maintenance problems over the years. These problems have nearly eliminated them from use in the municipal wastewater market. Excessive build-up of bio-mass has lead to structural problems with the shafts and media. Bearing failures have also been a maintenance problem at many installations. For these reasons, rotating biological contactors were eliminated as a possible treatment technology for Wimberley.

The remaining five treatment technologies including (1) Conventional Activated Sludge, (2) Membrane Biological Reactors, (3) Oxidation ditches, (4) Sequential Batch Reactors, and (5) Biological Aerated Filters were identified as viable technologies for the satellite plants. A more thorough screening of these five technologies was carried out to identify two technologies that should be fully developed as treatment alternatives.

6.3.2 Description of Viable Treatment Technologies

This section presents a description of the five treatment technologies identified in the previous section as viable technologies for the Wimberley WWTP.

A. Conventional Activated Sludge

Description. The activated sludge process consists of anoxic and aerated bioreactors followed by a clarifier. In the anoxic and aeration phases, organic matter is converted to bio-mass. In the aerobic phase, ammonia is converted to nitrate-nitrogen and in the anoxic phase, the nitrate is converted to nitrogen gas. In the settling phase, the activated sludge is separated from the treated water and is returned to the process. To maintain a good quality effluent, bio-mass is also removed from the process at the rate it is produced.

The aeration equipment typically used in activated sludge processes are coarse bubble diffused aeration, fine bubble diffused aeration, or mechanical surface aerators. It is assumed for the evaluation that fine bubble diffused aeration would be used. The secondary clarifiers would consist of concrete circular basins with energy dissipation inlets, full radius scum skimmers, and a spiral sludge scraper mechanism.

The clarifer effluent will require tertiary treatment using sand filtration followed by UV disinfection in order to meet the required reuse water quality criteria.

Advantages / Disadvantages. Activated sludge technology has the following advantages and disadvantages relative to the other technologies under consideration.

Advantages:
- Proven technology
- Reliable process

Disadvantages:
- Poor sludge settling can affect performance
- Requires daily operator attention, one shift per day minimum
- Requires Tertiary Treatment to meet reuse standards

B. Membrane Biological Reactors

Description. The membrane biological reactor (MBR) process is a developing technology that has emerged in the wastewater treatment market over the past seven years. The use of membranes does not change the kinetics of the activated sludge process, rather they simply replace several conventional treatment units. Membranes are installed in the aeration basins to separate the MLSS from the treated water. Therefore, secondary clarifiers and return activated sludge pumps are not required. In addition, it is not necessary to provide tertiary filters with this process since the membranes reduce solids to a very low level. Diffused aeration equipment is usually used to meet the oxygen demand of the process. Air is also injected below the membranes to agitate them to control bio-fouling of the membranes. The airflow required for agitating the membranes can be significantly greater than that required for the oxygen demand of the process. The process must operate at relatively long sludge ages because an older bio-mass reduces the level of membrane fouling. A vacuum is used to draw the process water through the membranes and recycle pumps are required to re-disperse the MLSS which tends to concentrate near the membranes.

The MBR process does not require a separate tertiary treatment step but will require UV disinfection in order to meet the required reuse water quality criteria.

Advantages / Disadvantages. The MBR technology has the following advantages and disadvantages relative to the other technologies under consideration.

Advantages:

- Small footprint
- Very High Effluent Quality
- Able to maintain effluent quality under a wide variation of influent loads
- Performance is not impacted by filamentous bacteria or bulking sludge
- Modular construction allows for easy expansions
- No tertiary filter required

Disadvantages:

- Equalization may be required to reduce peak flux rate on the membranes
- Fine screens required
- Relatively new technology

C. Oxidation Ditch

Description. The oxidation ditch technology is very similar to the conventional high rate activated sludge process described earlier. The process requires an aeration basin to convert organic matter to bio-mass and a separate settling basin to remove the settleable solids from the treated water. The aeration basin in an oxidation ditch design is generally shaped in an oval configuration often referred to as a racetrack. Mechanical brush aerators, draft tube turbine aerators, and surface turbine aerators are typically used to meet the oxygen demand of the process. The aeration equipment must also be designed to keep a minimum velocity around the racetrack to keep solids in suspension. In some designs, it is necessary to provide additional mechanical pumping equipment to maintain the required minimum velocities.

The oxidation ditch technology is typically classified as an extended aeration process and is designed and operated at relatively long sludge ages. The oxidation ditch process also usually provides some level of nitrogen removal as anoxic zones are developed at the furthest points from the aerators.

The effluent produced by the oxidation ditch process will require tertiary treatment using sand filtration followed by UV disinfection in order to meet the required reuse water quality criteria.

Advantages / Disadvantages. The oxidation ditch technology has the following advantages and disadvantages relative to the other technologies under consideration.

Advantages:

- Proven technology
- Reliable process
- Good nitrogen removal
- Less sensitive to load variations

Disadvantages:

- Requires large footprint to accommodate aeration basin volume
- Requires Tertiary Treatment to meet reuse standards

D. Sequential Batch Reactors

Description. The sequential batch reactor (SBR) process is a modification of the activated sludge process. The main difference is that it is a batch process rather than a continuous flow process. This allows one reactor to serve as both the aeration basin and settling basin. Therefore, secondary clarifiers and return activated sludge pumps are not required for this process.

The SBR process operates on a fill/draw basis. Raw wastewater is directed into the reactor and then discharges as treated effluent after a period of time in which the contents are aerated and settled. One full treatment cycle includes four separate phases including the fill phase, aeration/mixing or react phase, settling phase, and a decant phase. An idle phase can also be added to the end of the cycle to accommodate variations in flow, but it is not necessary for treatment. The aeration equipment typically used in an SBR process includes coarse bubble diffused aeration, fine bubble diffused aeration, mechanical surface aerators, or jet aeration. In addition to the aeration equipment, mechanical mixing equipment is also provided to keep solids in suspension during the non aerated phases of the SBR cycle.

The SBR effluent will require tertiary treatment using sand filtration followed by UV disinfection in order to meet the required reuse water quality criteria. The SBR process may also require an equalization tank in order change the intermittent effluent flow to a continuous flow for the downstream processes.

Advantages / Disadvantages. The sequential batch reactor technology has the following advantages and disadvantages relative to the other technologies under consideration.

Advantages:

- Proven technology
- Flexible treatment options possible through SBR phase adjustments
- Process automation possible

Disadvantages:

- Reduced process reliability
- Batch process results in high peak flows to downstream processes
- Influent and/or effluent flow equalization may be required
- Requires Tertiary Treatment to meet reuse standards

E. Biological Aerated Filters

Description. The biological aerated filter (BAF) technology consists of a reactor basin filled with a submerged media serving as both a surface for biological activity and a means for solids separation. Fine bubble aeration is typically used to support the process. The Kruger Biostyr process and the Infilco Degrement Inc. (IDI) Biofor process are currently the most popular BAF processes. The Kruger Biostyr process uses a low-density media which forms a floating bed in the upper portion of the reactor. The media is held in the basin with a ceiling equipped with filter nozzles. The process water is introduced at the bottom of the filter and flows upward through the reactor. Air is introduced through a grid of aeration equipment installed on the floor of the reactor.

The IDI Biofor process uses a granular clay media supported by a floor equipped with nozzles where the process water is introduced to the reactor. Similar to the Biostyr process, process water flows upward through the filter and discharges over weirs located above the media bed. Air is introduced to the media bed through a separate network of nozzles located above the process water inlet nozzles.

Both the Biostyr and Biofor processes require routine backwashing by taking the BAF unit out of service and introducing backwash water and air to agitate the bed to remove the captured solids from the media. In the Biofor process, backwash water is introduced through the same inlet nozzles used for process water. In the Biostyr process, backwash water is stored above the filter and during a backwash cycle the direction of the flow is reversed and the captured solids are removed with the backwash water at the bottom of the reactor.

The BAF technology has demonstrated reliable performance and effluent quality relative to the more conventional secondary treatment technologies. There are limited installations in the United States, however, and most are second stage polishing units designed for nitrification. These facilities are under low loadings, thus, design condition performance and reliability are not well documented.

It is likely that a multiple BAF units in series would be required to achieve BOD and TSS removal, nitrification and denitrification. The denitrification treatment step may also require methanol addition to enable the process to achieve the required nitrogen concentration. It may also be necessary to include a primary treatment stage, prior to the BAF processes.

Although the BAF process is capable of producing a good effluent with low TSS concentrations, it will require tertiary treatment using sand filtration followed by UV disinfection in order to meet the stringent reuse water quality criteria.

Advantages / Disadvantages. The BAF technology has the following advantages and disadvantages relative to the other technologies under consideration.

Advantages:

- Reduced footprint
- Modular construction allows for easy expansions

Disadvantages:

- Developing technology
- Full scale process reliability has not been well documented
- Primary treatment recommended to reduce solids load on BAF

6.3.3 Comparison of Viable Treatment Technologies

This section presents an economic comparison of the viable treatment technologies. The non-economic factors that must be considered are also presented and compared. The goal for this comparison was to narrow the process options down to two processes for further sizing and costing.

A. Economic Comparison

Costs. The Relative Capital and O&M costs associated with each of the secondary treatment technologies have been considered. A detailed analysis of the capital costs for all five processes was not possible within the scope of this project, so a relative cost factor, and a factor for the plant footprint was determined on a subjective basis. A more detailed cost for the two selected options can be found later in this report. On a life cycle basis, O&M costs are often the controlling factor in establishing the most economical alternative. Since these costs are difficult to quantify for the well established technologies, and nearly impossible to quantify for the developing technologies, an attempt was made to identify relative O&M cost differences between the treatment technologies on a subjective basis. This information was used to better understand the economic impact of each technology on a life cycle cost basis.

For each process option, the costs include all treatment steps required to meet the required reuse water quality. In the case of conventional activated sludge, oxidation ditch, SBR and BAF, this includes the cost for tertiary treatment. For the MBR process, a separate, tertiary treatment step is not required.

The costs were broken down into six categories: capital cost, footprint, labor, power, chemicals, and maintenance. A weighted numbering scale of 1 through 10 was used to score the relative difference in the costs for each technology. The technology with the greatest total sum indicates this technology would likely have the highest costs relative to the others. The costs were not quantified, however, and discretion must be used when drawing conclusions from this information.

A summary of the subjective O&M cost comparison is presented in Table 6-1.

Table 6-1
Subjective Capital and Operation and Maintenance Costs

Parameter	Conventional Activated Sludge	Membrane Biological Reactors	Oxidation Ditch	Sequencing Batch Reactors	Biological Aerated Filters
Capital Cost	5	8	6	7	7
Footprint	7	4	9	6	3
Labor	7	3	6	5	4
Power	7	7	8	6	5
Chemical	2	4	2	2	8

Table 6-1 (Continued) Subjective Capital and Operation and Maintenance Costs					
Maintenance	4	6	3	5	6
Sum	32	32	34	31	33
Subjective costs: 1- 10 with 1 = best and 10 = worst					

B. Non-economic Factors

The preliminary screening process also considered non-economic factors. These factors deal with things that are not easy to put a monetary value on but have an impact on the general appeal of a treatment option. The non-economic factors identified for the treatment technologies were:

1. Public perception – aesthetic considerations and odor
2. Process reliability – ability to cope with variable loads and produce consistent effluent quality under all conditions
3. Operational complexity – how complex is the operation and how difficult is it to understand how it functions.
4. Process flexibility – how easily can the process be modified to meet different effluent criteria or changes in influent load.

Table 6-2 shows a relative comparison of these non-economic factors based on a weighted numbering scale of 1 through 5. The treatment technology with the lowest total score is the most attractive process based on a comparison of these non-economic factors.

Table 6-2 Non-Economic Factor Comparison					
Treatment Technology	Public Perception	Process Reliability	Operational Complexity	Process Flexibility	Sum Total
Conventional	3	2	2	2	9
MBR	1	2	3	2	8
Oxidation Ditch	5	1	1	5	12
SBR	4	3	5	4	16
BAF	2	4	4	3	13
Comparison of non-economic factors where 1 = best and 5 = worst					

6.4 Summary and Recommendations

All of the treatment options appear to have relatively similar costs. The SBR technology, however, appears to be the most cost competitive, with the lowest score of 31. The oxidation ditch had the worst score at 34.

The comparison of non-economic factors uncovered that the SBR, BAF and oxidation ditch technologies are not as attractive as the other technologies based on the defined factors. The SBR technology with a relative score of 16 is significantly greater than the scores for the other technologies. The MBR processes had the lowest score at 9.

The results of the preliminary screening of the secondary treatment technologies are summarized in Table 6-4. The total score is calculated by adding together the scores from the cost and non-economic factor comparison. The final ranking is based on the total score. The MBR had the lowest total score at 40 and came first in the overall ranking. Conventional activated sludge was second.

It is recommended that the two technologies that should be carried forward to the next stage of assessment are conventional activated sludge and the MBR. The activated sludge process will require a tertiary filtration stage following the clarifier. Both processes require screening for preliminary treatment. It is recommended that UV treatment be used for disinfection for both processes.

Table 6-3

Preliminary Screening Results of Secondary Treatment Technologies

Technology Type	Process Name	Advantages	Disadvantages	Preliminary Screening Results
Suspended Growth	Aerated and Facultative Lagoons	• Low labor requirements	• Algae growth causes problems with UV • Unreliable nitrogen removal	First Round Elimination
Suspended Growth	**Conventional Activated Sludge**	• **Proven technology** • **Reliable process**	• **Poor sludge settling can affect performance** • **Labor intensive** • **Requires tertiary filters**	**Total Score: 41** **Ranking: 2nd** **Process Selected**
Suspended Growth	**Membrane Biological Reactors**	• **Smallest footprint** • **Very high effluent quality** • **Consistent effluent quality** • **Reliable performance** • **Modular construction possible** • **No tertiary filter required**	• **Influent flow equalization may be required** • **New technology**	**Total Score: 40** **Overall Ranking: 1st** **Process Selected**
Suspended Growth	Oxidation Ditch	• Proven technology • Reliable process • Good Nitrogen removal • Less sensitive to load variations	• Requires large footprint • Requires tertiary filters	Total Score: 46 Overall Ranking: =3rd
Suspended Growth	Sequential Batch Reactors	• Proven technology • Higher level of treatment possible • Process automation possible	• Reduced process reliability • Batch process causes high peak flows downstream • Influent flow equalization may be required • Requires tertiary filters	Total Score: 47 Overall Ranking: 5th
Attached Growth	Trickling Filters	• Proven technology	• Odors • Poor Nitrogen removal	First Round Elimination
Attached Growth	Rotating Biological Contactors	•	• Significant performance and maintenance problems	First Round Elimination
Attached Growth	Biological Aerated Filters	• Reduced footprint • Modular construction possible	• New technology • Full-scale process reliability undocumented • Primary treatment required • Requires tertiary filters	Total Score: 46 Overall Ranking: =3rd

6.5 Process Design

In the previous section, two process options were identified as the most viable alternatives for Wimberley. The selected processes were conventional activated sludge and MBR. In the following section, a brief description is given for the implementation of each process option at Wimberley and the design specifications are listed. For each process option the design is presented in three phases – firstly, for a plant treating the current downtown catchment, secondly, for the CCN area in 2010, and finally for the ultimate plant design to serve the whole CCN up to 2020. The design parameters presented for the second phase are total capacity values including any processes already present in the first phase.

6.5.1 Conventional Activated Sludge

The conventional activated sludge process was sized using the Black & Veatch activated sludge model. The aeration tankage for each phase will have 10% of the volume reserved for anoxic (unaerated) capacity for denitrification. The process also includes a mixed liquor recycle pump to increase denitrification potential.

The design surface overflow rate (SOR) for the clarifiers was targeted at 400 gpd/ft^2, which is the TNRCC rules, Chapter 317 maximum loading criterion for an extended air enhanced secondary clarifier. The enhanced clarifier has a lower loading rate to enable better capture of the solids.

The final element in the conventional activated sludge design is the tertiary filter which was designed to meet a surface loading criteria of 5 gpm/ft^2 at peak day flow with one unit out of operation.

Table 6-4 summarizes the design specification for the conventional treatment option.

6.5.2 Membrane Bio-Reactor (MBR)

Three different MBR suppliers were approached for typical sizing and cost information: Kubota (supplied by Enviroquip), Zenon and USFilter. In addition, the Black & Veatch activated sludge model was used to design the bioreactor and to validate the designs suggested by MBR suppliers.

Information supplied by the MBR manufacturers is included in the appendices. This includes diagrams of the equipment, case studies, reference sites and general information on their MBR designs for Wimberley. Figures 6-1 and 6-2 show the membrane equipment used by two of the three equipment suppliers approached for the Wimberley Project. The designs for all three suppliers use similar concepts but have differences in their design, which are noted here. Both Zenon and USFilter use hollow fiber membranes in their MBR whereas the Kubota MBR has plate membranes. The USFilter membrane units are contained in a tank separate from the main bioreactor, but both Kubota and Zenon have the membranes positioned within the main bioreactor.

Figure 6-3 shows an example 370,000gpd MBR plant in the Town of Creemore which was housed in a building designed to look like a Barn in order to blend in with the surrounding neighborhood. This particular facility was installed to replace the existing decentralized individual septic tank system. Figure 6-4 is an example layout for the Phase 1 plant, supplied by Enviroquip.

Table 6-4

Design Specification for Conventional Activated Sludge

		Phase 1 Design	Phase 2 Design	Ultimate Design
Annual Average (AA) Flow	gpd	235,300	1,175,785	1,545,726
Maximum Month (MM) Flow	gpd	305,900	1,528,520	2,009,444
Peak Day (PD) Flow	gpd	611,800	3,057,040	4,018,888
Aeration Tanks				
Overall Tank Volume	ft^3	39192	195833	257449
Aerobic Volume	ft^3	35273	176250	231704
Anoxic Volume	ft^3	3919	19583	25745
Depth	Ft	15	15	15
Design Winter SRT	Days	11	12	12
Design Winter MLSS	mg/L	3006	2972	2947
WAS	ppd	468	1909	2491
Temperature Range	ºC	13 to 28	13 to 28	13 to 28
Blower				
Airflow	scfm	582*	2908*	3823*
Diffuser Type		Fine Bubble	Fine Bubble	Fine Bubble
Power	whp	34	170	224
* - Mixing limited condition				
Clarifiers				
Number		2	3	4
Diameter	ft	25	45	45
SWD	ft	14	14	14
Solids Loading Rate (SLR)	ppd/sf	16	16	16
Surface Overflow Rate (SOR)	gpd/sf	312	320	316
Sand Filter (Mixed Media) Design				
Number of Units		2	6	7
AA Filter Loading	gpm/sf	0.8	1.4	1.5
PD Filter Loading (one unit out)	gpm/sf	4.2	4.2	4.7
Total Filter Surface Area	ft^2	200	600	700
Surface Area Per Unit	ft^2	100	100	100
Filter Media Depth	inches	24	24	24

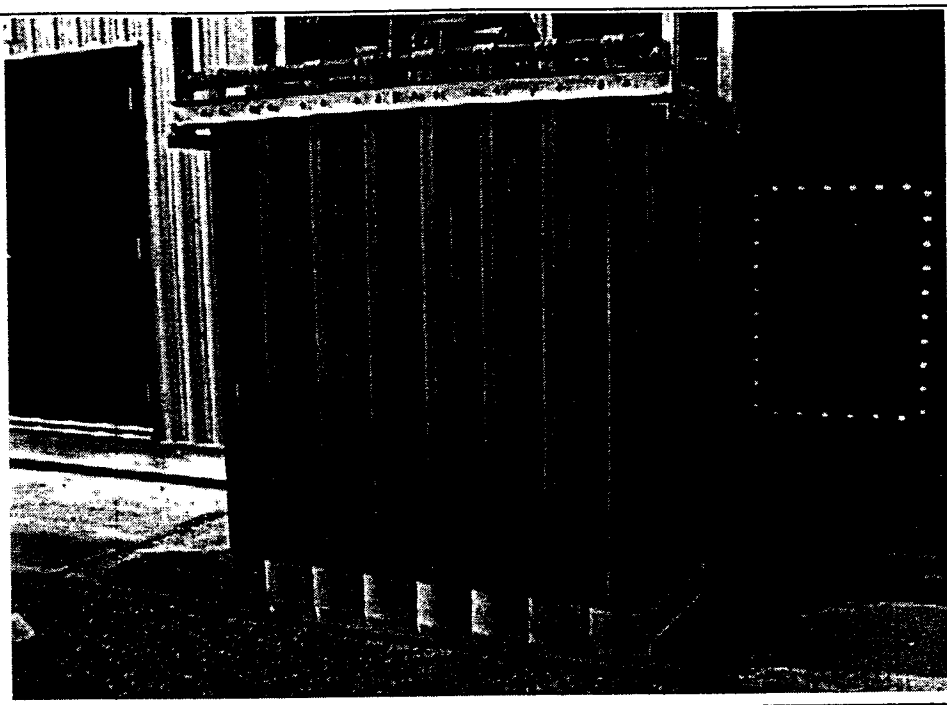

Figure 6-1. Zenon Membrane Cassettes. Courtesy Zenon Website and Brochures.

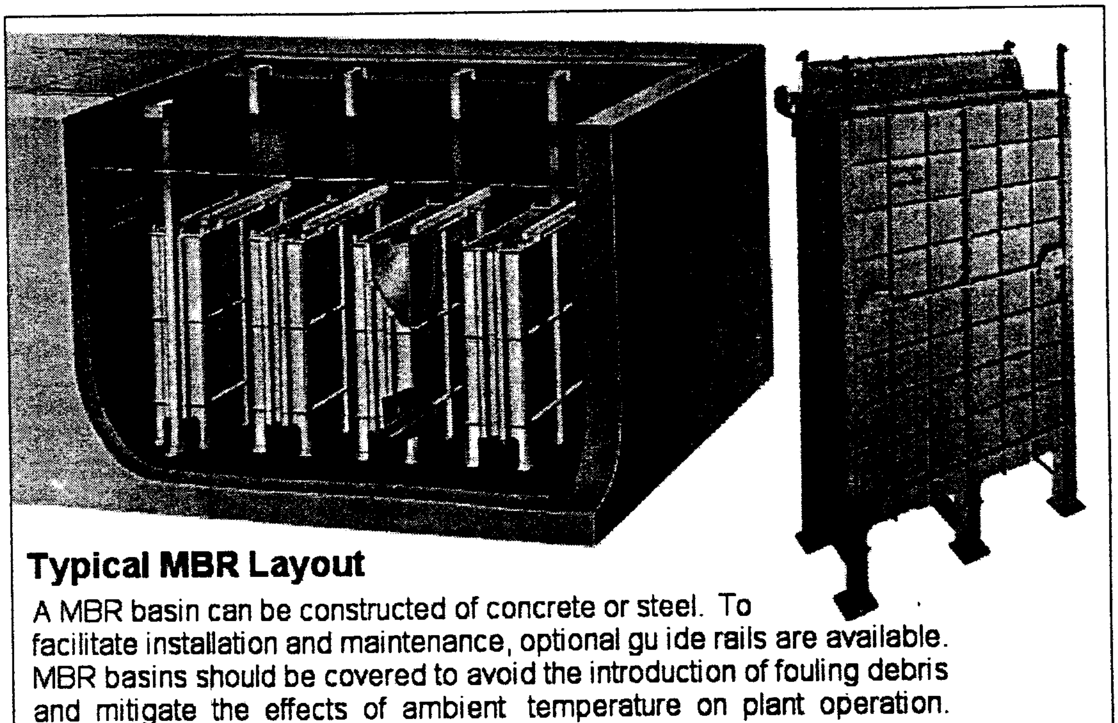

Figure 6-2. Kubota Bioreactor and Membrane Plate. Courtesy Enviroquip, Inc.

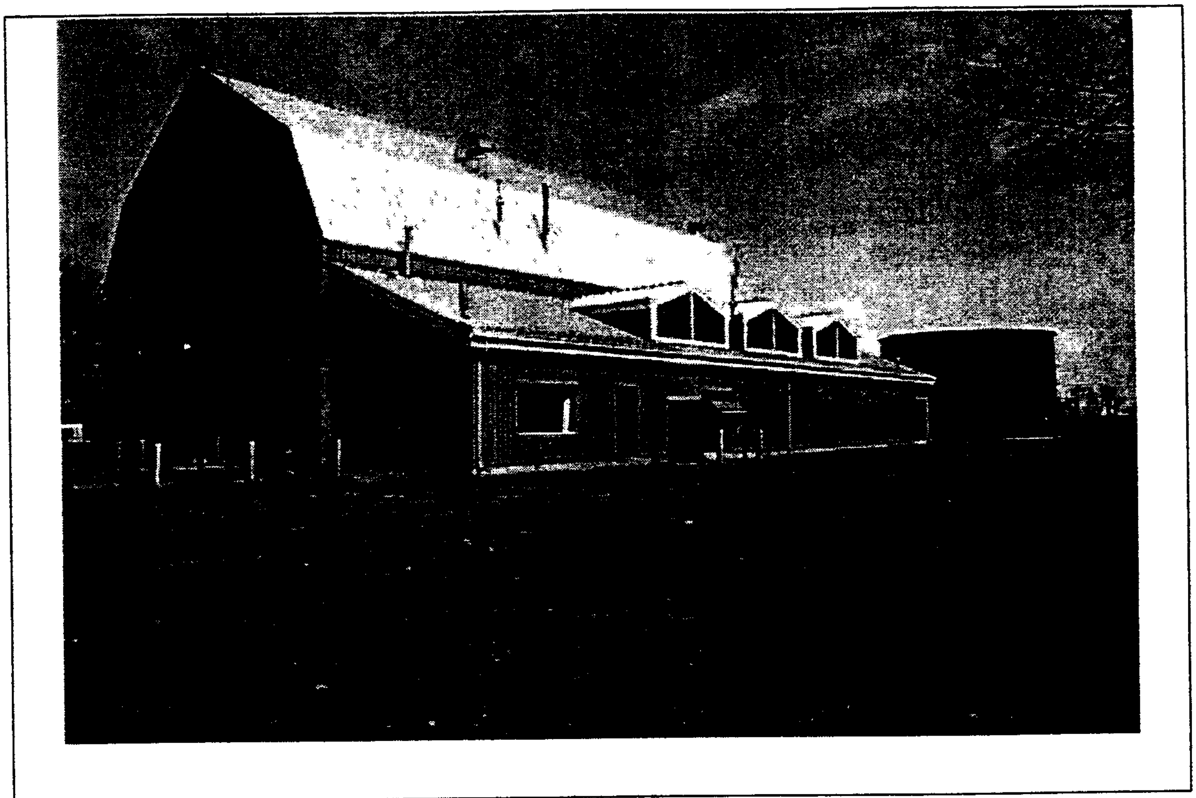

Figure 6-3: Example Unobtrusive 370,000gpd MBR Facility, Town of Creemore. Courtesy Zenon Website

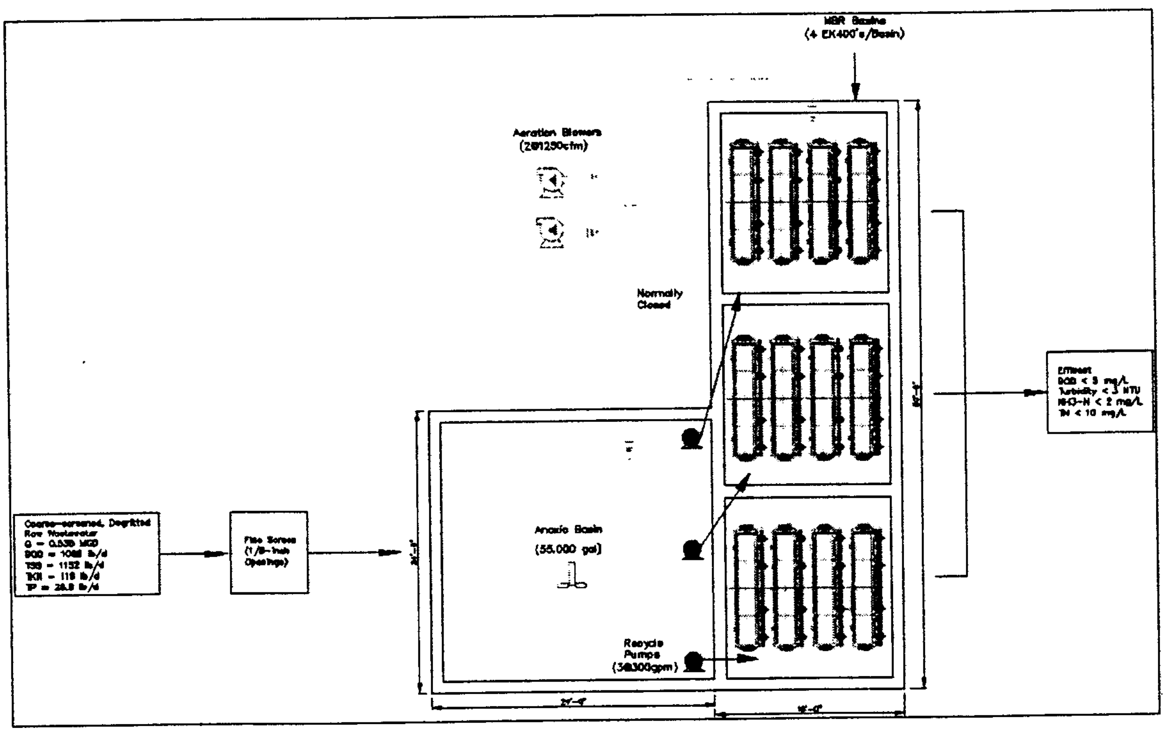

Figure 6-4: Example MBR Layout for Wimberley. Membrane units highlighted in green. Courtesy Enviroquip, Inc.

Table 6-5 shows the design specifications for the MBR plant design. The MBR does not require a secondary clarifier, nor does it need a tertiary filter to meet the required effluent criteria. However, in order to restrict the flux rate on the membranes during peak flow events it is necessary to include a flow equalization tank ahead of the treatment process. The membranes also require fine screens to be installed to prevent debris from damaging the membranes or become entangled on them.

Table 6-5
MBR Design Specifications

		Phase 1 Design	Phase 2 Design	Ultimate Design
AA Flow	gpd	235,300	1,175,785	1,545,726
MM Flow	gpd	305,900	1,528,520	2,009,444
PD Flow	gpd	611,800	3,057,041	4,018,887
Total Volume	ft³	10,565	46,830	61,564
Aerobic Volume	ft³	8,980	39,805	52,329
Anoxic Volume	ft³	1,585	7,024	9,235
Winter MLSS	mg/L	14,996	14,809	14,772
Winter SRT	days	14	15	15
Blower power	whp	44	177	231
Process Airflow	scfm	833	3,646	4,776
WAS	ppd	467	1,942	2,536
Mixed Liquor Recycle	mgd	1.83	9.15	12.03
Flow Equalization Tank	ft³	20,448	102,174	134,321
RT	hours	12	12	12
Screens		1/8"	1/8"	1/8"

6.5.3 UV Disinfection

Whichever process option is selected, disinfection will be required to meet the 20 cfu/100mL reclaimed water limit for Type I water reuse. The MBR produces an effluent with a much lower turbidity and suspended solids concentration than a tertiary sand filter, which means that the required UV dose to achieve the required level of disinfection is lower. Table 6-6 shows the design specifications for a UV disinfection system for Wimberley, based on the selected secondary treatment option.

Table 6-6
UV Design Specifications for Different Process Options

	Conventional Activated Sludge with Sand Filter	Membrane Bioreactor
Design Dose	100 mJ/cm²	80 mJ/cm²
UV Transmittance (@254nm)	>55%	>65%
Filtered Turbidity	<2 NTU (95%ile basis) (Not to exceed 5 NTU)	<0.2 NTU (95%ile basis) (Not to exceed 0.5 NTU)

6.5.4 Sludge Processing

Both of the selected processes require sludge processing facilities. It is proposed that sludge treatment should be carried out using an aerobic digester and sludge drying beds.

For the conventional activated sludge plant the waste activated sludge (WAS), must be thickened prior to being added to the digester in order to reduce the size of the digester needed. A gravity belt thickener is proposed for this purpose. The MBR WAS will not require thickening as it has a higher suspended solids concentration. Table 6-7 lists the design specifications for the proposed sludge treatment.

Table 6-7 Sludge Processing Design Specification							
		MBR			Conventional		
		Phase 1	Phase 2	Ultimate	Phase 1	Phase 2	Ultimate
WAS							
Mass flow	ppd	467	1942	2536	468	1909	2491
TSS	%	1.5	1.5	1.5	0.5	0.5	0.5
Flow	gpd	3733	15524	20272	11223	45779	59736
Gravity Belt Thickener							
Size	m	Not Required			2	2	2
Operation	hrs/wk				7	30	38
Application rate	gpm/m				94	89	92
Thickened Sludge							
Mass Flow	ppd				468	1909	2491
TSS	%				2.0	2.0	2.0
Flow	gpd				2806	11445	14934
Aerobic Digester							
Volume	ft^3	27,179	113,709	148,450	19,075	78,409	102,272
SRT	days	60	60	60	60	60	60
Drying Bed							
RT	days	100	100	100	100	100	100
Depth	inches	12	12	12	12	12	12
Surface Area	ft^2	1,697	7,101	9,271	1,588	6,529	8,516