Alternatives Evaluation Report

Morro Bay-Cayucos Wastewater Treatment Plant
Morro Bay, California

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Executive Summary

The City of Morro Bay and the Cayucos Sanitary District (MBCSD) jointly own the Morro Bay-Cayucos Wastewater Treatment Plant (WWTP), which discharges treated effluent to the Pacific Ocean. The WWTP is currently rated for a flow-rate of 2.06 million gallons per day (MGD) on an annual average basis and 2.36 MGD during peak seasonal periods. These ratings are based on 100% of the flow going through the primary treatment process and approximately 50% (or 1 MGD) going through the secondary process. Over the past three years, the average annual daily influent has been 1.14 mgd (55% of the rated capacity) with the majority of this flow being routed through the secondary process. Effluent concentrations during this same time period averaged 62 mg/l and 42 mg/l for BOD_5 and TSS, respectively, with overall removal efficiencies of 83% BOD_5 and 89% TSS. Comparatively, effluent concentrations averages over the past five years were 52.4 mg/l for BOD_5 and 35.5 mg/l for TSS.

Over the past three years, total plant influent flow has decreased, despite the fact that the population of both Morro Bay and Cayucos has continued to increase. However, during the same period, the solids loading to the WWTP has increased, in the form of higher influent concentrations of both BOD_5 and TSS. Currently, the WWTP is rated for an influent concentration of 280 mg/l for both BOD_5 and TSS. The average concentrations for BOD_5 and TSS over the last three years were 362 mg/l and 378 mg/l, respectively, (a 30% and 35% increase respectively over the rated values.) This is most likely attributable to water conservation efforts and inflow/infiltration reduction (repairs to the Atascadero Sewer Main in 1998 and 2000), which results in less water to transport solids and thus increases concentrations.

In an ongoing effort to lower the concentrations and mass emissions of effluent BOD_5 and TSS, the City/CSD has commissioned this study to evaluate two process modifications that could potentially increase the percentage of total plant influent through the secondary treatment process, increase the removal efficiencies of the secondary process, and ultimately lower the concentrations of BOD_5 and TSS being discharged to the ocean. The two process modifications are Flow Equalization and Trickling Filter Media Replacement.

Flow Equalization consists of incorporating a large holding basin within the treatment process to allow for the dampening of the diurnal fluctuations in the influent wastewater stream by controlling release from the holding basin. An in-line flow equalization basin after primary treatment was evaluated for the WWTP. This study shows that a 64-ft diameter basin, 10-ft deep, provides sufficient volume to equalize the current and future expected daily flow-rate fluctuations. In addition, flow equalization can provide nearly constant solids loadings to the secondary treatment process, thus simplifying operations and stabilizing clarifier-settling efficiency. Because prolonged aeration in the equalization basin coupled with equalized solids loadings to the secondary process are likely to reduce BOD_5 and increase solids settlability, a 5 to 10% increase in BOD_5 removal efficiency could reasonably be expected in the secondary process, thus lowering effluent BOD_5 concentrations in the range of 12 to 24 mg/l (5% to 10% of primary effluent BOD_5 = 240 mg/l.) One drawback to flow equalization is an increase in odors generated from aerating primary effluent in order to keep the wastewater in an aerobic condition. Budgetary costs for constructing an equalization basin are approximately $650,000.

Trickling Filter Media Replacement consists of replacing the existing rock media with plastic media. The plastic media provides a larger surface area than the rock media, allowing more contact with the BOD_5 pollutants, thus increasing the removal of BOD_5 and lowering of the effluent concentrations and corresponding mass emissions. Research has shown that simply replacing rock media with plastic media at the same depth does not necessarily provide an

appreciable increase in removal efficiency; however, increasing the depth of plastic media to at least 12-feet can provide a measurable difference in BOD_5 removal efficiency. At 12-ft in depth, removal efficiencies of approximately 90% BOD_5 removal (effluent BOD_5 concentrations of less than 30 mg/l) can be expected for both current and future conditions; however, the additional solids loading resulting from these improved removal efficiencies coupled with increasing influent wastewater flows and BOD_5 and TSS concentrations, could potentially put a strain on the existing secondary clarifier under future conditions. Budgetary costs for replacing the media and adding depth are estimated at $900,000.

While both alternatives are considered feasible and would enhance effluent water quality, they should not be considered stand-alone solutions to meeting more stringent long-term water quality requirements; rather, they should be viewed as realistic components of a more comprehensive plant upgrade. Even though WWTP influent flows have been significantly reduced in recent years, the increases in influent BOD_5 and TSS are significant enough to warrant concern over the ability of the secondary process (and solids handling facilities) to handle future solids loadings of the entire influent flow.

Prior to investing significant funds in the implementation of flow equalization and trickling filter modifications, a more comprehensive capacity evaluation of the entire WWTP (both liquid and solids streams) should be conducted based on potentially more stringent effluent discharge requirements. From this evaluation, a long-term WWTP Master Plan followed by a phased capital improvement program should be developed to proactively implement strategic process modifications/upgrades in anticipation of these potentially more stringent effluent water quality requirements.

In the meantime, the MBCSD should consider other low capital cost alternatives such as enhanced primary treatment (chemical coagulation) that will achieve the same goals of increasing the percentage of flow through the secondary process and enhancing effluent water quality.

1. Introduction

The City of Morro Bay and the Cayucos Sanitary District (MBCSD) jointly own the Morro Bay-Cayucos Wastewater Treatment Plant (WWTP), which is operated by the City of Morro Bay. Treated effluent from the plant, consisting of blended primary- and secondary-treated wastewater, discharges to unstressed open-ocean waters of the Pacific Ocean via an outfall pipeline and diffuser system which extends approximately 4,400 linear feet in a northwesterly direction and terminates approximately 2.2 miles north of Morro Rock (35°23'11"N latitude and 120°52'29"W longitude.) MBCSD currently optimizes the efficiency of wastewater treatment through the plant by routing a significant portion of flow through the secondary treatment process.

In an ongoing effort to improve effluent quality, MBCSD initiated this study of two specific process modifications in the hopes of maximizing the use of its existing facilities and minimizing the need for costly new expansion facilities: flow equalization and trickling filter media replacement. The goals for these process modifications are to increase both flow and removal efficiency through the WWTP's existing secondary treatment processes in order to improve the quality of effluent going to the ocean.

This study offers (1) a brief review of historical and anticipated future operations and effluent water quality, (2) an evaluation of flow equalization of primary effluent to stabilize flow and organic loadings on the existing secondary process, (3) an evaluation of replacing the trickling filter media and modifying the trickling basins with or without flow equalization to increase removal efficiencies within the trickling filters themselves and to stabilize organic loadings on the solids contact tank and secondary clarifier, and (4) conclusions and recommendations for next steps.

2. Background

This section offers a brief overview of the existing facility operations and influent and effluent quality. This section provides the basis for assumptions made in examination of the process modification alternatives studied below.

2.1 Existing Wastewater Facilities and Operations

Facilities

The WWTP is designed for an Average Dry Weather Flow (ADWF) of 2.06 million gallons per day (MGD), a Peak Season Dry Weather Flow (PSDWF) of 2.36 MGD, and a Peak Wet Weather Flow of 6.6 MGD. The preliminary, primary, and disinfection treatment facilities are sized for these design flows. The secondary treatment facilities, consisting of interstage pumping facilities, two trickling filters, a solids contact tank, and one clarifier, are designed for an ADWF of 0.97 MGD to meet effluent concentrations of 30 milligrams per liter (mg/l) for both 5-day Biochemical Oxygen Demand (BOD_5) and Total Suspended Solids (TSS) (Carollo, 1999). The original design criteria for the WWTP, including size, number of basins, organic and hydraulic loading rates, etc. are shown in Appendix A. The current WWTP Layout and Hydraulic Profile are shown in Appendix B.

Two major factors to consider when evaluating the treatment capacity of processes within the WWTP are total influent wastewater flow (MGD) and influent strength (mg/l). The design criteria for influent wastewater strength at the time of the 1985 major plant upgrade was 280 mg/l for both BOD_5 and TSS. As discussed below, the influent wastewater strength has increased over the past 3 years (2001, 2002, and 2003) to average concentrations for BOD_5 of 362 mg/l and TSS of 378 mg/l. These concentrations show an approximate increase of 30% and 35% over rated influent concentrations and are most likely attributable to water conservation and inflow/infiltration reduction efforts. (Basically, there is less water to transport the solids, thus increasing the concentrations.)

Operations

An understanding of the current flow control mechanisms at the WWTP is necessary to evaluate the feasibility of increasing wastewater flow through the existing secondary process. As stated in the 1999 Carollo Report and confirmed by WWTP staff, wastewater-flow to the secondary process is controlled by three different control devices: a *24-inch butterfly valve* (a.k.a. blending valve) located downstream of the primary clarifiers; a *sluice gate* at the Interstage Pumping Station facility; and a *6-inch drain* which bypasses the sluice gate and flows directly into the Interstage chamber. The current facility layout is shown in Appendix B.

As the blending valve is opened and the sluice gate is raised, *less* wastewater flow is conveyed to the secondary process. Conversely, as the blending valve is closed and the sluice gate lowered, *more* wastewater flow is conveyed to the secondary process. The maximum wastewater flow to the secondary process is achieved when the blending valve is fully closed, the sluice gate is completely lowered, and the 6-inch drain line is fully open. During interviews with WWTP staff, operations personnel stated that when they operate the plant in this maximum-flow-to-the-secondary mode, wastewater flow tends to back-up into the primaries during peak hourly periods. Despite this minor inconvenience, operations staff continue to maximize flow to the secondary process while closely observing primary effluent launder levels.

To validate this observation and to determine the theoretical maximum flow, a hydraulic capacity calculation was performed to evaluate the piping between the primaries and the Interstage. The results from the calculation indicate that the maximum flow rate is approximately 6.6 MGD. (See Appendix C.) In reviewing the hourly flow data for years 2002 and 2003, there were 5 days when the hourly flow coming into the plant exceeded this capacity. In actuality, Operations staff observe increases in primary effluent launder levels when the flow rate is greater than 3.5 MGD, which occurs at certain times during the year, especially peak wet weather days and occasional peak hourly summer days. One reason for the discrepancy between the theoretical and actual flow capacities may be that the hydraulic resistance of the valving and piping within the system is greater than estimated.

Once primary effluent enters the Interstage chamber, it flows by gravity through a series of baffles into the Trickling Filter Recirculation Pump Station. The TF Recirculation Pump Station operates at a constant flowrate, typically 1-2 times the average dry weather flow rate. This operation ensures minimum wetting requirements for the trickling filters to maintain biological activity and treatment during low flow periods. The Secondary Clarifier Pump Station operates at a varying rate by maintaining a constant wetwell level in the Interstage Chamber. With this control strategy in place, diurnal fluctuations of wastewater flow and strength are routed through the secondary clarifier, resulting in hourly peak hydraulic and solids loadings on the clarifier.

2.2 Historical WWTP Wastewater Influent Quantity and Quality

Historical wastewater data for WWTP influent and effluent (both quantity and quality) has been well documented over the years. Figures 2.2-1 through 2.2-4 present a summary of the data from the last 10 years. The most notable observations from these trends occur in the last three years (2001 - 2003) and include a 25% *decrease* in 3-year average influent flow compared to the 10-year average influent flow, and an *increase* in the 3-year average wastewater strength compared to the 10-year average wastewater strength of 32% for BOD_5 and 15% and TSS. This trend is most likely attributable to repairs of the influent trunk sewer in Atascadero Road in 1998 and 2000 (resulting in less infiltration) and the installation of a more accurate influent flow meter in 08/01. See 2001 Annual Report.

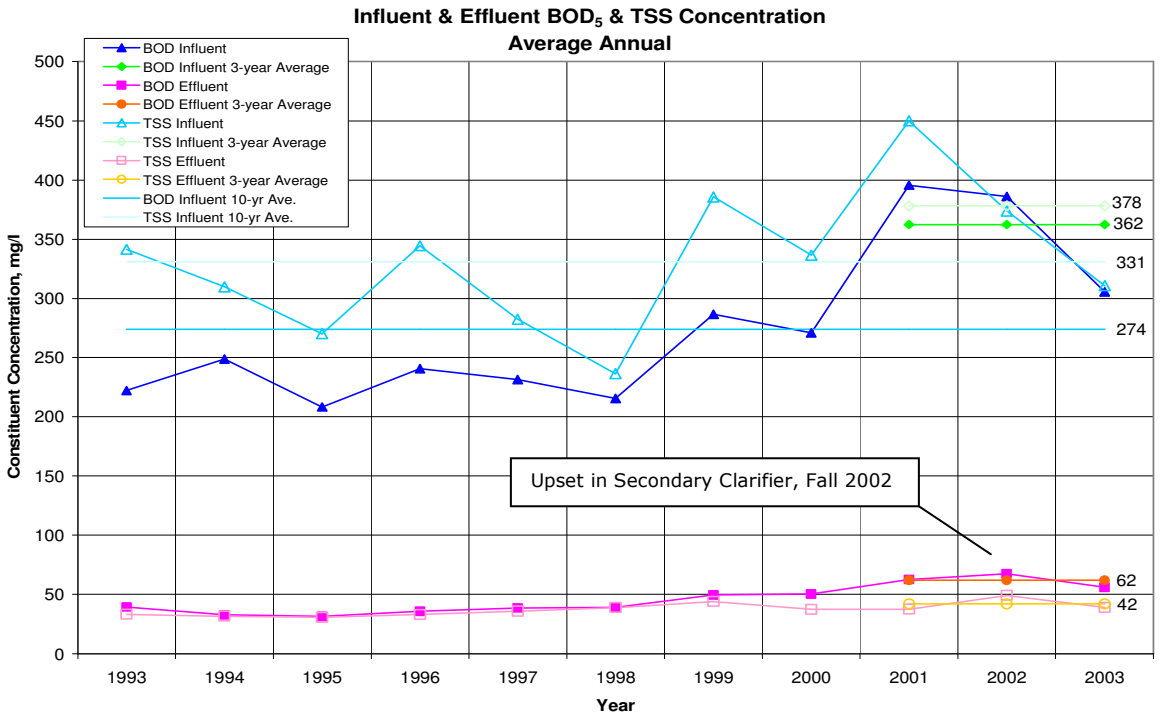

Figure 2.2-1: Average Annual Influent and Effluent BOD_5 and TSS Concentration

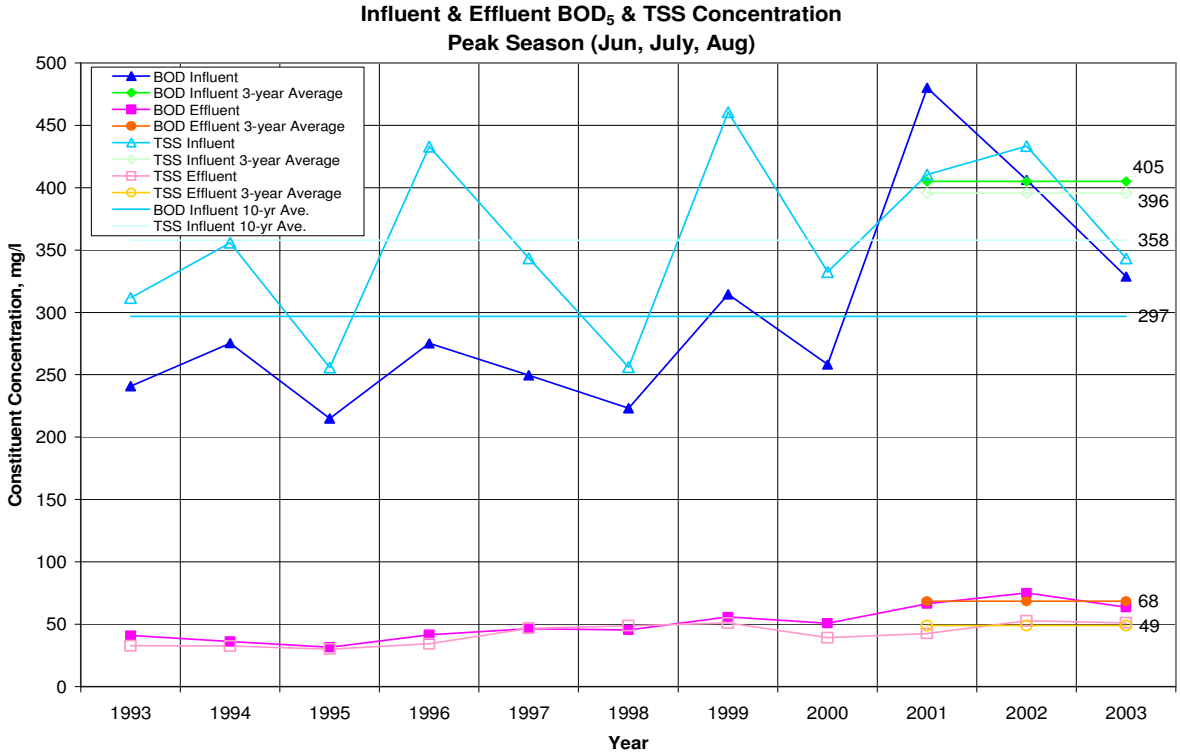

Figure 2.2-2: Peak Season Influent and Effluent BOD_5 and TSS Concentration

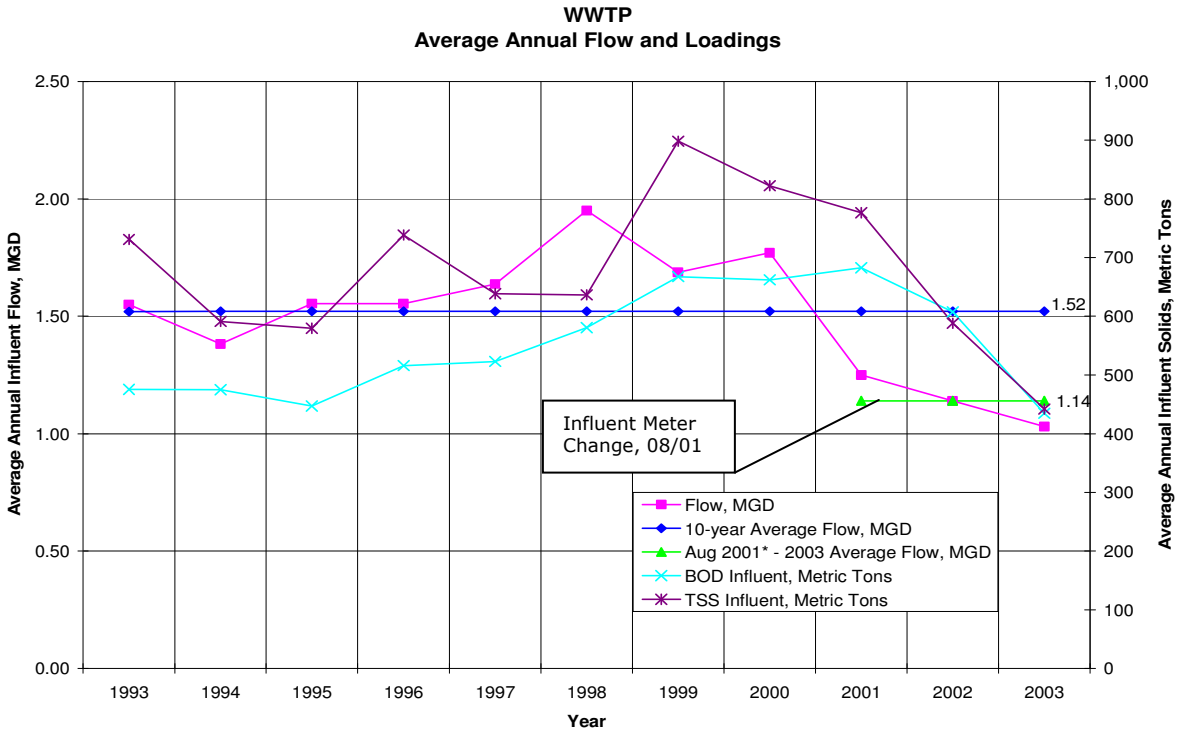

Figure 2.2-3: WWTP Average Annual Flow and Solids Loadings

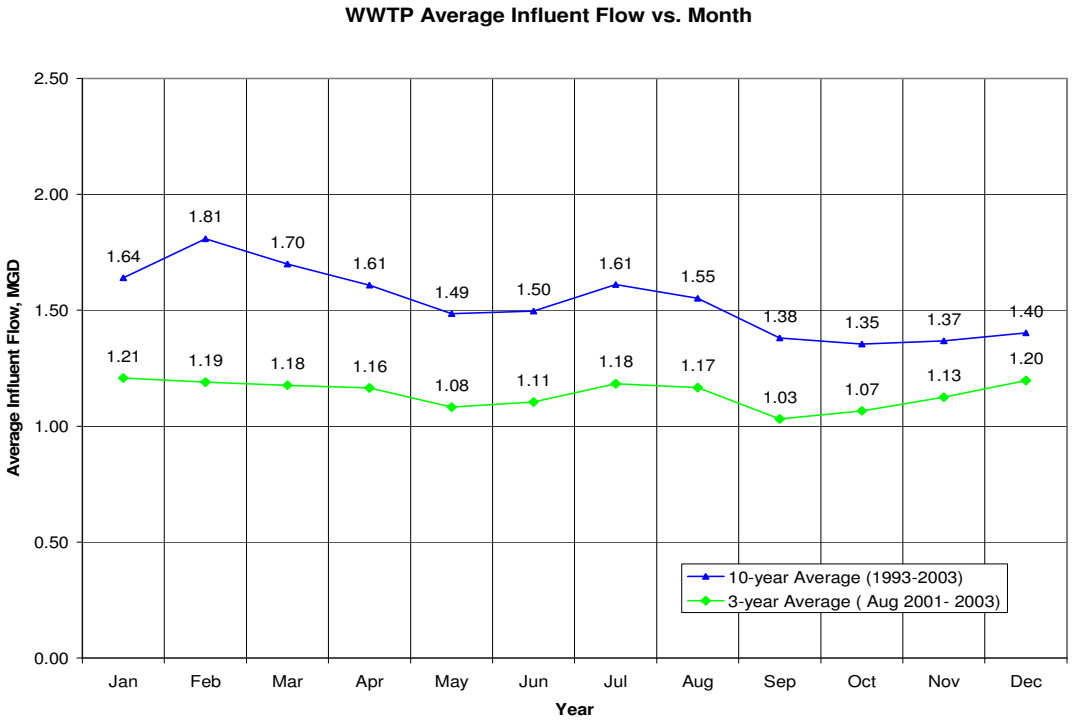

Figure 2.2-4: Average Influent Flow vs. Month

2.3 Projected WWTP Wastewater Influent Quantity and Quality

Wastewater influent flows are projected to increase at a relatively slow rate over the next few years until reaching the planned build-out populations for both communities, 12,200 for the City of Morro Bay and 4,733 for Cayucos. As estimated in the recent Marine Research Specialists Report, the population and corresponding influent flows will likely increase by approximately 20% over the next 30 years, increasing the current annual average daily flow total from approximately 1.14 mgd to 1.38 mgd, and the peak season flow from 1.25 mgd to 1.50 mgd (2003).

For purposes of this report, four different design scenarios are used to analyze the two process modifications: current average annual and current peak season, and anticipated future average annual and future peak season. Future influent flow projections are based on the population projections described above. However, even though the average influent concentration strength has shown an increase over the past three years, it is likely that this trend is a result of system repairs and improved equipment accuracy, rather than an actual change in influent concentrations. Therefore, future concentrations of BOD_5 and TSS are expected to remain approximately the same as present levels.

Table 2.3-1 – Wastewater Influent Projections

Scenario	Influent Flow (MGD)	Influent BOD_5 (mg/l)	Influent TSS (mg/l)
Current-Average Annual	1.14	362	378
Current – Peak Season	1.25	405	396
Future-Average Annual	1.38	362	378
Future-Peak Season	1.50	405	396

2.4 Effluent Water Quality Comparison

The WWTP has a 301(h) modified National Pollutant Discharge Elimination System (NPDES) permit for discharge of blended primary and secondary effluent. Compared to the standard full secondary treatment permit, two major secondary effluent limitations, Biochemical Oxygen Demand (BOD_5) and Total Suspended Solids (TSS), are eased under this permit, whereas all other secondary discharge limitations remain in effect. NPDES requirements are further discussed in Appendix H.

As shown in Table 2.4-1, the WWTP is well within the limitations of the current Modified Discharge Permit in almost all instances. An asterisk (*) is placed next to the category where, on rare occasion, the limitation was exceeded. See Appendix I for a summary of these occurrences.

Table 2.4-1 Effluent Quality Comparison

Constituent	Limitation	Units	Effluent Limitation Value for Current Modified Discharge Permit	WWTP Effluent Quality (Average of Past 3-years)
BOD_5	30-day average	mg/l	120	62
	Maximum at any time	mg/l	180	114
	30-day removal efficiency	%	30	82.1
TSS	30-day average	mg/l	70	42*
	Maximum at any time	mg/l	105	105*
	30-day removal efficiency	%	75	87.4
pH	Minimum		6.0	7.1
	Maximum		9.0	7.7

Despite the fact that the WWTP consistently meets permit effluent limitations, the MBCSD is in favor of implementing process modifications that will continually improve the quality of plant effluent. The remainder of this report examines two proposed process modifications.

3. Description of Alternatives

Two process modifications were evaluated as part of this study: (1) flow equalization of primary effluent and (2) trickling filter media replacement. Both of these modifications were developed from the MBCSD's desire to improve effluent water quality by increasing the percentage of overall plant flow routed through the secondary process thereby improving overall removal efficiencies and lowering effluent concentrations and mass emissions of the major wastewater constituents.

As mentioned above, the treatment plant staff currently route as much of the plant flow as possible through the secondary process by closing down the blending valve. The proposed modifications are designed to allow a greater percentage of plant flow to pass through secondary treatment process and to improve the efficiency of that secondary treatment process.

3.1 Flow Equalization

Flow equalization is defined as the dampening of diurnal-flowrate and constituent-loading variations in such a way that nearly constant flowrates and constituent loadings are discharged to downstream processes. The effect of nearly constant flowrates and nearly constant constituent loadings is an elimination of drastic peaks in flow rate, allowing a reduction in the size and cost of downstream facilities to handle the same total volume of flow. Two of the principle benefits that may derive from flow equalization are enhanced biological treatment due to elimination of shock loadings from daily peak periods, and enhanced effluent quality and thickening performance of secondary sedimentation tanks following biological treatment due to nearly constant solids loading (Metcaff & Eddy, 1991).

For these reasons and because the total average daily flowrates are not significantly greater than the rated capacity of the secondary process, 1.14 mgd vs. 0.97 mgd, respectively, the idea of flow equalization was determined to be deserving of further evaluation.

3.2 Trickling Filter Media Replacement

Media replacement at the WWTP would consist of changing the existing rock media to the latest in plastic media. The main benefits of changing the media include increasing the surface area available for treatment and enhancing the distribution of flow across the media. This combination of improved distribution over an increased surface area creates a more efficient surface for the biological fixed-film to grow upon and increases the total contact area for fixed-film growth. Because the biological fixed-film is what performs the wastewater treatment, an increased surface area increases the total amount of BOD_5 removed, thereby improving removal efficiency. While research has shown that shallow trickling filters (such as WWTP's trickling filters) perform about the same regardless of media type, it also shows that by increasing the depth of plastic media and by distributing the wastewater more uniformly across the media, greater efficiencies can be realized.

Because research indicates that the potential for improved trickling filter efficiency depends not just on media type but on media depth as well, this study examines the potential modification of trickling filter depth along with replacement of media at the WWTP.

4. Analysis of Alternatives

A focus of this study is to determine what impact the two process modifications described above will have on the quality of effluent from the WWTP. To try to predict effluent concentrations under modified processes, it was first necessary to collect data or make assumptions on the strength of wastewater flows under the existing treatment process. Sections 4.1, 4.2, and 4.3 describe the data collection and preliminary data analysis used as a basis for the process modification study. Sections 4.4 and 4.5 provide the specific analysis of the two alternatives.

4.1 Data Collection

The focus of data collection was to estimate the effects of flow equalization on the secondary process and to determine how process efficiency varies throughout the day and in the different unit processes. Hourly water quality and flow data was collected during the months of September through November 2003 to develop a better understanding of the diurnal patterns through the treatment plant. Because September through November is the low flow period of the year, the trends established during this period are normalized in the following sections of this report to more realistically evaluate the performance of the alternatives under average and peak conditions.

Figures 4.1-1 through 4.1-5 illustrate the diurnal patterns of wastewater quantity (mgd) and strength (mg/l of BOD_5 and TSS) throughout the day during the sampling period. Wastewater quality sampling was conducted at the following locations and time periods during the study period: raw influent, 10/27/03 - 11/03/03; primary effluent, 9/29/03 - 10/8/03; trickling filter effluent, 10/15/03 - 10/27/03; secondary clarifier effluent, 11/03/03 - 11/10/03; and, final effluent, 11/10/03 - 11/17/03. Representative samples were taken for all days of the week including weekends and weekdays. BOD_5 and TSS data is included in Appendix D of this report.

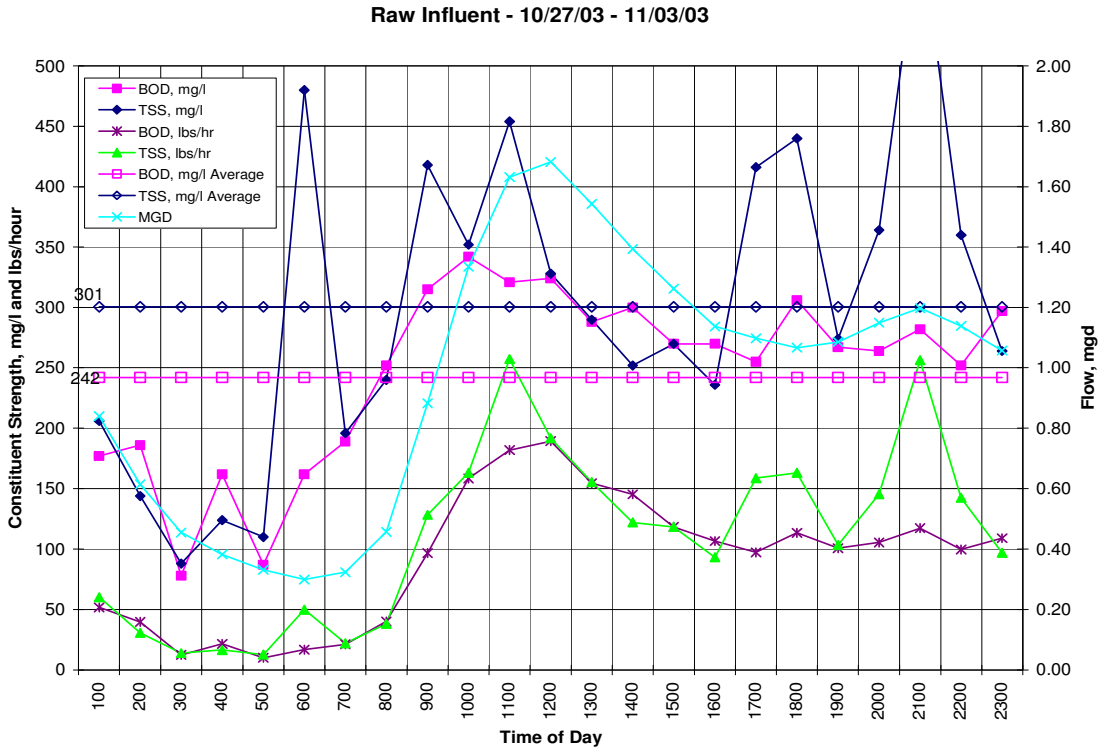

Figure 4.1-1: Raw Influent Strength vs. Time of Day

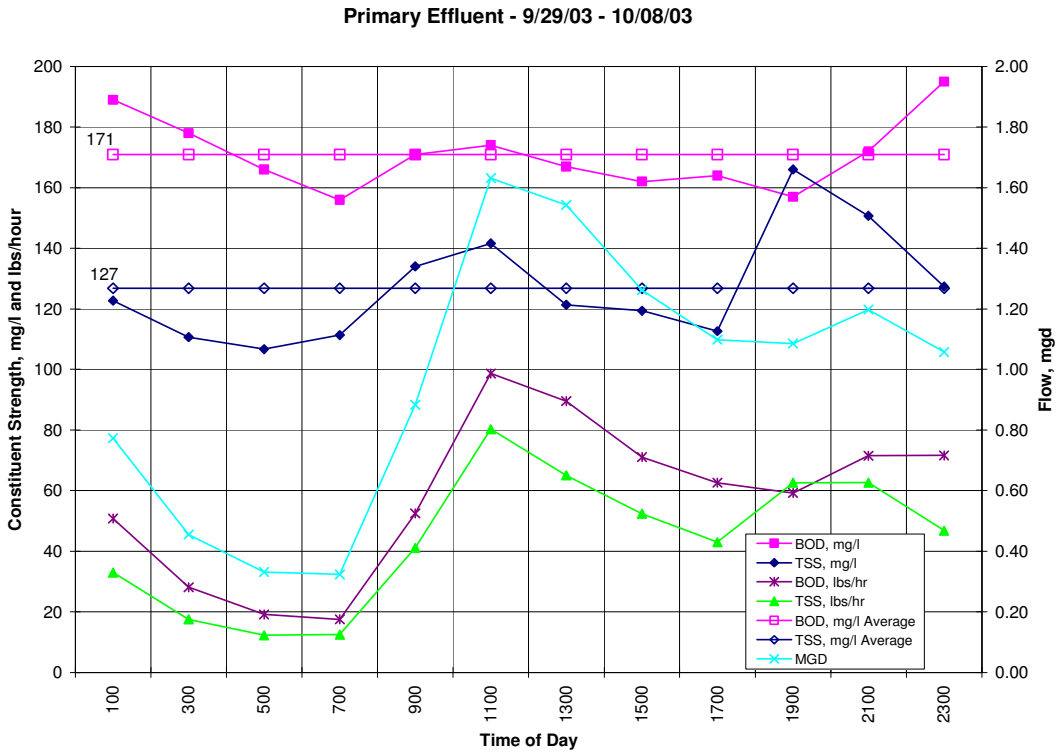

Figure 4.1-2: Primary Effluent Strength vs. Time of Day

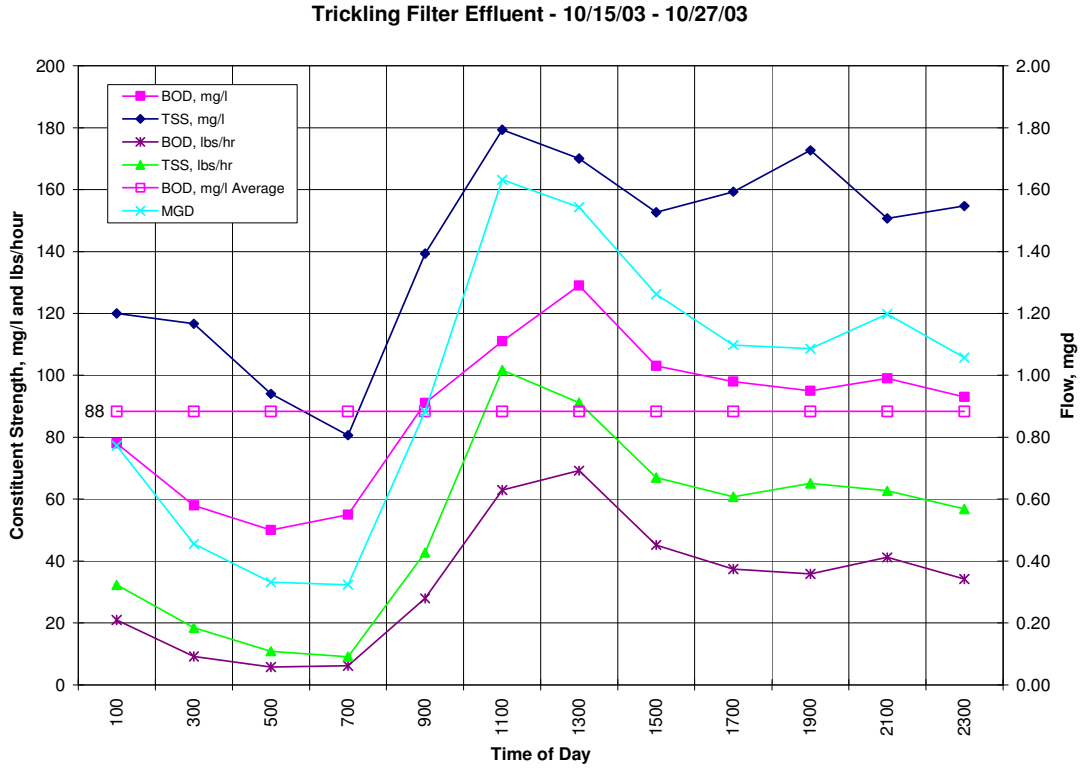

Figure 4.1-3: Trickling Filter Strength vs. Time of Day

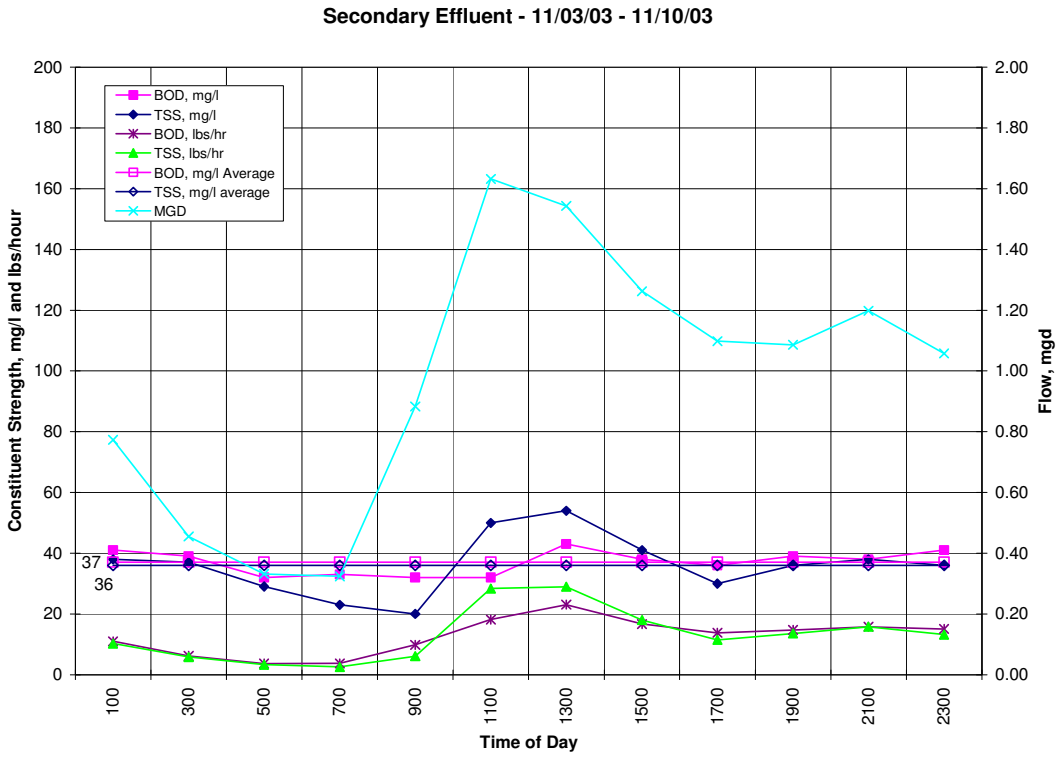

Figure 4.1-4: Secondary Effluent vs. Time of Day

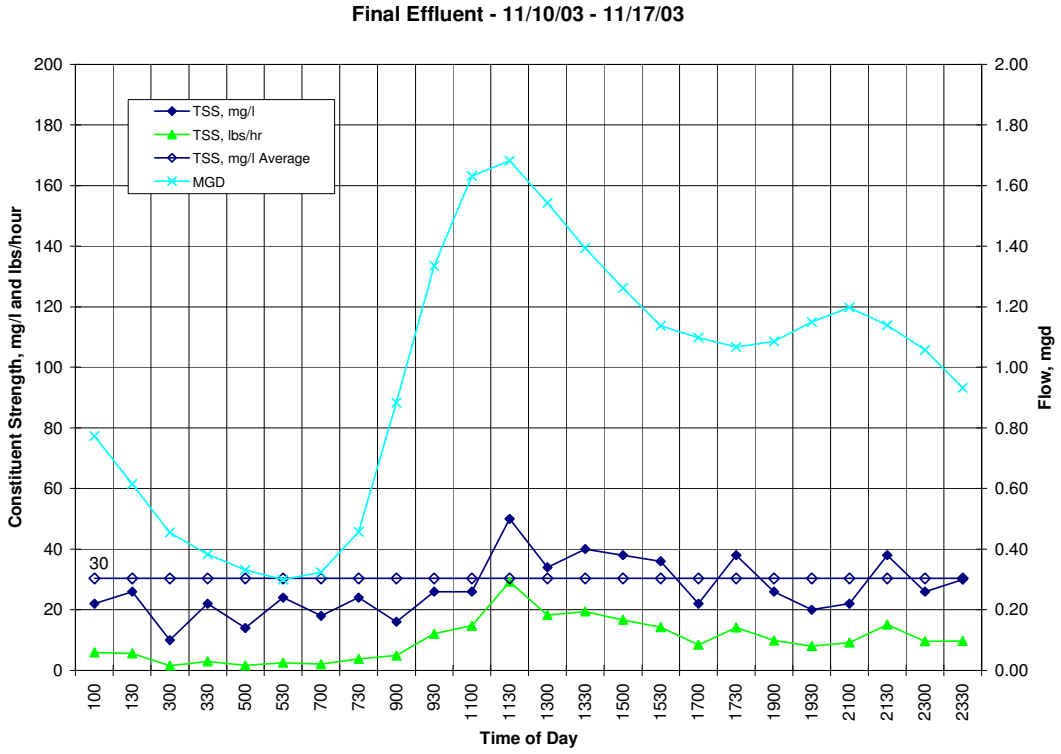

Figure 4.1-5: Final Effluent vs. Time of Day

4.2 Process Removal Efficiencies during Sampling Period

This section provides a brief discussion on the removal efficiencies of BOD_5 and TSS as the wastewater progresses through the treatment process. In order to understand the efficiency of the secondary process itself, an approximation was made regarding the efficiency of the preliminary/primary treatment process (which determines the concentration of influent to the secondary treatment process).

Table 4.2-1 shows the concentration and removal efficiency values used for the analysis of the two process modifications. These values assist in developing the criteria for evaluating the alternatives and provide a method of estimating the effluent quality with or without process modifications.

These values were determined based on the following assumptions: (1) wastewater strengths and flows remained fairly constant over the three-month sampling period, (2) 90%+ of the total wastewater was routed through both primary and secondary unit processes during the sampling period, and (3) plant operations remained normal and without any significant disruptions and/or upsets during the sampling period.

Table 4.2-1

Unit Process	Average Influent Concentration		Average Effluent Concentration		Removal Efficiency	
	BOD_5, mg/l	TSS, mg/l	BOD_5, mg/l	TSS, mg/l	BOD_5, %	TSS, %
Preliminary/ Primary	242	301	171	127	29	58
Secondary	171	127	37	36	78	72
Overall	242	301	37	30	85	90

Typical BOD_5 and TSS removal efficiency ranges for preliminary/primary treatment for municipal WWTP(s) in the United States are 25 to 40% and 40 to 70%, respectively (Metcalf and Eddy, 1991). Typical BOD_5 and TSS removal efficiency ranges for a Trickling Filter/Solids Contact secondary treatment process vary depending on many factors, including solids loading rates (lbs/1,000 cf/day), hydraulic loadings (gal/day/sf), temperature, media type and depth, etc. Typical BOD_5 removal efficiencies for trickling filters, with rock media at 5-ft of depth with a 1:1 recycle rate, range from 70 to 90% for BOD_5 mass loading rates from 90 lbs/1,000 cf/day to 10 lbs/1,000 cf/day, respectively.

During the sampling period, the WWTP secondary process had an average BOD_5 solids loading rate of approximately 45 lbs/1,000 cf/day and showed an average removal efficiency of 78%. When compared to the typical removal rates for stone media, the WWTP's trickling filter process should remove between 75 to 80%, indicating that the performance of the existing trickling filters is within normal range.

4.3 Normalization of Wastewater Data for Evaluation of Alternatives

One of the major factors in evaluating alternatives and estimating potential performance is determining appropriate criteria that will yield realistic results and still provide a sufficient level of conservatism to ensure adequate treatment during abnormal conditions. Because the two alternatives being considered are secondary treatment alternatives, primary effluent characteristics need to be estimated to effectively evaluate the proposed process modifications.

Table 4.3-1 is a summary of the expected average flow quantity and strengths of primary effluent for the four different design scenarios established in Section 2.3. These values are simply an adjustment of the Table 2.3-1 values, assuming typical BOD_5 and TSS removal

efficiencies in the primary process of 30-35% and 45-50%, respectively. (Example: Primary Effluent, Current-Average Annual BOD_5 = 362 mg/l – (362 mg/l x 0.34) = 240 mg/l.)

Table 4.3-1 Primary Effluent Projections

Scenario	Flow, MGD	BOD_5, mg/l	TSS, mg/l
Current-Average Annual	1.14	240	190
Current – Peak Season	1.25	280	230
Future-Average Annual	1.38	240	190
Future-Peak Season	1.50	280	230

4.4 Flow Equalization of Primary Effluent

This section provides an evaluation of implementing flow equalization of primary effluent at the WWTP. The evaluation includes hydraulic calculations for determining the size and effects of a flow equalization basin, a preliminary layout and discussion of the various components of the process, a general description of system operations, a budget level cost estimate, and a brief summary of the pros and cons of flow equalization.

Hydraulic Calculations and Mass Loading Effects

A mass diagram was created for each of the four scenarios to determine the size requirements for the equalization basin. The data used in this analysis was based on the diurnal fluctuations of flow and wastewater strength observed in the recent sampling program, adjusted to match the four different design scenarios. This adjustment was accomplished by increasing the hourly flow and concentration values observed in sampling so that the daily averages equal the values in Table 4.3-1 – Primary Effluent Projections.

The results for sizing the basin under the different scenarios are summarized in Table 4.4-1 and the detailed mass diagrams and calculations are included in Appendix E.

Table 4.4-1 Flow Equalization Storage Requirements

Condition	Flow Equalization Basin Requirements		
	Storage	Usable Depth	Diameter
	($FT^{3)}$	(FT)	(FT)
Current-Average	24,600	10	56.0
Current-Peak Season	26,900	10	58.5
Future-Average	29,600	10	61.5
Future-Peak Season	32,300	10	64.0

The effects of installing an equalization basin are graphically shown in Figures 4.4-1 through 4.4-4 and are tabulated in Tables 4.4-2 through 4.4-5. To summarize, the daily fluctuations of the maximum hourly solids loading to the minimum hourly solids loading can theoretically be reduced from 4.62 to 1.12 for BOD_5, and from 5.47 to 1.24 for TSS. This dampening of the peaks stabilizes the maximum hourly solids loadings on the existing clarifier, thereby increasing the potential for improved consistency and settleability of the suspended solids. By keeping the suspended solids concentration nearly constant, Operations staff can more easily maintain the settling characteristics of the solids, thereby reducing the potential for plant upsets.

Within the existing trickling filters themselves, flow equalization would not result in a significant effect on the removal of BOD_5 and TSS. In general, trickling filters convert the soluble portion of BOD_5 into a settleable fixed-film biological growth. The efficiency of this conversion process varies in proportion to the BOD_5 loading rate. (See Appendix J) During a low BOD_5 loading rate (such as during the early morning hours of the day), the removal

efficiency can be as high as 90%. Conversely, during a high loading rate (such as around noon), the removal efficiency can be as low as 70%. Therefore, over the course of a day, the removal efficiency of the average daily solids loading rate with the normal diurnal flow patterns (without flow equalization) will be approximately the same as a steady state loading rate (with flow equalization.)

The potential for increased removal efficiencies due to flow equalization will occur in the solids contact and secondary clarifier facilities. While reference material discussing flow equalization suggests improved removal efficiencies for BOD_5 and TSS in downstream processes, none provide a definitive estimate of how much. This is most likely due to the variability of solids characteristics from plant to plant, from process to process, and from the ability of Operations staff to adjust to daily flow fluctuations. However, reference material does suggest that because prolonged aeration is required to keep the wastewater in an aerobic state while in the equalization basin, a reduction in BOD_5 is somewhat likely to occur as a result of exposing the BOD_5 to additional oxygen; studies suggest that a 10 to 20% reduction would likely be an <u>upper</u> limit for an in-line equalization basin of <u>raw</u> wastewater. (WEF/ASCE, 1998). Therefore, equalized primary-effluent solids loading to the downstream clarifier in conjunction with prolonged aeration should reasonably increase BOD_5 removal efficiencies between 5 to 10%, thereby decreasing secondary effluent concentrations. For the last 3-years, average effluent concentrations for BOD_5 were 62 mg/l; a 5 to 10% improvement in efficiency would lower effluent BOD_5 concentrations to a range of approximately 38 to 50 mg/l. (5% more removal of primary effluent BOD_5 of 240 mg/l = 12 mg/l; therefore, 62 mg/l – 12 mg/l = 50 mg/l; 10% more removal from 240 mg/l = 24 mg/l; therefore, 62 mg/l – 24 mg/l = 38 mg/l.)

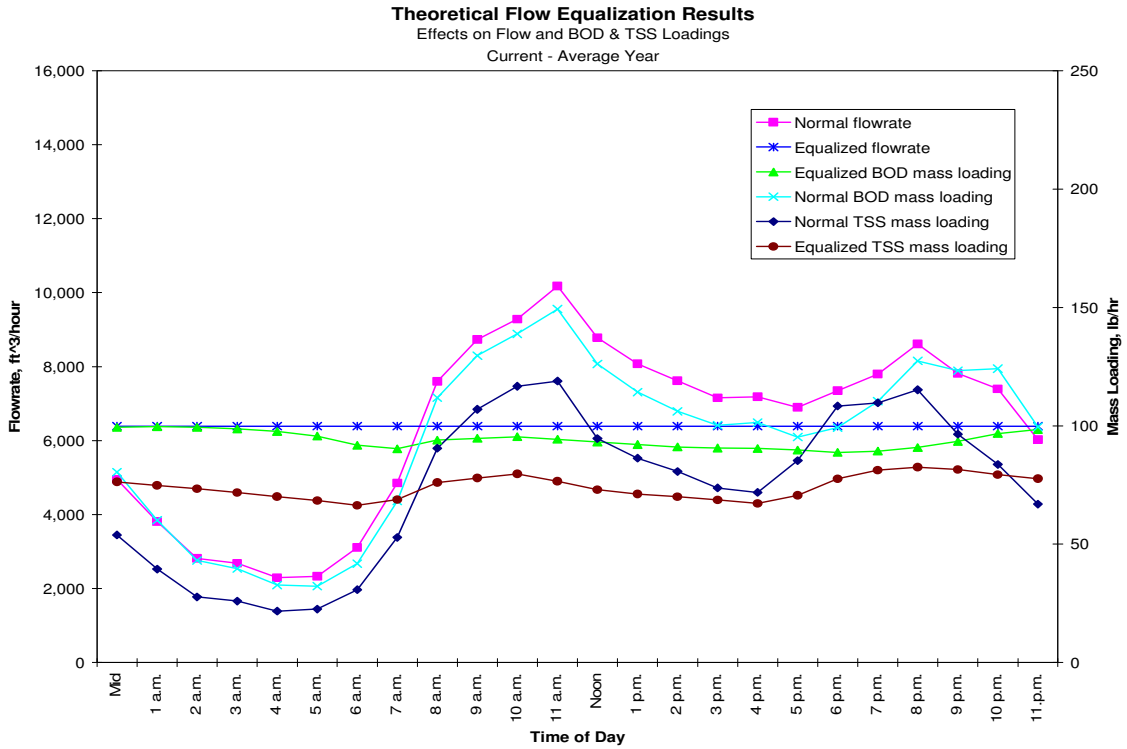

Figure 4.4-1 – Effects of Flow Equalization – Current Average Flow of 1.14 MGD

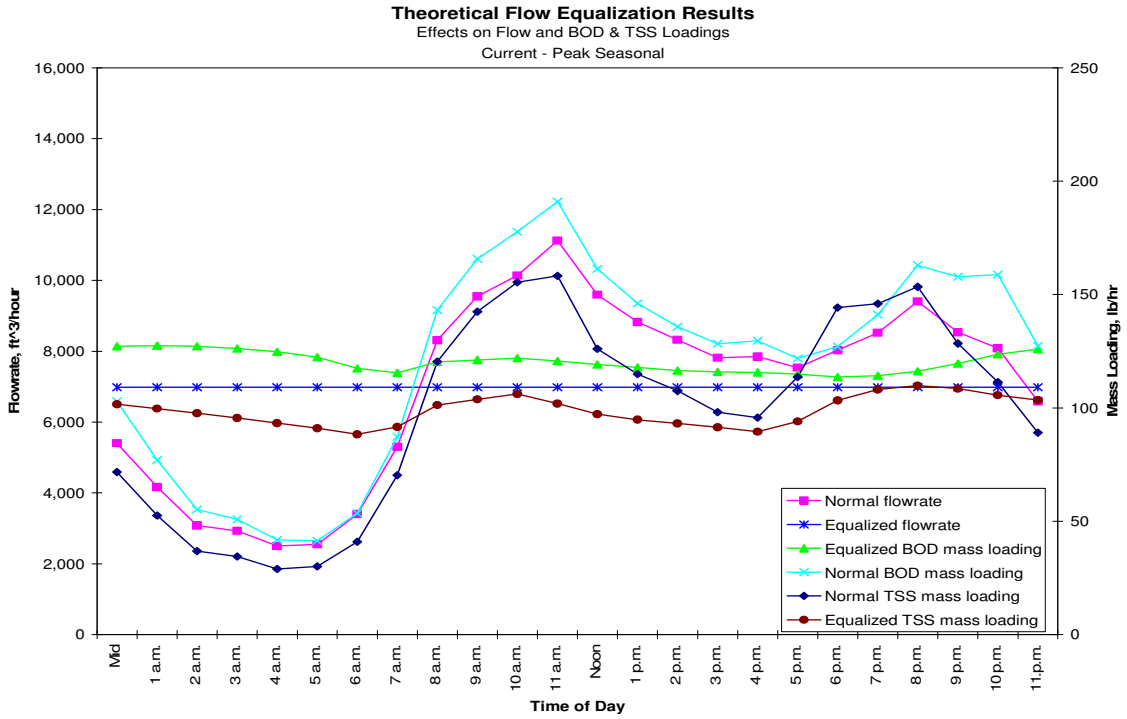

Figure 4.4-2 – Effects of Flow Equalization – Current Peak Season – Flow of 1.20 MGD

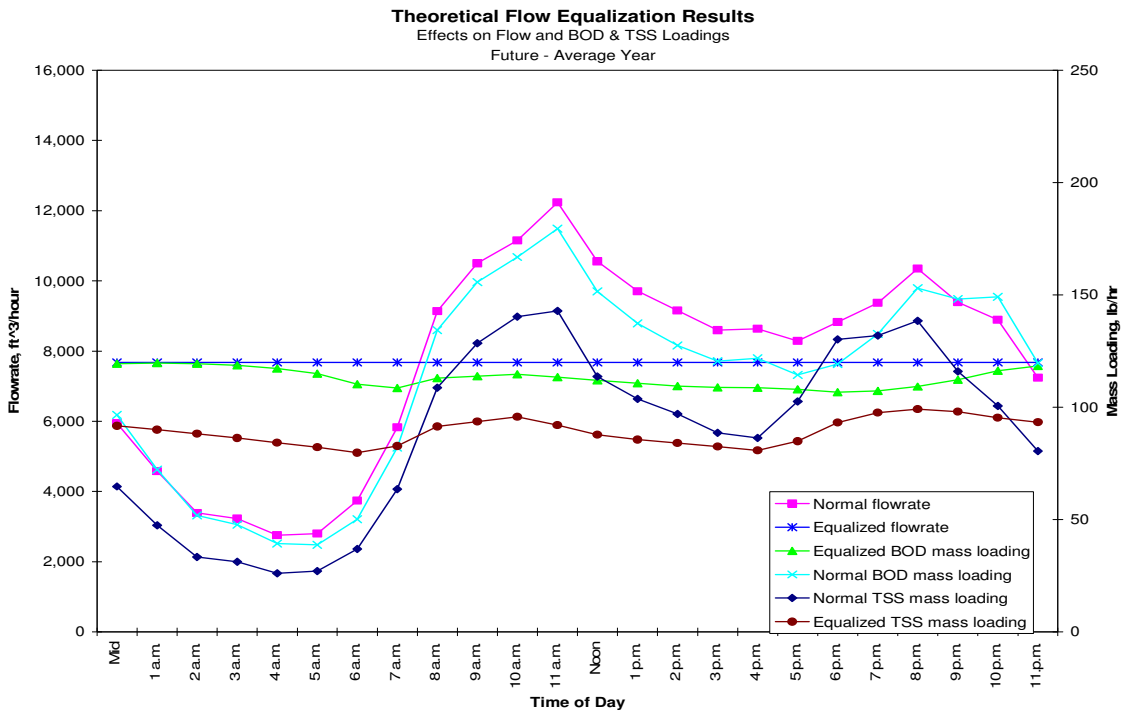

Figure 4.4-3 – Effects of Flow Equalization – Future Average – Flow of 1.38 MGD

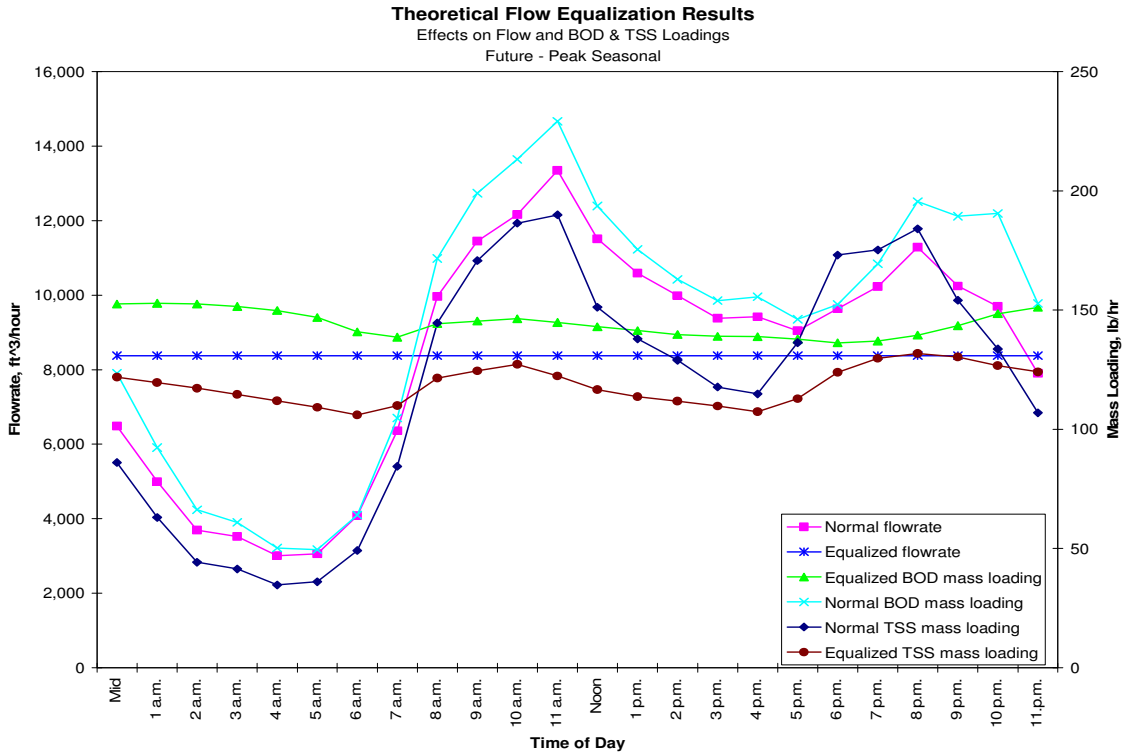

Figure 4.4-4 – Effects of Flow Equalization – Future Peak Season – Flow of 1.50 MGD

Table 4.4-2 Effects of Flow Equalization - Current-Average Flow of 1.14 MGD

Ratios	BOD_5 Mass Loading				TSS Mass Loading			
	Unequalized		Equalized		Unequalized		Equalized	
	lbs/hr	Ratio	lbs/hr	Ratio	lbs/hr	Ratio	lbs/hr	Ratio
Peak	149	1.59	100	1.06	119	1.60	83	1.11
Average	94		94		74		74	
Minimum	32	0.34	89	0.94	22	0.29	66	0.89
Average	94		94		74		74	
Peak	149	4.62	100	1.12	119	5.47	83	1.24
Minimum	32		89		22		66	

Table 4.4-3 Effects of Flow Equalization - Current-Peak Season Flow of 1.25 MGD

Ratios	BOD_5 Mass Loading				TSS Mass Loading			
	Unequalized		Equalized		Unequalized		Equalized	
	lbs/hr	Ratio	lbs/hr	Ratio	lbs/hr	Ratio	lbs/hr	Ratio
Peak	191	1.59	127	1.06	158	1.60	110	1.11
Average	120		120		99		99	
Minimum	41	0.34	114	0.94	29	0.29	88	0.89
Average	120		120		99		99	
Peak	191	4.62	127	1.12	158	5.47	110	1.24
Minimum	41		114		29		88	

Table 4.4-4 Effects of Flow Equalization - Future-Average Flow of 1.38 MGD

Ratios	BOD_5 Mass Loading				TSS Mass Loading			
	Unequalized		Equalized		Unequalized		Equalized	
	lbs/hr	Ratio	lbs/hr	Ratio	lbs/hr	Ratio	lbs/hr	Ratio
Peak	179	1.59	120	1.06	143	1.60	99	1.11
Average	113		113		89		89	
Minimum	39	0.34	107	0.94	26	0.29	80	0.89
Average	113		113		89		89	
Peak	179	4.62	120	1.12	143	5.47	99	1.24
Minimum	39		107		26		80	

Table 4.4-5 Effects of Flow Equalization – Future Peak Season Flow of 1.50 MGD

Ratios	BOD_5 Mass Loading				TSS Mass Loading			
	Unequalized		Equalized		Unequalized		Equalized	
	lbs/hr	Ratio	lbs/hr	Ratio	lbs/hr	Ratio	lbs/hr	Ratio
Peak	229	1.59	153	1.06	189	1.60	132	1.11
Average	144		144		118		119	
Minimum	49	0.34	136	0.94	35	0.29	106	0.89
Average	144		144		118		119	
Peak	229	4.62	153	1.12	189	5.47	132	1.24
Minimum	49		136		35		106	

Preliminary Layout

The most practical location at the WWTP for a 64' diameter flow equalization basin (FEB) would be as shown in Figure 4.4-5. The top and bottom elevation of the FEB would be set to accommodate gravity-flow from the primary clarifiers via new underground piping. After being equalized, a nearly constant flow could either be pumped to the existing Interstage or could flow by gravity with a modulating flow-control valve/gate.

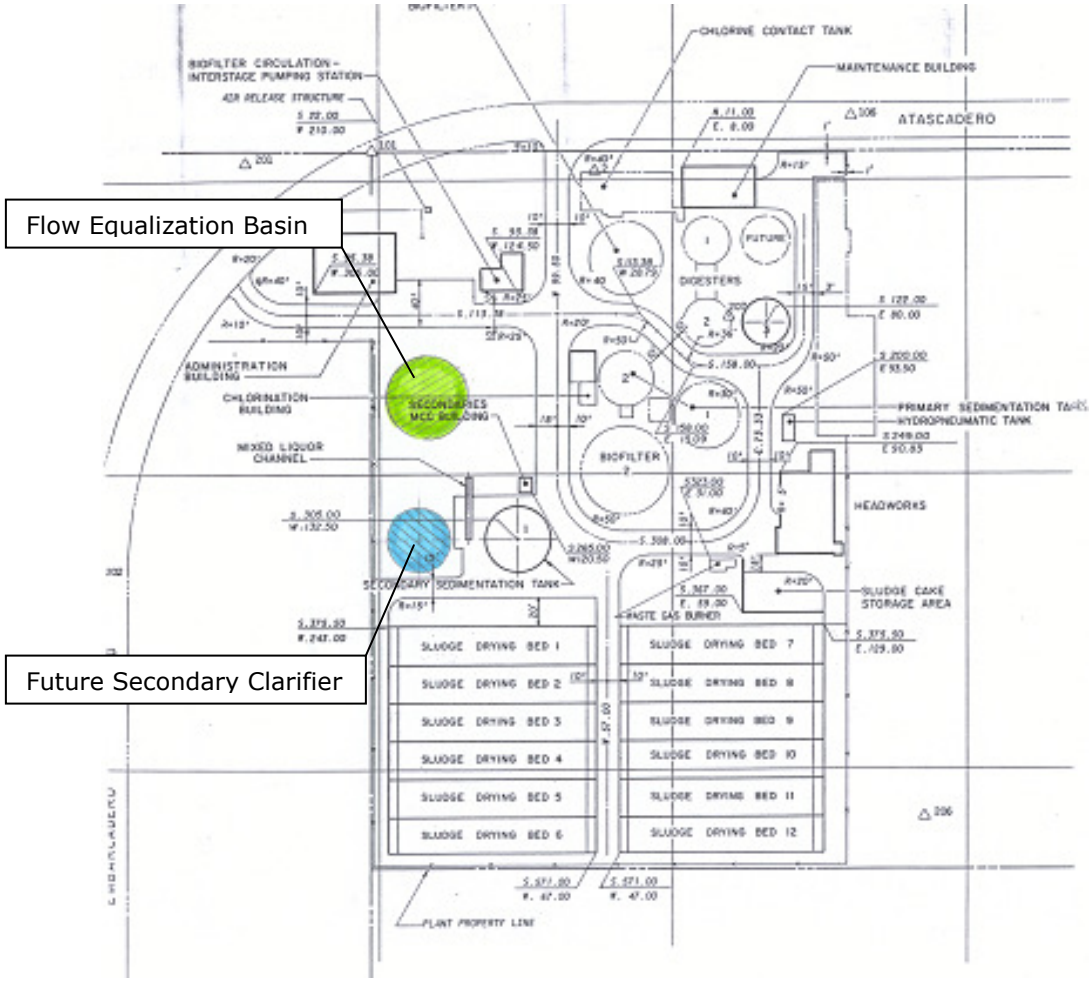

Figure 4.4-5 Preliminary Flow Equalization Basin Layout

Operations Description

As mentioned above, primary effluent would flow by gravity from the primaries directly into the in-line flow equalization basin. When the flow rate coming into the basin is greater than the daily average flow rate, typically from 8:00 a.m. until 10:00 p.m., the basin would begin filling. From 10:00 p.m. until 8:00 a.m., the flow rate coming into the basin is less than the daily average, and the basin would be draining. A constant speed pump station or a modulating valve/gate at the outlet of the equalization basin would allow a near constant discharge to the Interstage and subsequent downstream unit processes.

Because of the nearly constant flow-rate, the existing trickling filter recirculation pump station and the existing secondary clarifier pump station would most likely be adequate as currently installed, as they can easily handle the projected average daily flowrates, peak-season or average-annual.

Ancillary Facilities

In addition to a tank and a modulating outlet-valve/gate, there are several components associated with flow equalization of primary effluent: mixing equipment, solids and scum removal and pumping, and odor control. Mixing equipment is needed to keep oxygen in the system and to stabilize the organics in the wastewater to maintain aerobic conditions and to reduce the potential for odor problems. This mixing equipment would most likely consist of a diffused air system or jet mixers that would supply oxygen to the organics and keep the solids in suspension. Both solids and scum mechanisms may also be desired to keep the basin from going septic (due to solids deposition) thereby generating odor problems. Odor control equipment such as a cover and scrubber may also be desired to minimize off-gases released from prolonged aeration.

Approximate Costs

Planning level costs for constructing a flow equalization basin and its ancillary components are shown below.

System Component	**Costs**
Equalization Basin and Mechanical Equipment	$200,000
Constant Speed Pump Station/Modulating Flow-Control Gate	$50,000
Aeration System/Jet Mixers	$35,000
Yard Piping	$35,000
Odor Control Equipment	$50,000
Subtotal	$370,000
Estimating Contingency (20%)	$74,000
Construction Contingency (15%)	$56,000
Total Construction Costs	$500,000
Administration, Legal, Planning, Design, & Construction Management (30%)	$150,000
Total Project Costs	**$650,000**

General Discussion

The benefits for constructing a primary-effluent flow equalization basin include: stabilized flow and organic loadings on the trickling filters and secondary clarifier; increased operability of handling solids in the secondary solids contact channel and clarifier; decreased potential for secondary clarifier process upsets; increased potential for greater settling/removal efficiencies within the clarifier; and decreased secondary clarifier pump station and ocean outfall capacity requirements. As shown above, installation of a 64' diameter basin will equalize flows and loadings on the downstream processes. This equalization of loadings coupled with prolonged aeration should reasonably lower effluent BOD_5 concentrations between 5 to 10%.

Prior to performing additional design effort towards implementation of flow equalization, a more detailed evaluation of the solids contact/secondary clarifier operation should be performed to determine the extent of actual solids removal increases and the impacts to the solids handling facilities.

4.5 Trickling Filter Media Replacement

This section provides an evaluation of the effects of replacing the existing stone media with plastic media. The evaluation includes a review of manufacturer's product information, discussions with the manufacturer's technical staff regarding applicability of their products to the WWTP, preliminary performance projections, and preliminary cost estimates.

Trickling Filter Plastic Media

There are basically two types of plastic trickling filter media currently being used in the industry: random-dump media and structured plastic media. A detailed comparison of random-dump vs. structured plastic media is beyond the scope of this study; however, the following discussion provides basic information about each type along with certain design parameters used in estimating performance.

The term "random-dump media" generally refers to media that is spherical/cylindrical in shape with an internal web type structure that is designed to maximize surface area. Random-dump media is typically manufactured from high density polyethylene (HDPE) or polypropylene (PP). The specific surface area typically ranges from 25 to 50 square-feet/cubic-ft with void space around 95%. As a comparison, the specific surface area for rock media typically ranges from 15 to 20 square-feet/cubic-ft with a void space of only 50 to 60%. (WEF/ASCE, 1998)

Figure 4.5-1 – NSW Bio-Pack-SF#30 Random-Dump Media

Structured plastic media generally consists of thin layers of plastic sheeting (PVC) interconnected (by welding or gluing) into standard size modules, typically 2' x 2' x 4'. The two most common module types are cross-flow and vertical-flow media, as shown below.

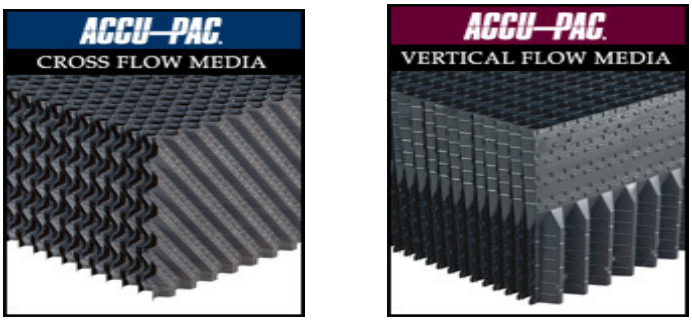

Figure 4.5-2 – Brentwood Industries Structured Plastic Media

The cross-flow media provides a more uniform distribution across the media and offers better performance than the vertical-flow, but under heavy solids loading, tends to be more susceptible to plugging than the vertical-flow. The specific surface area for structured

plastic media (used for soluble carbonaceous BOD_5 removal) typically ranges from 27 to 32 square-feet/cubic-ft with void space around 95% (WEF/ASCE, 1998).

Preliminary Performance Projections

The two major USA manufacturers of trickling filter plastic media, Brentwood Industries and NSW Environmental Systems, were contacted to gather information regarding the applicability of their products for the WWTP. Both companies were given basic design criteria about the plant including the projected influent flows and wastewater strengths for the four planning conditions. Both companies were also asked to provide an initial analysis of whether the existing trickling filters could be retrofitted with their media to provide a significant improvement in the overall plant effluent quality.

Initially, NSW provided an estimate of the BOD_5 removal efficiency and effluent concentration if the existing rock media was simply replaced with their random-dump media at the same depth of 5-ft. Two scenarios were modeled, current peak-season and future peak-season, as follows.

1. Current peak-season influent flow of 1.25 mgd, influent BOD_5 concentration of 393 mg/l, primary effluent BOD_5 concentration of 275 mg/l, available trickling filter volume for media of 33,300 cubic-feet, yielding an average daily BOD_5 loading of 86 lbs/1,000 ft^3/d. From this, their model estimated a removal efficiency of approximately 80% and an average effluent concentration of 55 mg/l.

2. Future peak-season influent flow of 1.5 mgd, influent BOD_5 concentration of 393 mg/l, primary effluent BOD_5 concentration of 275 mg/l, available trickling filter volume for media of 33,300 cubic-feet, yielded an average daily BOD_5 loading of 104 lbs/1,000 ft^3/d. From this, their model estimated a removal efficiency of approximately 77% and an average effluent concentration of 65 mg/l.

These same hypothetical loadings when applied to a shallow rock media filter would yield removal efficiencies of approximately 72% and 68% and average effluent concentrations of approximately 77 mg/l and 88 mg/l, respectively (Hammer, 1986). Theoretically, replacing the existing rock media with random-dump plastic media has the potential to moderately increase removal efficiencies (8 to 9%). However, actual removal efficiencies from the existing rock media filters are slightly higher than the theoretical, making the potential increase more realistically 3 to 5%. In addition, it should also be noted that research conducted by the Water Environment Federation suggests that plastic media (the cross-flow type), in the range of 3 to 8 feet deep, does not necessarily perform "better than or sometimes as good as" a rock media filter (WEF, 2000).

Given this result and the fact that removal efficiencies increase in proportion to filter media depth, both manufacturers were asked to use their models to determine the depth of media required to obtain BOD_5 removal efficiencies of 90%+ while maintaining the existing trickling filter configurations (locations and diameters.) NSW modeled their random-dump media (Bio-Pack-SF#30, specific surface area of 30 sf/cf) and Brentwood their cross-flow media (CF/S-3000, specific surface area of 31 sf/cf). Given the same wastewater loadings as before, both manufacturer's models yielded the same result: in order to obtain a removal efficiency of 90% or better without forced ventilation, the depth of the media needs to be increased to at least 12 feet. This equates to a media volume of approximately 80,000 cubic-feet for both media types and a resultant BOD_5 solids loading rate of 43 lbs/1,000 ft^3/d. From this, their models yielded a removal efficiency of approximately 90% and average effluent concentrations ranging from 21 to 28 mg/l. Modeling results are included in Appendix F.

Approximate Costs

Planning level costs for retrofitting the existing trickling filters from shallow rock media to random dump plastic media are estimated as shown below. (See Appendix G for more detail regarding media replacement costs.) For comparison purposes, two cost estimates were prepared, one for the replacement of media at the current depth (5-ft) and the other for 12-ft of media depth. Simply replacing the media at the same depth is less costly and easier to construct. Adding depth to the existing filters will require more extensive retrofitting. The Center-well and Distribution Arm Assembly will need to be raised to handle the additional filter height. The walls will need to be built-up (such as welded-steel ring, etc.) to support the media. (The random-dump media requires more lateral support than the cross-flow media; however, both require substantially less than the rock media). Adding depth to the filters may require a minor retrofitting of the re-circulation pump station due to an increase in the lift to the top of the filters. Finally, if the existing filters are to be reconstructed in their current location, construction activities will need to be closely coordinated during low-flow periods of the year with the possibility that there may be a need to set-up temporary treatment systems during construction.

System Component	**Costs**
Existing Rock and Under-drain Removal and Disposal	$40,000
Random-Dump/Cross Flow Media	$100,000
Under-drain System/Vent Piping/etc.	$30,000
Temporary Treatment Systems (during construction)	$30,000
Subtotal	$200,000
Estimating Contingency (20%)	$40,000
Construction Contingency (15%)	$30,000
Total Construction Costs	$270,000
Administration, Legal, Planning, Design, & Construction Management (30%)	$80,000
Total Project Costs (5-ft deep media)	**$350,000**

System Component	**Costs**
Existing Rock and Under-drain Removal and Disposal	$40,000
Center-well and Distribution Arm Modifications	$50,000
Trickling Filter Wall Modifications (Structural) (depends on detailed eval.)	$70,000
Random-Dump/Cross Flow Media	$250,000
Under-drain System/Vent Piping/etc.	$30,000
Recirculation Pump Station Modifications (increased TDH requirements)	$20,000
Temporary Treatment Systems (during construction)	$40,000
Subtotal	$500,000
Estimating Contingency (20%)	$100,000
Construction Contingency (15%)	$75,000
Total Construction Costs	$675,000
Administration, Legal, Planning, Design, & Construction Management (30%)	$202,500
Total Project Costs (12-ft deep media)	**$880,000**

General Discussion

Upgrading the trickling filter media from rock to plastic and increasing the media height from 5 feet to 12 feet is feasible and has the potential to substantially improve effluent water quality by increasing BOD_5 removal efficiencies. With this increase, additional solids loadings to the downstream clarifier and solids handling facilities can be expected and must be accounted for. The additional solids loadings on the downstream clarifier are generated from the sloughing-off of fixed film biological growth as it converts more soluble BOD_5 into settleable solids. In order to treat these additional solids, a larger population of microorganisms is needed in the clarification process. These microorganisms when mixed with the effluent stream from the trickling filters (measured in terms of Mixed Liquor Suspended Solids, MLSS, concentrations, mg/l), constitute the solids loadings on the clarifier. Based on information obtained from Operations staff regarding the settling characteristics of the existing solids, the following analysis was conducted to determine whether or not the existing clarifier can handle additional loads.

A brief comparison of Current (Peak Season) and Future (Peak Season) vs. Typical Design ranges for each of these criteria is shown below. The major design criteria for determining the impacts of these additional solids loadings on the clarifier are: hydraulic loading rate (gal/day/square-feet of clarifier area), hydraulic retention time (hours), solids loading rate (lb Mixed-Liquor-Suspended-Solids/day/square-feet of clarifier area), and solids settleability (expressed as Sludge Volume Index (SVI), ml/g.) The typical design ranges and values shown for the various parameters were taken from several references: WEF/ASCE 2000, Hammer 1986, and the Vancouver Study.

Current Conditions (Peak Season)

Average Daily Dry Weather Flow (ADWF) = 1.25 MGD
Average Daily Peak Hourly Flow (APHF) = 2.5 MGD
Average Primary Effluent BOD_5 concentration, mg/l = 171 mg/l
Mixed Liquor Suspended Solids Concentration (MLSS) = 3,000 mg/l*
Clarifier Surface Area = 2,347 ft^2
Sludge Volume Index (SVI) = 35 to 50 ml/g (typical values per Operations staff)
*Value to increase as primary influent concentration increases.

Parameter	Units	Future Conditions	Typical Design Range
Hydraulic Loading @ ADWF	gal/day/sf	532	400 - 800
Hydraulic Loading @ APHF	gal/day/sf	1,065	<1,200
Hydraulic Retention Time @ ADWF	hours	5	2 to 3
Hydraulic Retention Time @ APHF	hours	2.5	2 to 3
Solids Loading @ ADWF	lbs of MLSS/day/sf	13.5	<15
Solids Loading @ APHF	lbs of MLSS/day/sf	27	<30
SVI	ml/gram	35 - 50	< 135

Future Conditions (Peak Season)

Average Daily Dry Weather Flow (ADWF) = 1.5 MGD
Average Daily Peak Hourly Flow (APHF) = 3.0 MGD
Average Primary Effluent BOD_5 concentration, mg/l = 171 mg/l
Mixed Liquor Suspended Solids Concentration (MLSS) = 3,600 mg/l*
Clarifier Surface Area = 2,347 ft^2
Sludge Volume Index (SVI) = 50 to 100 ml/g
*Value to increase as primary influent concentration increases.

Parameter	Units	Future Conditions	Typical Design Range
Hydraulic Loading @ ADWF	gal/day/sf	640	400 - 800
Hydraulic Loading @ APHF	gal/day/sf	1,280	<1,200
Hydraulic Retention Time @ ADWF	hours	4.2	2 to 3
Hydraulic Retention Time @ APHF	hours	2.1	2 to 3
Solids Loading @ ADWF	lbs of MLSS/day/sf	19.2	<15
Solids Loading @ APHF	lbs of MLSS/day/sf	38.4	<30
SVI	ml/gram	50 - 100	< 135

Theoretically, under current peak season conditions and current primary effluent strengths, the existing clarifier will operate within typical design ranges; whereas, under future peak season conditions, the existing clarifier will be either at the upper limits or slightly exceed the typical design criteria. The limiting factor will most likely be the MLSS solids loading concentrations and its effect on the settleability of the sludge. As wastewater-flow and strength from the primaries increases and/or trickling filter soluble BOD_5 removal efficiency increases, the MLSS concentration will need to increase in order to handle the solids loadings. This in turn may change the settling characteristics of the solids, resulting in higher SVI values and the possibility of losing solids over the clarifier weirs.

Prior to initiating the design of trickling filter modifications, additional analysis should be performed to determine the effects that increases in MLSS concentrations will have on the existing clarifier and the solids handling facilities (WAS pumping, digesters, drying beds, etc.)

5.0 Conclusions and Recommendations

Due to the MBCSD's desire to continually investigate and evaluate opportunities to increase effluent water quality from the WWTP, this study was commissioned to conduct research into the viability of two process modifications: flow equalization of primary effluent, and trickling filter media replacement. As part of this effort, influent and effluent data was reviewed to estimate future wastewater influent characteristics (both quantitative and qualitative.) Extensive diurnal wastewater quality information was collected and analyzed at key locations within the plant to assist in evaluating process performance of existing and proposed processes. The process piping between the primary and secondary facilities was analyzed to determine flow capacities and limitations of routing additional flow through the secondary process. The following conclusions were derived from the research and analysis of the two process modifications.

1.) Influent Wastewater Flow to the WWTP has averaged 25% less in the past three years than in the past 10-years, 1.14 MGD vs. 1.52 MGD. This trend is most likely attributable to repairs of the influent trunk sewer in Atascadero Road in 1998 and 2000 (resulting in less infiltration) and the installation of a more accurate influent flow meter in August of 2001. Influent Wastewater flows are projected to increase from current conditions to future build-out conditions, 1.14 MGD to 1.38 MGD for average annual, and 1.25 MGD to 1.50 MGD for peak season, respectively.

2.) Influent BOD_5 and TSS concentrations to the WWTP are 30% higher in the last three years then when the secondary process was originally designed, 362 mg/l and 378 mg/l vs. 280 mg/l and 280 mg/l, respectively. This trend is most likely attributable to water conservation and inflow/infiltration reduction efforts. Basically, there is less water to transport the same solids per capita loadings resulting in increases in influent concentrations.

3.) The WWTP is operating well within the limitations of the current Modified Discharge Permit in almost all instances as Operations staff continue to enhance effluent water quality by maximizing the amount of flow through the secondary process.

4.) Construction of a 64' diameter primary-effluent flow equalization basin between the primary clarifiers and the Interstage has several benefits: stabilized flow and organic loadings on the trickling filters and secondary clarifier; increased operability of handling solids in the secondary solids contact channel and clarifier; decreased potential for secondary clarifier process upsets; increased settling/removal efficiencies within the clarifier; and decreased secondary clarifier pump station and ocean outfall capacity requirements.

5.) Implementing flow equalization without upgrading the trickling filters would most likely result in the operational benefits described above, but there would not be a significant increase in the quantity of the soluble BOD_5 removed. Even though the total BOD_5 and TSS mass loadings are significantly damped by the basin, the amount of soluble BOD_5 removed by the existing rock media trickling filters would not decrease significantly and would continue to pass through the secondary clarifier.

6.) Upgrading the trickling filter media from rock to plastic and increasing the media height from 5-ft to 12-ft has the potential to substantially improve effluent water quality by increasing soluble BOD_5 removal efficiencies. However, with this increase, additional solids loadings to the downstream clarifier and solids handling facilities can be expected and must be accounted for. The additional solids loadings on the

downstream clarifier are generated from the sloughing-off of fixed film biological growth as it converts the soluble BOD_5 into settleable solids. These settleable solids must then be removed in the clarifier.

7.) Implementing the trickling filter media upgrade without flow equalization would increase the removal of the soluble BOD_5 but would also increase peak solids loading on the downstream clarifier. The trickling filter media upgrade in conjunction with flow equalization would result in an increase in soluble BOD_5 removal and a nearly constant solids loading rate; however, the solids loading rate on the downstream clarifier may be too much to maintain a consistent, high-quality effluent under future conditions. A more detailed evaluation of the solids contact/secondary clarifier operation should be performed to determine the extent to which actual solids can be removed.

8.) While both alternatives are considered feasible and would enhance effluent water quality, they should not be considered stand-alone solutions to meeting more stringent long-term water quality requirements; rather, they should be viewed as realistic components of a more comprehensive plant upgrade. Even though WWTP influent flows have been significantly reduced in recent years, the increases in influent BOD_5 and TSS are significant enough to warrant concern over the ability of the secondary process (and solids handling facilities) to handle future solids loadings of the entire influent flow.

The following recommendations are provided to assist MBCSD in implementing a program to continue to enhance effluent water quality from the treatment plant.

<u>Primary Recommendations</u>

1.) The WWTP, as currently configured, is meeting its NPDES permit discharge requirements; however, if permit requirements become more stringent, MBCSD will most likely need to increase the scope of the expansion to match the discharge requirements. Prior to investing significant funds in the implementation of flow equalization and trickling filter modifications, a more comprehensive capacity evaluation of the entire WWTP (both liquid and solids streams) should be conducted based on potentially more stringent effluent discharge requirements. From this evaluation, a long-term WWTP Master Plan followed by a phased capital improvement program should be developed to proactively implement strategic process modifications/upgrades in anticipation of these potentially more stringent effluent water quality requirements.

2.) As part of the WWTP Master Plan effort, develop a more comprehensive understanding of the influent wastewater trends for both BOD and TSS. Determine why the trend in concentrations has decreased in the last 3-years. Should concentrations begin to rise again, develop a strategy for minimizing the loads at the source.

3.) Investigation into Enhanced Primary Treatment, also referred to as Chemical Coagulation, should be conducted. This consists of the addition of chemicals (usually ferric chloride and/or anionic polymer) prior to primary sedimentation to promote solids flocculation and increased solids settling characteristics. Primary clarification with coagulation may remove 60 to 90% of the total suspended solids (TSS) and 40 to 70% of the BOD_5 (Metcalf & Eddy, 1991). Given that existing removal efficiencies for TSS and BOD_5 at the WWTP are approximately 60% and 30%, respectively, enhanced primary treatment has the potential to significantly improve effluent water

quality. Prior to implementation, a detailed evaluation should be conducted to determine the effects on downstream solids-handling processes.

4.) Further investigation into the hydraulic restriction in the piping between the primary clarifiers and the Interstage facility should be performed. One possible solution would be to increase the size of the sluice gate and wall-opening at the Interstage structure. This modification would provide more hydraulic-gradient between the facilities and would increase the flow-carrying capacity. A review of downstream impacts should be conducted prior to implementation.

Secondary Recommendations

1.) Increase the frequency of sampling and data collection between the different unit processes. More analysis should be conducted to develop a better understanding of the relationship between TSS, Total BOD_5, and soluble BOD_5 at the various stages of treatment within the plant. This information should be recorded in electronic format to provide easy retrieval for future studies, evaluations, and designs.

2.) Upgrade and/or modify the existing in-plant process flowmeters in order to be able to record and store data for future studies, evaluations, and designs.

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Appendix A

Existing WWTP Design Criteria

DESIGN DATA

PLANT CAPACITY

Flow, mgd	
Average dry weather flow (ADWF)	2.06
Peak seasonal dry weather flow (PSDWF)	2.36
Peak dry weather flow (PDWF)	6.64
Peak wet weather flow (PWWF)	6.60
Strength	
Biochemical oxygen demand (BOD_5), mg/l	280
Suspended solids (SS), mg/l	280

PRELIMINARY TREATMENT

Mechanically cleaned bar screen	
Number	1
Capacity, mgd	8.2
Manually cleaned bar screen	
Number	1
Capacity, mgd	8.2
Influent pumps (variable speed)	
Number	3
Capacity, each, mgd	3.3
Total head, feet	31.6
Aerated grit removal tank	
Length, feet	30
Width, feet	16
Depth, feet	8
Detention time at PWWF, minutes	6.3

PRIMARY TREATMENT

Sedimentation tanks (existing)	
Number	2
Diameter, feet	
Tank 1	50
Tank 2	40
Average side water depth, feet	9
Surface loading rate at PSDWF, gal/sq. ft./day	730

TRICKLING FILTERS (in partial secondary treatment mode of operation)

Flow Distribution at PSDWF, mgd	
Filter 1	0.39
Filter 2	0.58
Diameter, feet	
Filter 1	60
Filter 2	70
Net media surface area, sq. ft.	
Filter 1	2,820
Filter 2	3,770
Average media height, feet	
Filter 1	4.5
Filter 2	5.0
Specific organic loading rate, lb BOD_5/day/1,000 cu. ft. media	47
Hydraulic loading rate, gpm/sq. ft. media surface	
Filter 1	0.34
Filter 2	0.38

SOLIDS CONTACT CHANNEL

Channel length, feet	
Reaeration portion	25
Contact portion	25
Channel depth, feet	4
Recirculated sludge flow, gpm	
Minimum	100
Maximum	500
Interstage pumps	
Number	2
Capacity, each, gpm	2,300
Total head, feet	27.0

SECONDARY SEDIMENTATION TANK

Diameter, feet	55
Tank surface area, sq. ft.	2,376
Tank volume, cu. ft.	35,640
Average water depth, feet	15
Overflow rate at PSDWF gal/sq. ft./day	408

TOTAL TREATMENT

Overall treatment efficiencies, percent	
BOD_5 removal	57
SS removal	75
Expected effluent quality, mg/l	
BOD_5	120
SS	70

SOLIDS STABILIZATION

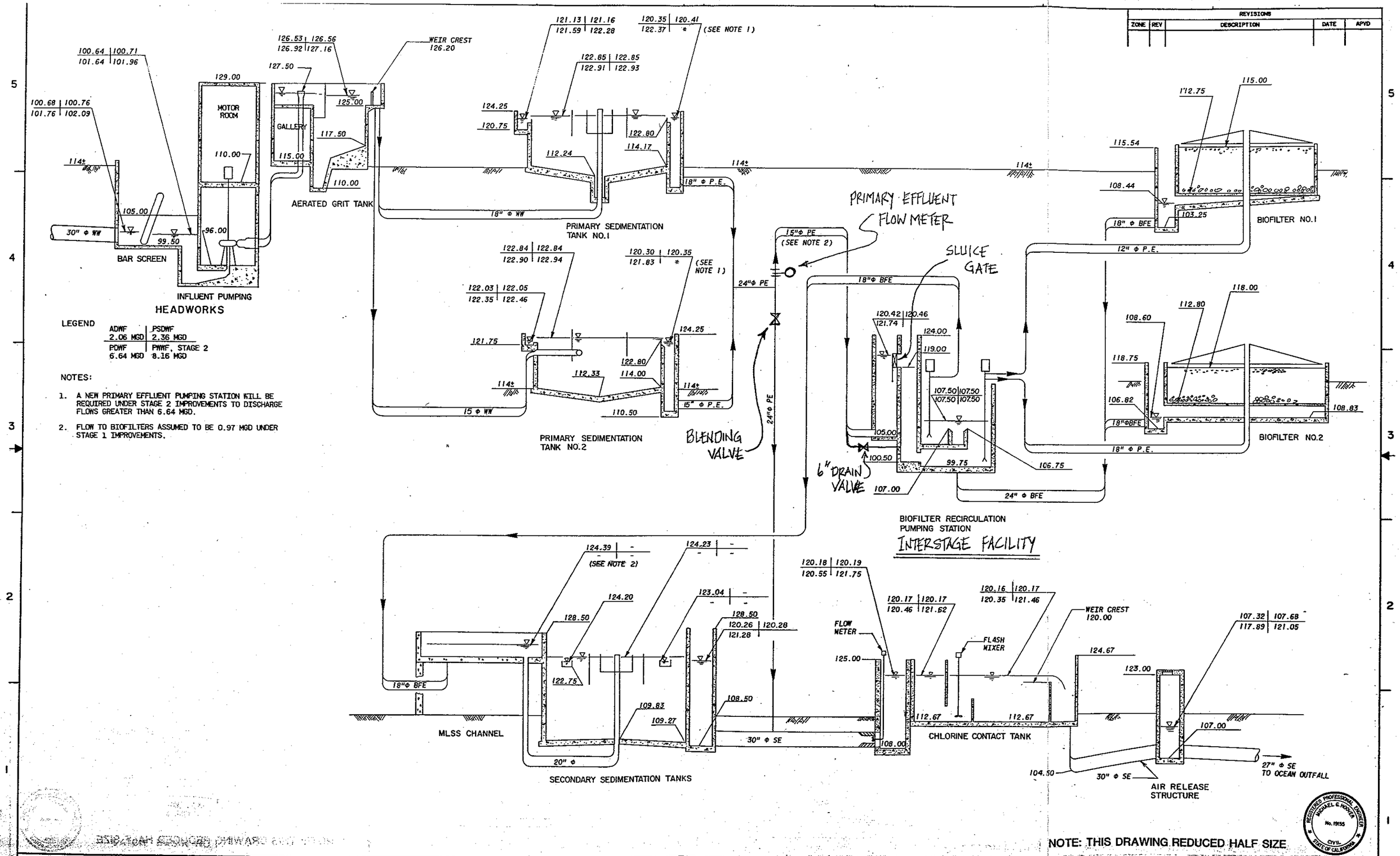
Anaerobic digesters	
Loading, 1,000 lb/day	
Primary solids	3.58
Secondary solids	0.88
Assumed sludge volatile content, percent	
Primary solids	70
Secondary solids	82
Sludge volume, 1,000 cu. ft./day	1.79
Digester 1 (fixed cover)	
Diameter, feet	40
Side water depth, feet	16
Volume, 1,000 cu. ft.	22.2
Digester 2 (fixed cover)	
Diameter, feet	40
Side water depth, feet	19
Volume, 1,000 cu. ft.	25.6
Digester 3 (floating cover)	
Diameter, feet	35
Side water depth, feet, average	22.5
Volume, 1,000 cu. ft.	22.8
Hydraulic detention time based on net volume of digesters 2 & 3, days	23
Expected sludge gas production 1,000 cu. ft./day	28.4
Sludge drying beds	
Number	12
Length, each, feet	162
Width, each, feet	32
Loadings, lb of solids/sq. ft./year	16

OCEAN OUTFALL

Length, feet	
Land portion	1,100
Ocean portion	4,060
Diameter, inches	27
Discharge depth, feet below MLLW	50
Multiport diffuser	
Ports open	29
Ports closed (future)	5
Port diameter, inches	2

Appendix B

Facility Layout and Hydraulic Profile



LEGEND

ADWF	PSDWF
2.06 MGD	2.36 MGD
PDWF	PWWF, STAGE 2
6.64 MGD	8.16 MGD

NOTES:

1. A NEW PRIMARY EFFLUENT PUMPING STATION WILL BE REQUIRED UNDER STAGE 2 IMPROVEMENTS TO DISCHARGE FLOWS GREATER THAN 6.64 MGD.
2. FLOW TO BIOFILTERS ASSUMED TO BE 0.97 MGD UNDER STAGE 1 IMPROVEMENTS.

REVISIONS

ZONE	REV	DESCRIPTION	DATE	APVD

NOTE: THIS DRAWING REDUCED HALF SIZE

REGISTERED PROFESSIONAL ENGINEER MICHAEL G. HOOVER No. 19155 CIVIL STATE OF CALIFORNIA

BROWN AND CALDWELL CONSULTING ENGINEERS
EUGENE • PASADENA • SEATTLE
TUCSON • WALNUT CREEK

DESIGNED BAP
CHECKED BAP
CHECKED LYY

SUBMITTED [signature] DATE
APPROVED [signature] DATE 10-7-82
APPROVED [signature] DATE 10-7-82

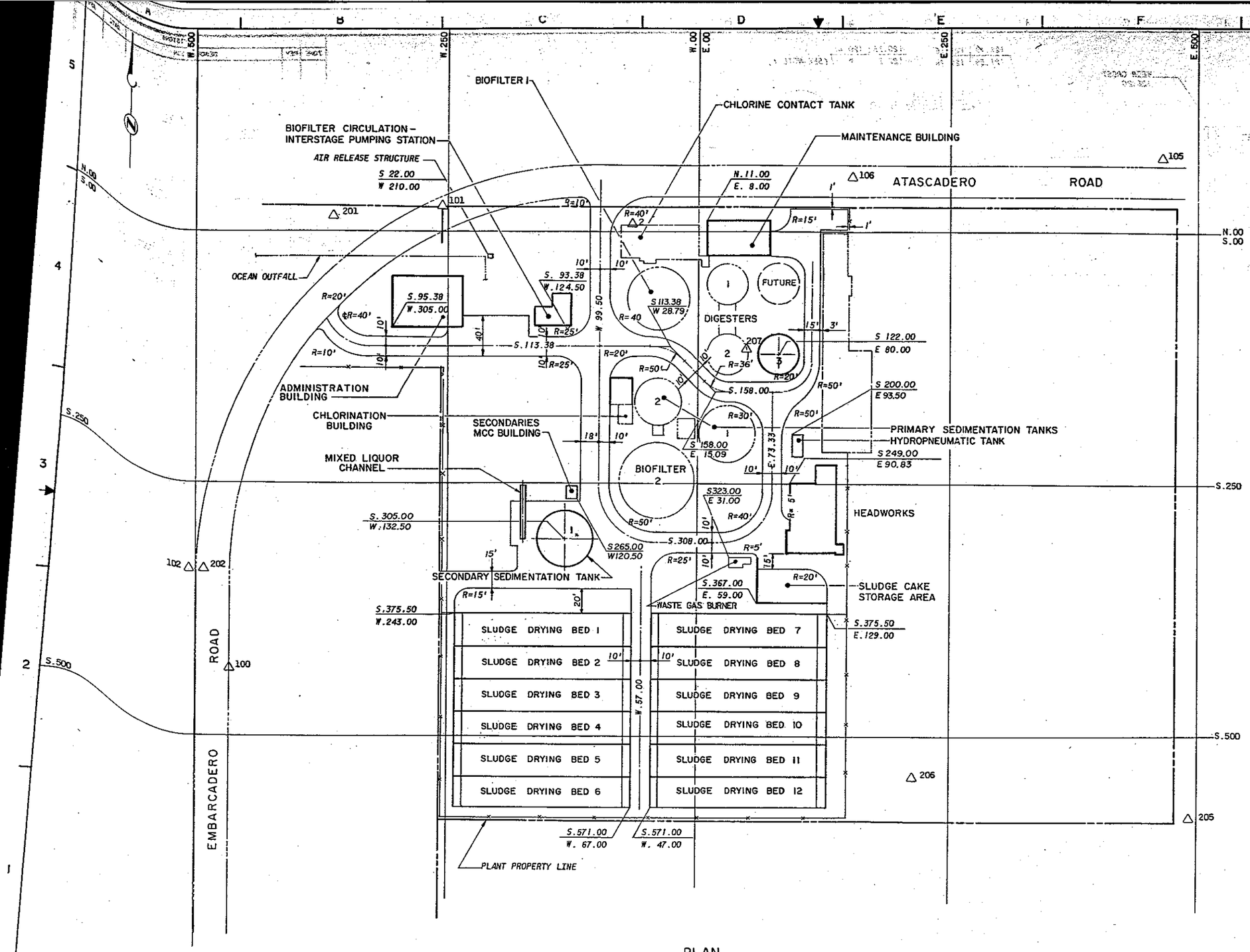
FILE 515
DATE SEPT '81

CITY OF MORRO BAY

IMPROVEMENTS TO MORRO BAY-CAYUCOS WWTP

HYDRAULIC PROFILE

DRAWING NUMBER G5
SHEET 5
D



PLAN

SCALE : 1" = 50'

SURVEY CONTROL DATA

PT. NO.	PLANT COORDINATES		ELEVATION	DESCRIPTION
2	N.004.61	W.065.01	24.72	BRASS CAP MON., TOP OF WALL, MARKED "RM 2"
100	S.434.99	W.456.85	-------	2" IRON PIPE, R.E. 12748
101	N.024 58	W.256.51	9.35(MON.) 13.91(RIM)	2" IRON PIPE IN WELL, "ATASCADERO"
102	S.334.92	W.506.97	-------	BRASS CAP MON. IN CONC. MARKED "M.B.C."
105	N.072.99	E.464.10	17.02	BRASS CAP MON.
106	N.052.93	E.152.24	15.20	BRASS CAP MON. IN PAVING MARKED "M.B.C." 1"
201	N.017.33	W.364.33	-------	1" IRON PIPE
202	S.334.95	W.491.97	19.05	SPIKE & WASHER
205	S.581.73	E.493.18	19.77	3" IRON PIPE
206	S.542.01	E.215.04	16.64	LEAD & TAG IN CONC. SLAB
207	S.119.74	E.048.03	-------	LEAD & TAG IN TOP OF CONC. TANK

BOUNDARIES.....SHOWN PER WASTE WATER TREATMENT PLANT PROPERTIES MAP DATED APRIL 23, 1980.

HORIZONTAL CONTROL.....ESTABLISHED FROM FOUND MONUMENTS AS SHOWN ON CALIFORNIA STATE LANDS COMMISSION (C.S.L.C.) MAP OF THE GRANT TO COUNTY OF SAN LUIS OBISPO SHEET 4 OF 5 DATED SEPTEMBER 1957 AND ON PARCEL MAP NO. MB-68-29 DATED APRIL 1968.
CALIFORNIA STATE PLANE COORDINATE SYSTEM ZONE V COORDINATES ESTABLISHED FROM VALUES SHOWN ON STATE LANDS COMMISSION MAP AND PARCEL MAP NO. MB-69-29.
LOCAL PLANT GRID COORDINATE SYSTEM ESTABLISHED FROM TIES TO EXISTING STRUCTURES AS SHOWN ON SEWER TREATMENT PLANT PLOT PLANS BY MENDENHALL, SHEET 3 OF 18 AND 4 OF 18 DATED SEPTEMBER AND OCTOBER 1953.

VERTICAL CONTROL.....STANDARD C.S.L.C. BRASS CAP SET IN A DRILL HOLE ON TOP OF THE CLARIFIER 12.5 FT. E. OF THE N.W. CORNER AND 0.9 FT. S. OF THE N. EDGE. STAMPED "ATASCADERO RM 2 1958." ELEV. 24.72 PER PLOT PLAN AND TOPOGRAPHY BY JOHN CAROLLO ENGINEERS SHEET 3 OF 38 DATED AUGUST 1964.
NOTE: 100' ADDED TO CONTOURS SHOWN HEREON.

NOTES:

1. SEE STRUCTURE DRAWINGS FOR WORK POINT COORDINATES OF STRUCTURES.
2. FOR OUTSIDE PIPING LOCATION AND ELEVATION, SEE DWG'S C7 THROUGH C9.

NOTE: THIS DRAWING REDUCED HALF SIZE

REGISTERED PROFESSIONAL ENGINEER MICHAEL G. HOOKER No. 19155 CIVIL STATE OF CALIFORNIA

BROWN AND CALDWELL CONSULTING ENGINEERS
PASADENA • SEATTLE
WALNUT CREEK
EUGENE • TUCSON

DESIGNED LY
CHECKED BAP
CHECKED RLM

SUBMITTED _____ DATE _____
APPROVED _____ (For Brown and Caldwell) DATE 10-7-82
APPROVED _____ (For the City of Morro Bay) DATE 10-7-82

FILE 515
DATE SEPT '81

CITY OF MORRO BAY

IMPROVEMENTS TO MORRO BAY-CAYUCOS WWTP

PLANT LAYOUT AND SURVEY CONTROL DATA

DRAWING NUMBER G 6
SHEET 6

Appendix C

Hydraulic Calculations

Cannon ASSOCIATES
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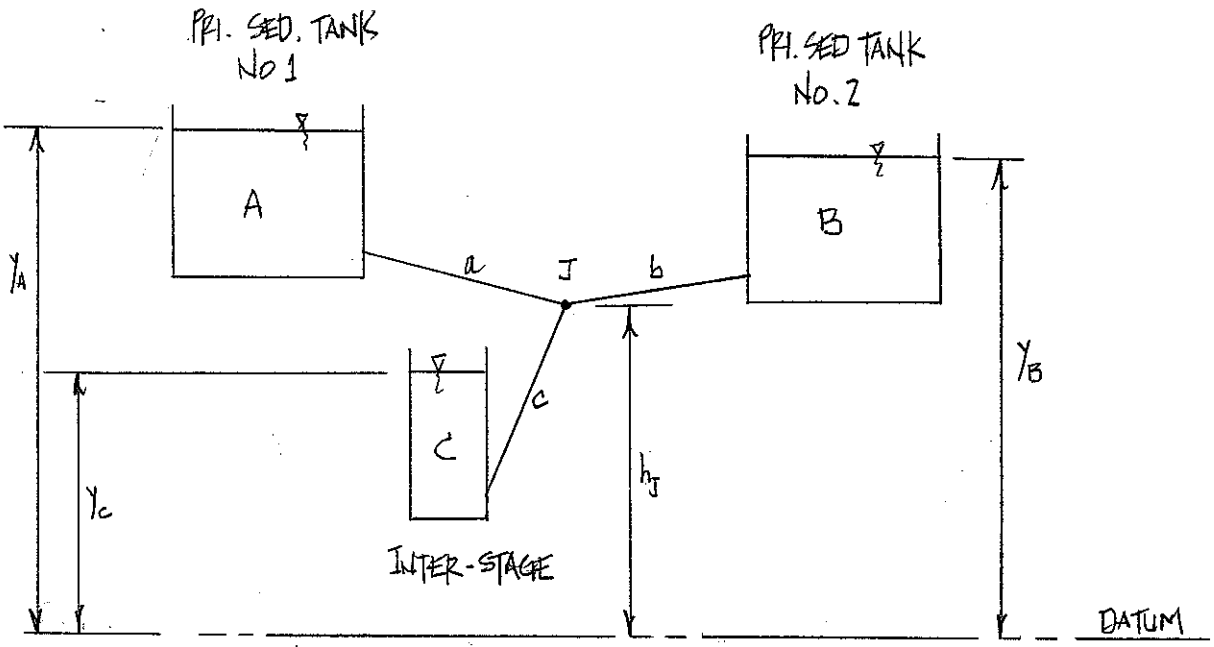
PROJECT: Morro Bay / Cayucos WWTP

PAGE: 1

BY: LPK

DATE: 9/29/03

Hydraulic Capacity Analysis



RESERVOIR	INITIAL WATER SURFACE ELEVATION	PIPE	DIAMETER	EQUIV. LENGTH	f	fL/D^5
A - 1° SED. TANK #1	122.37	a	18"	190 LF	0.010	0.250
B - 1° SED. TANK #2	122.27	b	15"	35'	0.010	0.115
C - INTERSTAGE PIPING	120.00	c	24", 12", 15"	30', 30', 131'	0.010	0.739

Pipe "a"

Equivalent length 18"

Pipe	=	100'
1 - 11° bend	=	10'
2 - 45° bend	=	2×15'
1 - 90° bend	=	22'
1 - BFV F.O.	=	25'
		≈190 LF

Pipe "b"

Equivalent length 15"

Pipe	=	10'
1 - BFV	=	25'
		≈ 35'

Pipe "c"

"12" pipe	=	20'
12"×15" red	=	10'
		30'
"15" pipe	=	88'
1 90°	=	18'
2 45°	=	25'
		131'
24" pipe	=	30'

Cannon ASSOCIATES
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HYDRAULIC CAPACITY ANALYSIS

USING BERNOULLI ENERGY EQUATION

$$Y_A - h_J = h_{L_a} = f_a \frac{L_a}{D_a^5}\left(\frac{4}{\pi}\right)^2 \frac{Q_a^2}{2g}$$

$$h_J - Y_c = h_{L_c} = f_c\left(\frac{L_{24}}{D_{24}^5} + \frac{L_{12}}{D_{12}^5} + \frac{L_{15}}{D_{15}^5}\right)\left(\frac{4}{\pi}\right)^2 \frac{Q_c^2}{2g}$$

FIRST TRIAL, ASSUME $h_J = Y_B$

SOLUTION: TRIAL & ERROR $h_J = 122.27$ (SEE HYDRAULIC PROFILE, APPENDIX B) FOR POWF ELEVATIONS

TRIAL #1

$$122.37 - 122.27 = 0.250\left(\frac{4}{\pi}\right)^2 \frac{Q_a^2}{2g} \Rightarrow Q_a = \left(0.10/0.0063\right)^{1/2} = 3.98 \text{ cfs}$$

$$122.27 - 120.00^{*} = 0.739\left(\frac{4}{\pi}\right)^2 \frac{Q_c^2}{2g} \Rightarrow Q_c = \left(2.27/0.0186\right)^{1/2} = 11.05 \text{ cfs}$$

* ASSUME 1' DEPTH OVER WEIR WHEN SLUICE GATE IS FULLY OPEN.

$Q_c > Q_a \therefore Q_b$ FLOWS TO Q_c

TRIAL #2 : $h_J = 122.15$

$$122.37 - 122.15 = 0.25\left(\frac{4}{\pi}\right)^2 \frac{Q_a^2}{2g} \Rightarrow Q_a = \left(0.22/0.0063\right)^{1/2} = 5.91 \text{ cfs}$$

$$122.27 - 122.15 = 0.115\left(\frac{4}{\pi}\right)^2 \frac{Q_b^2}{2g} \Rightarrow Q_b = \left(0.112/0.0029\right)^{1/2} = 6.44 \text{ cfs}$$

$$122.15 - 120.00 = 0.739\left(\frac{4}{\pi}\right)^2 \frac{Q_c^2}{2g} \Rightarrow Q_c = \left(2.15/0.0186\right)^{1/2} = 10.75 \text{ cfs}$$

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PROJECT: Morro Bay / Cayucos WWTP

PAGE: 3/

BY: LPK

DATE: 9/29/03

Hydraulic Capacity Analysis

Trial #3 $h_J = 122.20$; Water Surface Elev. @ Interstage = 120.20 ; $Q = 10.14$ cfs

$$122.37 - 122.20 = 0.250\left(\frac{4}{\pi}\right)^2 \frac{Q_a^2}{2g} \Rightarrow Q_a = \left(0.17/0.0063\right)^{1/2} = 5.2 \text{ cfs}$$

$$122.27 - 122.20 = 0.115\left(\frac{4}{\pi}\right)^2 \frac{Q_b^2}{2g} \Rightarrow Q_b = \left(0.07/0.0029\right)^{1/2} = 4.91 \text{ cfs}$$

$$122.20 - 120.20 = 0.739\left(\frac{4}{\pi}\right)^2 \frac{Q_c^2}{2g} \Rightarrow Q_c = \left(2.00/0.0186\right)^{1/2} = 10.37 \text{ cfs}$$

Approx. Balanced

$Q \approx 10.14$ cfs ≈ 6.6 MGD

Worksheet
Worksheet for Broad Crested Weir

Project Description	
Worksheet	Weir - 1
Type	Broad Crested \
Solve For	Discharge

Input Data		
Headwater Elev:	20.20	ft
Crest Elevation	19.00	ft
Tailwater Elevati	19.00	ft
Crest Surface T	Paved	
Crest Breadth	0.67	ft
Crest Length	2.50	ft

Results		
Discharge	6.56	mgd
Headwater Height Above	1.20	ft
Tailwater Height Above C	0.00	ft
Discharge Coefficient	3.09	US
Submergence Factor	1.00	
Adjusted Discharge Coef	3.09	US
Flow Area	3.0	ft²
Velocity	3.38	ft/s
Wetted Perimeter	4.90	ft
Top Width	2.50	ft

Appendix D

BOD_5 and TSS Data

Influent Sampling102803.xls

Sampling Location	Date	Time	Suspended Solids, mg/L	BOD, mg/L
Headworks-influent	10/27/2003	1300	236	264
Headworks-influent	10/27/2003	1500	236	252
Headworks-influent	10/27/2003	1700	332	228
Headworks-influent	10/27/2003	1900	316	258
Headworks-influent	10/27/2003	2100	572	264
Headworks-influent	10/27/2003	2300	300	306
Headworks-influent	10/27/2003	100	204	162
Headworks-influent	10/28/2003	300	88	84
Headworks-influent	10/28/2003	500	116	66
Headworks-influent	10/28/2003	700	228	192
Headworks-influent	10/28/2003	900	448	300
Headworks-influent	10/28/2003	1100	412	306
Note slight shift in sample time due to sample program				
Sample Volume collected: 500 ml's				
TSS calculated using 25ml samples				
BOD's calculated using 5ml dilution per bottle, two bottles per sample				

Influent Sampling103103.xls

Sampling Location	Date	Time	Suspended Solids, mg/L	BOD, mg/L
Headworks-influent	10/30/2003	1030	516	258
Headworks-influent	10/30/2003	1230	284	246
Headworks-influent	10/30/2003	1430	304	240
Headworks-influent	10/30/2003	1630	804	432
Headworks-influent	10/30/2003	1830	336	270
Headworks-influent	10/30/2003	2030	288	306
Headworks-influent	10/30/2003	2230	232	210
Headworks-influent	10/31/2003	30	204	216
Headworks-influent	10/31/2003	230	112	132
Headworks-influent	10/31/2003	430	72	90
Headworks-influent	10/31/2003	630	180	162
Headworks-influent	10/31/2003	830	320	312

Sampler malfunctioned on Wednesday, so sample pushed backed to sample from Thursday to Friday

Note slight shift in sample time due to sampler programming

Sample Volume collected: 500 ml's

TSS calculated using 25ml samples

BOD's calculated using 5ml dilution per bottle, two bottles per sample

Influent Sampling110103.xls

Sampling Location	Date	Time	Suspended Solids, mg/L	BOD, mg/L
Headworks-influent	10/31/2003	1200	328	324
Headworks-influent	10/31/2003	1400	252	300
Headworks-influent	10/31/2003	1600	236	270
Headworks-influent	10/31/2003	1800	440	306
Headworks-influent	10/31/2003	2000	364	264
Headworks-influent	10/31/2003	2200	360	252
Headworks-influent	10/31/2003	2400	248	234
Headworks-influent	11/1/2003	200	144	186
Headworks-influent	11/1/2003	400	124	162
Headworks-influent	11/1/2003	600	480	162
Headworks-influent	11/1/2003	800	240	252
Headworks-influent	11/1/2003	1000	352	342

Note slight shift in sample time due to sampler programming

Sample Volume collected: 500 ml's

TSS calculated using 25ml samples

BOD's calculated using 5ml dilution per bottle, two bottles per sample

PriClarifier1 Sampling100703.xls

Sampling Location	Date	Time	Suspended Solids, mg/L	BOD, mg/L
primary clarifier #1	10/6/2003	1100	132	180
primary clarifier #1	10/6/2003	1300	100	168
primary clarifier #1	10/6/2003	1500	92	168
primary clarifier #1	10/6/2003	1700	116	174
primary clarifier #1	10/6/2003	1900	80	168
primary clarifier #1	10/6/2003	2100	132	192
primary clarifier #1	10/6/2003	2300	148	204
primary clarifier #1	10/7/2003	100	160	210
primary clarifier #1	10/7/2003	300	164	198
primary clarifier #1	10/7/2003	500	140	204
primary clarifier #1	10/7/2003	700	128	180
primary clarifier #1	10/7/2003	900	136	198

Sample Volume collected: 500 ml's

TSS calculated using 25ml samples

BOD's calculated using 5ml dilution per bottle, two bottles per sample

PriClarifier1 Sampling100903.xls

Sampling Location	Date	Time	Suspended Solids, mg/L	BOD, mg/L
primary clarifier #1	10/8/2003	1100	748	168
primary clarifier #1	10/8/2003	1300	172	150
primary clarifier #1	10/8/2003	1500	136	150
primary clarifier #1	10/8/2003	1700	140	138
primary clarifier #1	10/8/2003	1900	388	138
primary clarifier #1	10/8/2003	2100	140	150
primary clarifier #1	10/8/2003	2300	112	174
primary clarifier #1	10/9/2003	100	108	168
primary clarifier #1	10/9/2003	300	96	162
primary clarifier #1	10/9/2003	500	100	138
primary clarifier #1	10/9/2003	700	104	144
primary clarifier #1	10/9/2003	900	228	144

Lab staff reported large chunks on filter paper for the 1100 sample; probably not representative sample

Sample Volume collected: 500 ml's

TSS calculated using 25ml samples

BOD's calculated using 5ml dilution per bottle, two bottles per sample

PriClarifier2 Sampling 093003.xls

Sampling Location	Date	Time	Suspended Solids, mg/L	BOD, mg/L
primary clarifier #2	9/29/2003	1100	188	162
primary clarifier #2	9/29/2003	1300	140	138
primary clarifier #2	9/29/2003	1500	176	138
primary clarifier #2	9/29/2003	1700	124	144
primary clarifier #2	9/29/2003	1900	164	144
primary clarifier #2	9/29/2003	2100	136	156
primary clarifier #2	9/29/2003	2300	96	186
primary clarifier #2	9/30/2003	100	104	180
primary clarifier #2	9/30/2003	300	84	162
primary clarifier #2	9/30/2003	500	84	132
primary clarifier #2	9/30/2003	700	128	126
primary clarifier #2	9/30/2003	900	112	186
Sample Volume collected: 500 ml's				
TSS calculated using 25ml samples				
BOD's calculated using 5ml dilution per bottle, two bottles per sample				

Sampling Location	Date	Time	Suspended Solids, mg/L	BOD, mg/L
primary clarifier #2	10/1/2003	1100	140	162
primary clarifier #2	10/1/2003	1300	116	180
primary clarifier #2	10/1/2003	1500	108	144
primary clarifier #2	10/1/2003	1700	100	144
primary clarifier #2	10/1/2003	1900	136	138
primary clarifier #2	10/1/2003	2100	272	138
primary clarifier #2	10/1/2003	2300	140	174
primary clarifier #2	10/2/2003	100	120	162
primary clarifier #2	10/2/2003	300	96	156
primary clarifier #2	10/2/2003	500	104	150
primary clarifier #2	10/2/2003	700	96	132
primary clarifier #2	10/2/2003	900	140	156
Sample Volume collected: 500 ml's				
TSS calculated using 25ml samples				
BOD's calculated using 5ml dilution per bottle, two bottles per sample				

PriClarifier2 Sampling 100403.xls

Sampling Location	Date	Time	Suspended Solids, mg/L	BOD, mg/L
primary clarifier #2	10/3/2004	1100	156	192
primary clarifier #2	10/3/2004	1300	120	168
primary clarifier #2	10/3/2004	1500	116	162
primary clarifier #2	10/3/2004	1700	104	168
primary clarifier #2	10/3/2004	1900	112	168
primary clarifier #2	10/3/2004	2100	124	186
primary clarifier #2	10/3/2004	2300	152	204
primary clarifier #2	10/4/2003	100	104	192
primary clarifier #2	10/4/2003	300	96	180
primary clarifier #2	10/4/2003	500	88	162
primary clarifier #2	10/4/2003	700	96	156
primary clarifier #2	10/4/2003	900	84	156
Sample Volume collected: 500 ml's				
TSS calculated using 25ml samples				
BOD's calculated using 5ml dilution per bottle, two bottles per sample				

PriClarifier2 Sampling100603.xls

Sampling Location	Date	Time	Suspended Solids, mg/L	BOD, mg/L
primary clarifier #2	10/5/2006	1100	92	
primary clarifier #2	10/5/2006	1300	80	
primary clarifier #2	10/5/2006	1500	88	
primary clarifier #2	10/5/2006	1700	92	
primary clarifier #2	10/5/2006	1900	116	
primary clarifier #2	10/5/2006	2100	100	
primary clarifier #2	10/5/2006	2300	116	
primary clarifier #2	10/6/2003	100	140	
primary clarifier #2	10/6/2003	300	128	
primary clarifier #2	10/6/2003	500	124	
primary clarifier #2	10/6/2003	700	116	
primary clarifier #2	10/6/2003	900	104	
Sample Volume collected: 500 ml's				
TSS calculated using 25ml samples				
BOD's calculated using 5ml dilution per bottle, two bottles per sample				

TF#1 Sampling102103.xls

Sampling Location	Date	Time	Suspended Solids, mg/L	BOD, mg/L
Trickling Filter #1	10/20/2003	1100	132	114
Trickling Filter #1	10/20/2003	1300	120	96
Trickling Filter #1	10/20/2003	1500	132	84
Trickling Filter #1	10/20/2003	1700	152	90
Trickling Filter #1	10/20/2003	1900	100	78
Trickling Filter #1	10/20/2003	2100	168	84
Trickling Filter #1	10/20/2003	2300	92	96
Trickling Filter #1	10/21/2003	100	68	78
Trickling Filter #1	10/21/2003	300	120	66
Trickling Filter #1	10/21/2003	500	96	54
Trickling Filter #1	10/21/2003	700	60	66
Trickling Filter #1	10/21/2003	900	92	114

Sample Volume collected: 500 ml's

TSS calculated using 25ml samples

BOD's calculated using 5ml dilution per bottle, two bottles per sample

TF#1 Sampling102303.xls

Sampling Location	Date	Time	Suspended Solids, mg/L	BOD, mg/L
Trickling Filter #1	10/22/2003	1100	232	102
Trickling Filter #1	10/22/2003	1300	300	246
Trickling Filter #1	10/22/2003	1500	180	162
Trickling Filter #1	10/22/2003	1700	188	126
Trickling Filter #1	10/22/2003	1900	232	96
Trickling Filter #1	10/22/2003	2100	140	96
Trickling Filter #1	10/22/2003	2300	172	84
Trickling Filter #1	10/23/2003	100	204	66
Trickling Filter #1	10/23/2003	300	188	54
Trickling Filter #1	10/23/2003	500	116	42
Trickling Filter #1	10/23/2003	700	76	54
Trickling Filter #1	10/23/2003	900	148	84

Sample Volume collected: 500 ml's

TSS calculated using 25ml samples

BOD's calculated using 5ml dilution per bottle, two bottles per sample

TF#1 Sampling102503.xls

Sampling Location	Date	Time	Suspended Solids, mg/L	BOD, mg/L
Trickling Filter #1	10/24/2003	1100	168	108
Trickling Filter #1	10/24/2003	1300	136	84
Trickling Filter #1	10/24/2003	1500	148	90
Trickling Filter #1	10/24/2003	1700	144	84
Trickling Filter #1	10/24/2003	1900	212	96
Trickling Filter #1	10/24/2003	2100	176	96
Trickling Filter #1	10/24/2003	2300	156	96
Trickling Filter #1	10/25/2003	100	144	90
Trickling Filter #1	10/25/2003	300	116	66
Trickling Filter #1	10/25/2003	500	124	60
Trickling Filter #1	10/25/2003	700	96	66
Trickling Filter #1	10/25/2003	900	132	96

Sample Volume collected: 500 ml's

TSS calculated using 25ml samples

BOD's calculated using 5ml dilution per bottle, two bottles per sample

TF#1 Sampling102703.xls

Sampling Location	Date	Time	Suspended Solids, mg/L	BOD, mg/L
Trickling Filter #1	10/26/2003	1100	144	66
Trickling Filter #1	10/26/2003	1300	144	114
Trickling Filter #1	10/26/2003	1500	172	66
Trickling Filter #1	10/26/2003	1700	172	66
Trickling Filter #1	10/26/2003	1900	192	72
Trickling Filter #1	10/26/2003	2100	140	84
Trickling Filter #1	10/26/2003	2300	148	66
Trickling Filter #1	10/27/2003	100	92	60
Trickling Filter #1	10/27/2003	300	84	30
Trickling Filter #1	10/27/2003	500	88	24
Trickling Filter #1	10/27/2003	700	88	24
Trickling Filter #1	10/27/2003	900	236	54

Sample Volume collected: 500 ml's

TSS calculated using 25ml samples

BOD's calculated using 5ml dilution per bottle, two bottles per sample

TF#2 Sampling101603.xls

Sampling Location	Date	Time	Suspended Solids, mg/L	BOD, mg/L
Trickling Filter #2	10/15/2003	1100	196	162
Trickling Filter #2	10/15/2003	1300	132	126
Trickling Filter #2	10/15/2003	1500	120	108
Trickling Filter #2	10/15/2003	1700	116	102
Trickling Filter #2	10/15/2003	1900	108	108
Trickling Filter #2	10/15/2003	2100	128	114
Trickling Filter #2	10/15/2003	2300	128	120
Trickling Filter #2	10/16/2003	100	100	96
Trickling Filter #2	10/16/2003	300	84	72
Trickling Filter #2	10/16/2003	500	56	60
Trickling Filter #2	10/16/2003	700	68	66
Trickling Filter #2	10/16/2003	900	108	108

Sample Volume collected: 500 ml's

TSS calculated using 25ml samples

BOD's calculated using 5ml dilution per bottle, two bottles per sample

TF#2 Sampling101803.xls

Sampling Location	Date	Time	Suspended Solids, mg/L	BOD, mg/L
Trickling Filter #2	10/15/2003	1100	200	96
Trickling Filter #2	10/15/2003	1300	164	84
Trickling Filter #2	10/15/2003	1500	124	60
Trickling Filter #2	10/15/2003	1700	128	66
Trickling Filter #2	10/15/2003	1900	140	78
Trickling Filter #2	10/15/2003	2100	128	72
Trickling Filter #2	10/15/2003	2300	136	78
Trickling Filter #2	10/16/2003	100	116	54
Trickling Filter #2	10/16/2003	300	80	42
Trickling Filter #2	10/16/2003	500	64	30
Trickling Filter #2	10/16/2003	700	80	24
Trickling Filter #2	10/16/2003	900	116	72

Sample Volume collected: 500 ml's

TSS calculated using 25ml samples

BOD's calculated using 5ml dilution per bottle, two bottles per sample

TF#2 Sampling102003.xls

Sampling Location	Date	Time	Suspended Solids, mg/L	BOD, mg/L
Trickling Filter #2	10/19/2003	1100	200	180
Trickling Filter #2	10/19/2003	1300	156	150
Trickling Filter #2	10/19/2003	1500	160	156
Trickling Filter #2	10/19/2003	1700	172	156
Trickling Filter #2	10/19/2003	1900	160	150
Trickling Filter #2	10/19/2003	2100	152	162
Trickling Filter #2	10/19/2003	2300	224	138
Trickling Filter #2	10/20/2003	100	96	120
Trickling Filter #2	10/20/2003	300	112	90
Trickling Filter #2	10/20/2003	500	76	90
Trickling Filter #2	10/20/2003	700	84	96
Trickling Filter #2	10/20/2003	900	112	126

Sample Volume collected: 500 ml's

TSS calculated using 25ml samples

BOD's calculated using 5ml dilution per bottle, two bottles per sample

SecClarifier112803Sampling.xls

Sampling Location	Date	Time	Suspended Solids, mg/L	BOD, mg/L
Secondary Clarifier	11/27/2003	1100	48	56
Secondary Clarifier	11/27/2003	1300	48	56
Secondary Clarifier	11/27/2003	1500	40	60
Secondary Clarifier	11/27/2003	1700	24	58
Secondary Clarifier	11/27/2003	1900	36	58
Secondary Clarifier	11/27/2003	2100	28	58
Secondary Clarifier	11/27/2003	2300	36	60
Secondary Clarifier	11/28/2003	100	40	58
Secondary Clarifier	11/28/2003	300	28	52
Secondary Clarifier	11/28/2003	500	16	42
Secondary Clarifier	11/28/2003	700	16	46
Secondary Clarifier	11/28/2003	900	12	42

Sample Volume collected: 500 ml's

TSS calculated using 25ml samples

BOD's calculated using 15ml dilution per bottle, two bottles per sample

SecClarifier112903Sampling.xls

Sampling Location	Date	Time	Suspended Solids, mg/L	BOD, mg/L
Secondary Clarifier	11/28/2003	1100	44	52
Secondary Clarifier	11/28/2003	1300	44	56
Secondary Clarifier	11/28/2003	1500	44	62
Secondary Clarifier	11/28/2003	1700	24	62
Secondary Clarifier	11/28/2003	1900	28	62
Secondary Clarifier	11/28/2003	2100	44	60
Secondary Clarifier	11/28/2003	2300	32	58
Secondary Clarifier	11/29/2003	100	44	54
Secondary Clarifier	11/29/2003	300	32	52
Secondary Clarifier	11/29/2003	500	40	46
Secondary Clarifier	11/29/2003	700	28	42
Secondary Clarifier	11/29/2003	900	24	38

Sample Volume collected: 500 ml's

TSS calculated using 25ml samples

BOD's calculated using 15ml dilution per bottle, two bottles per sample

SecClarifier112903Sampling.xls

Sampling Location	Date	Time	Suspended Solids, mg/L	BOD, mg/L
Secondary Clarifier	11/28/2003	1100	44	52
Secondary Clarifier	11/28/2003	1300	44	56
Secondary Clarifier	11/28/2003	1500	44	62
Secondary Clarifier	11/28/2003	1700	24	62
Secondary Clarifier	11/28/2003	1900	28	62
Secondary Clarifier	11/28/2003	2100	44	60
Secondary Clarifier	11/28/2003	2300	32	58
Secondary Clarifier	11/29/2003	100	44	54
Secondary Clarifier	11/29/2003	300	32	52
Secondary Clarifier	11/29/2003	500	40	46
Secondary Clarifier	11/29/2003	700	28	42
Secondary Clarifier	11/29/2003	900	24	38

Sample Volume collected: 500 ml's

TSS calculated using 25ml samples

BOD's calculated using 15ml dilution per bottle, two bottles per sample

SecClarifier113003Sampling.xls

Sampling Location	Date	Time	Suspended Solids, mg/L	BOD, mg/L
Secondary Clarifier	11/29/2003	1100	36	56
Secondary Clarifier	11/29/2003	1300	60	66
Secondary Clarifier	11/29/2003	1500	40	68
Secondary Clarifier	11/29/2003	1700	40	66
Secondary Clarifier	11/29/2003	1900	40	68
Secondary Clarifier	11/29/2003	2100	56	70
Secondary Clarifier	11/29/2003	2300	32	66
Secondary Clarifier	11/30/2003	100	32	68
Secondary Clarifier	11/30/2003	300	20	58
Secondary Clarifier	11/30/2003	500	20	56
Secondary Clarifier	11/30/2003	700	20	52
Secondary Clarifier	11/30/2003	900	16	54

Sample Volume collected: 500 ml's

TSS calculated using 25ml samples

BOD's calculated using 15ml dilution per bottle, two bottles per sample

EffWaterfall111103Sampling.xls

Sampling Location	Date	Time	Suspended Solids, mg/L	BOD, mg/L	
Effluent-Waterfall	11/10/2003	1130	44	39	
Effluent-Waterfall	11/10/2003	1330	164	27	chunk noted in sample
Effluent-Waterfall	11/10/2003	1530	32	24	
Effluent-Waterfall	11/10/2003	1730	28	24	
Effluent-Waterfall	11/10/2003	1930	20	27	
Effluent-Waterfall	11/10/2003	2130	36	27	
Effluent-Waterfall	11/10/2003	2330	24	27	
Effluent-Waterfall	11/11/2003	130	16	21	
Effluent-Waterfall	11/11/2003	330	20	15	
Effluent-Waterfall	11/11/2003	530	16	15	
Effluent-Waterfall	11/11/2003	730	20	12	
Effluent-Waterfall	11/11/2003	930	24	18	

Sample Volume collected: 500 ml's

TSS calculated using 25ml samples

BOD's calculated using 10ml dilution per bottle, two bottles per sample

EffWaterfall111303Sampling.xls

Sampling Location	Date	Time	Suspended Solids, mg/L	BOD, mg/L
Effluent-Waterfall	11/12/2003	1100	28	21
Effluent-Waterfall	11/12/2003	1300	28	15
Effluent-Waterfall	11/12/2003	1500	32	21
Effluent-Waterfall	11/12/2003	1700	28	18
Effluent-Waterfall	11/12/2003	1900	20	9
Effluent-Waterfall	11/12/2003	2100	12	6
Effluent-Waterfall	11/12/2003	2300	28	12
Effluent-Waterfall	11/13/2003	100	24	9
Effluent-Waterfall	11/13/2003	300	12	9
Effluent-Waterfall	11/13/2003	500	20	6
Effluent-Waterfall	11/13/2003	700	20	6
Effluent-Waterfall	11/13/2003	900	16	9

Sample Volume collected: 500 ml's

TSS calculated using 25ml samples

BOD's calculated using 10ml dilution per bottle, two bottles per sample

EffWaterfall111503Sampling.xls

Sampling Location	Date	Time	Suspended Solids, mg/L	BOD, mg/L
Effluent-Waterfall	11/14/2003	1130	56	
Effluent-Waterfall	11/14/2003	1330	60	
Effluent-Waterfall	11/14/2003	1530	40	
Effluent-Waterfall	11/14/2003	1730	48	
Effluent-Waterfall	11/14/2003	1930	20	
Effluent-Waterfall	11/14/2003	2130	40	
Effluent-Waterfall	11/14/2003	2330	36	
Effluent-Waterfall	11/15/2003	130	36	
Effluent-Waterfall	11/15/2003	330	24	
Effluent-Waterfall	11/15/2003	530	32	
Effluent-Waterfall	11/15/2003	730	28	
Effluent-Waterfall	11/15/2003	930	28	

BOD Meter malfuctioned on the read portion of the test so that no BOD's are available for this twenty four hour period

Sample Volume collected: 500 ml's

TSS calculated using 25ml samples

BOD's calculated using 10ml dilution per bottle, two bottles per sample

EffWaterfall111703Sampling.xls

Sampling Location	Date	Time	Suspended Solids, mg/L	BOD, mg/L
Effluent-Waterfall	11/16/2003	1100	24	15
Effluent-Waterfall	11/16/2003	1300	40	27
Effluent-Waterfall	11/16/2003	1500	44	30
Effluent-Waterfall	11/16/2003	1700	16	30
Effluent-Waterfall	11/16/2003	1900	32	21
Effluent-Waterfall	11/16/2003	2100	32	27
Effluent-Waterfall	11/16/2003	2300	24	21
Effluent-Waterfall	11/17/2003	100	20	24
Effluent-Waterfall	11/17/2003	300	8	18
Effluent-Waterfall	11/17/2003	500	8	15
Effluent-Waterfall	11/17/2003	700	16	15
Effluent-Waterfall	11/17/2003	900		12

No TSS was reported for 11/17/03 at 0900 due to lab error in handling the sample

Sample Volume collected: 500 ml's

TSS calculated using 25ml samples

BOD's calculated using 10ml dilution per bottle, two bottles per sample

Appendix E

Flow Equalization Basin Calculations

Flow Equalization Calcs_Current_Average Condition.xls

Time Period	Average Flow Rate During Time Period, ft^3/s	*Average BOD concentration during time period, mg/L	*Average TSS concentration during time period, mg/L	Cumulative volume flow at end of time period, ft^3	BOD Mass Loading during time period, lb/hr	TSS Mass Loading during time period, lb/hr
M				0		
M-1	1.37	260	174	4,947	80	54
1-2	1.06	252	166	8,758	60	39
2-3	0.78	245	157	11,576	43	28
3-4	0.75	237	154	14,261	40	26
4-5	0.64	228	151	16,553	33	22
5-6	0.65	221	155	18,886	32	23
6-7	0.86	215	158	21,998	42	31
7-8	1.35	225	174	26,850	68	53
8-9	2.11	235	190	34,455	112	90
9-10	2.43	237	196	43,192	129	107
10-11	2.58	239	201	52,473	139	116
11-N	2.83	234	187	62,656	149	119
N-1	2.44	230	172	71,440	126	94
1-2	2.24	226	171	79,517	114	86
2-3	2.12	223	169	87,135	106	81
3-4	1.99	224	165	94,290	100	74
4-5	2.00	226	160	101,477	101	72
5-6	1.92	221	198	108,374	95	85
6-7	2.04	216	236	115,724	99	108
7-8	2.17	226	225	123,526	110	109
8-9	2.39	237	214	132,136	127	115
9-10	2.17	252	197	139,951	123	96
10-11	2.06	268	181	147,351	124	83
11-M	1.67	264	178	153,379	99	67
Average	1.78	235	180		94	74
Ave MGD	1.15					
Minimum	0.64	215	151		32	22
Maximum	2.83	268	236		149	119

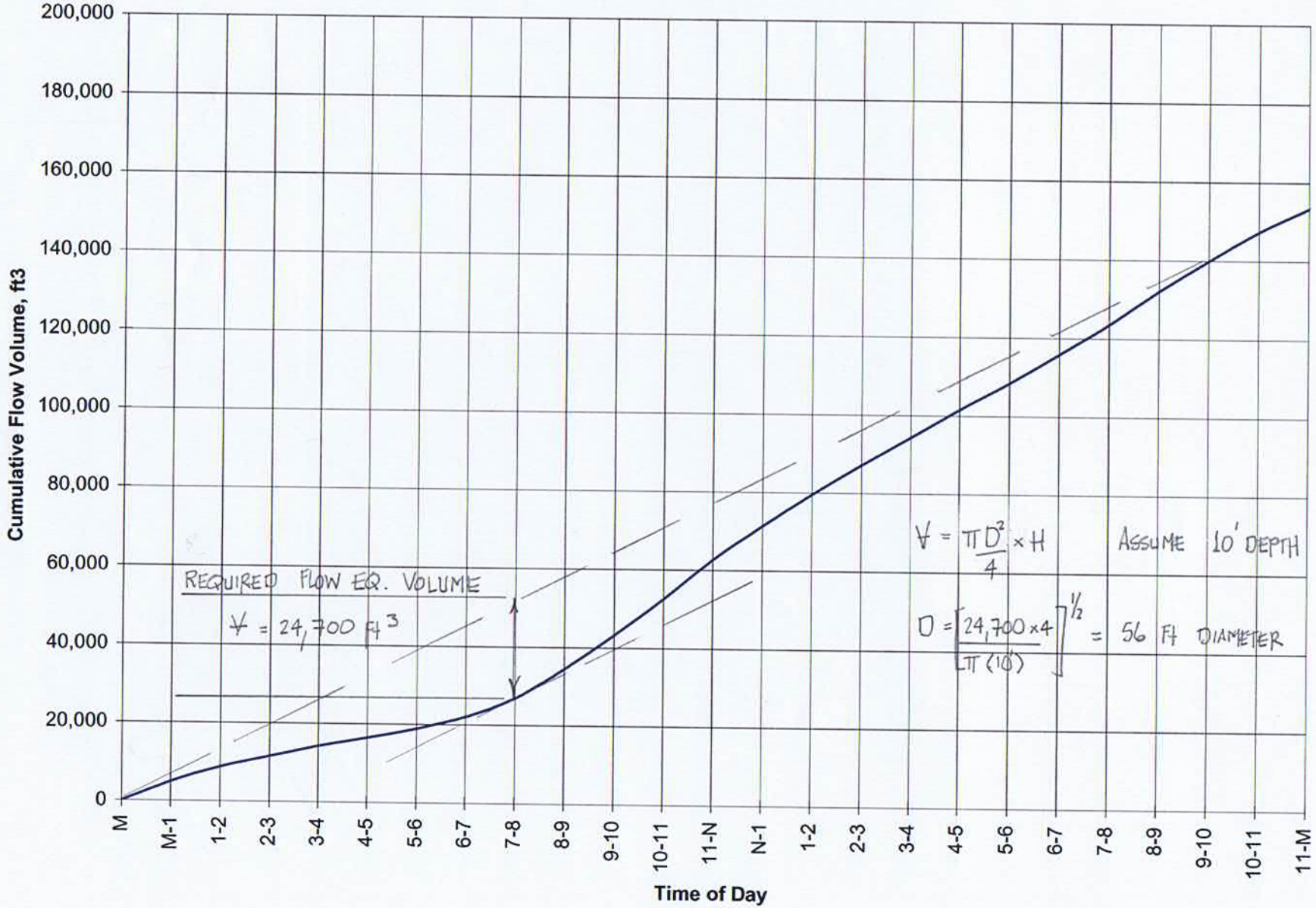
* Data was compiled for every other hour; intermediate hours are an average of the hours before and after; the BOD and TSS values are increased 37.5% and 42%, respectively, to account for increased wastewater strength during the peak seasonal periods and to normalize the data with past performance data.

Flow Equalization Calcs_Current_Average Condition.xls

Time Period	Volume of Flow during time period, ft3	Volume in Storage at end of time period, ft3	Average BOD concentration during time period, mg/l	Equalized BOD concentration during time period, mg/l	Equalized BOD mass loading during time period, lb/hr	Average TSS concentration during time period, mg/l	Equalized TSS concentration during time period, mg/l	Equalized TSS mass loading during time period, lb/hr	Organic Loadings to Existing Trickling Filter, lbs/1000cf/d
8-9	7,605	1,214	235.125	235	94	190	190	76	58
9-10	8,737	3,560	237.1875	237	95	196	195	78	43
10-11	9,281	6,451	239.25	239	95	201	199	80	31
11-N	10,183	10,243	234.4375	236	94	187	192	77	29
N-1	8,784	12,636	229.625	233	93	172	183	73	24
1-2	8,078	14,323	226.1875	230	92	171	178	71	23
2-3	7,617	15,549	222.75	228	91	169	175	70	30
3-4	7,156	16,314	224.125	227	91	165	172	69	49
4-5	7,186	17,110	225.5	226	90	160	168	67	80
5-6	6,898	17,616	220.6875	225	90	198	177	71	93
6-7	7,350	18,576	215.875	222	89	236	194	78	100
7-8	7,801	19,986	226.1875	223	89	225	203	81	107
8-9	8,611	22,206	236.5	227	91	214	206	83	91
9-10	7,814	23,630	252.3125	234	93	197	204	82	82
10-11	7,400	24,639	268.125	242	97	181	199	79	76
11-M	6,028	24,276	264	246	98	178	194	78	72
M-1	4,947	22,833	259.875	249	99	174	191	76	73
1-2	3,811	20,253	252.3125	249	100	166	187	75	68
2-3	2,818	16,680	244.75	249	99	157	184	73	71
3-4	2,684	12,974	236.5	247	99	154	180	72	79
4-5	2,293	8,876	228.25	244	98	151	175	70	92
5-6	2,332	4,818	221.375	239	96	155	171	68	89
6-7	3,112	1,539	214.5	230	92	158	166	66	89
7-8	4,852	0	224.8125	226	90	174	172	69	72
Average	6,391	= ft^3			94		Average=	74	23
	1,147,271	= gal/day							107

Theoretical Flow Equalization Basin Results
Volume Determination

Current - Average Year



Flow Equalization Calcs_Current_Peak Season.xls

Time Period	Average Flow Rate During Time Period, ft^3/s	*Average BOD concentration during time period, mg/L	*Average TSS concentration during time period, mg/L	Cumulative volume flow at end of time period, ft^3	BOD Mass Loading during time period, lb/hr	TSS Mass Loading during time period, lb/hr
M				0		
M-1	1.50	304	212	5,405	103	72
1-2	1.16	295	202	9,568	77	52
2-3	0.86	287	191	12,647	55	37
3-4	0.81	277	188	15,579	51	34
4-5	0.70	267	185	18,084	42	29
5-6	0.71	259	189	20,632	41	30
6-7	0.94	251	193	24,032	53	41
7-8	1.47	263	212	29,332	87	70
8-9	2.31	275	232	37,640	143	120
9-10	2.65	278	238	47,184	165	142
10-11	2.82	280	245	57,323	177	155
11-N	3.09	275	227	68,448	191	158
N-1	2.67	269	210	78,043	161	126
1-2	2.45	265	208	86,868	146	115
2-3	2.31	261	206	95,189	135	107
3-4	2.17	262	201	103,006	128	98
4-5	2.18	264	195	110,857	129	95
5-6	2.09	258	241	118,392	121	113
6-7	2.23	253	287	126,421	127	144
7-8	2.37	265	274	134,944	141	146
8-9	2.61	277	261	144,351	163	153
9-10	2.37	295	240	152,887	157	128
10-11	2.25	314	220	160,972	158	111
11-M	1.83	309	216	167,556	127	89
Average	1.94	275	220		120	99
Ave MGD	1.25					
Minimum	0.70	251	185		41	29
Maximum	3.09	314	287		191	158

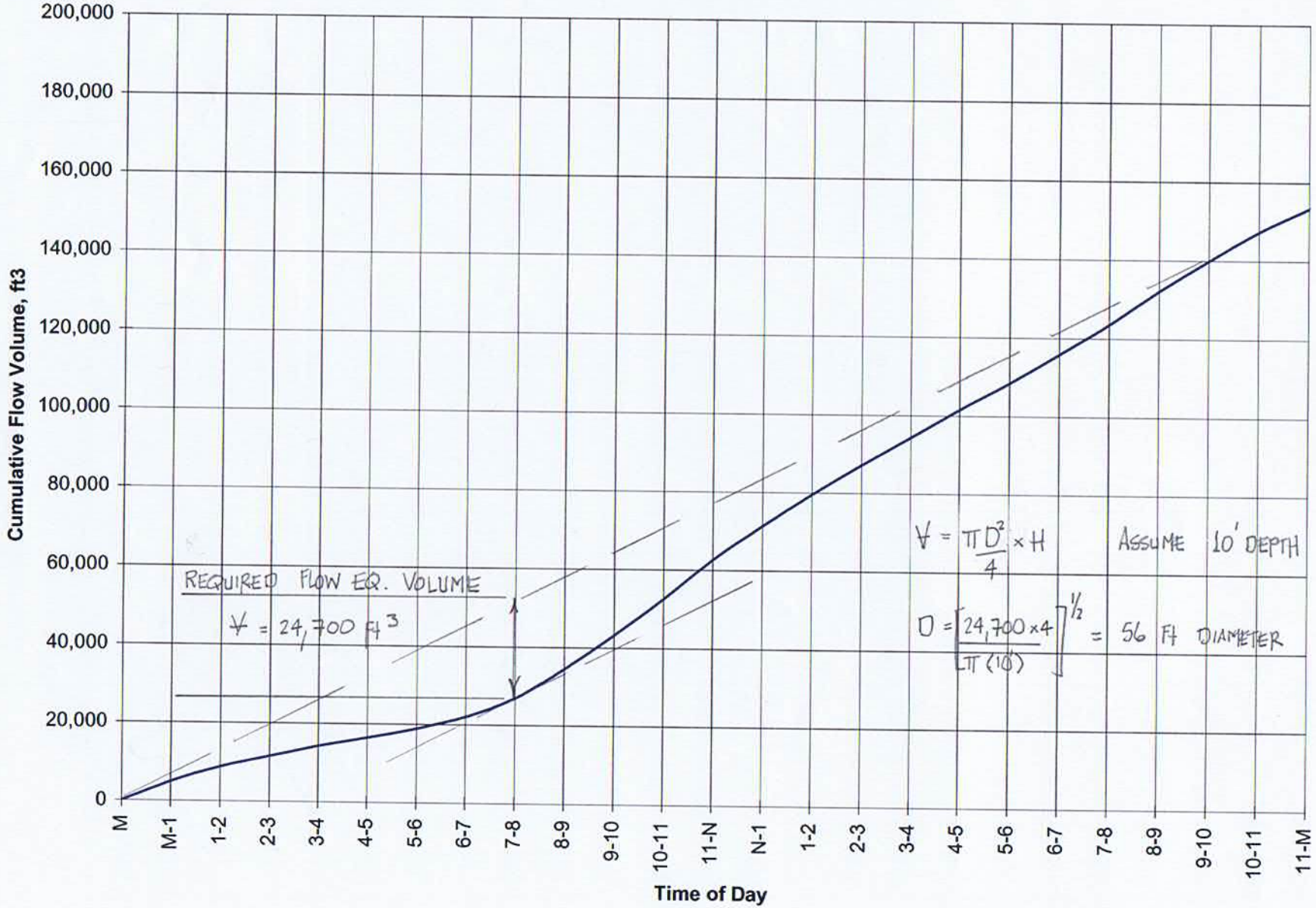
* Data was compiled for every other hour; intermediate hours are an average of the hours before and after; the BOD and TSS values are increased 61% and 73%, respectively, to account for increased wastewater strength during the peak seasonal periods and to normalize the data with past performance data.

Flow Equalization Calcs_Current_Peak Season.xls

Time Period	Volume of Flow during time period, ft3	Volume in Storage at end of time period, ft3	Average BOD concentration during time period, mg/l	Equalized BOD concentration during time period, mg/l	Equalized BOD mass loading during time period, lb/hr	Average TSS concentration during time period, mg/l	Equalized TSS concentration during time period, mg/l	Equalized TSS mass loading during time period, lb/hr
8-9	8,308	1,326	275.31	275	120	232	232	101
9-10	9,545	3,889	277.725	277	121	238	238	104
10-11	10,139	7,047	280.14	279	122	245	243	106
11-N	11,124	11,190	274.505	276	121	227	233	102
N-1	9,596	13,804	268.87	273	119	210	223	97
1-2	8,824	15,647	264.845	270	118	208	217	95
2-3	8,321	16,987	260.82	267	116	206	213	93
3-4	7,817	17,822	262.43	265	116	201	209	91
4-5	7,850	18,691	264.04	265	116	195	205	89
5-6	7,535	19,245	258.405	263	115	241	215	94
6-7	8,030	20,293	252.77	260	114	287	236	103
7-8	8,522	21,834	264.845	261	114	274	248	108
8-9	9,407	24,259	276.92	266	116	261	251	110
9-10	8,537	25,814	295.435	274	120	240	249	109
10-11	8,084	26,917	313.95	283	124	220	242	106
11-M	6,585	26,520	309.12	288	126	216	237	103
M-1	5,405	24,944	304.29	291	127	212	233	102
1-2	4,163	22,125	295.435	292	127	202	228	100
2-3	3,079	18,222	286.58	291	127	191	224	98
3-4	2,933	14,173	276.92	289	126	188	219	96
4-5	2,504	9,696	267.26	286	125	185	214	93
5-6	2,548	5,263	259.21	280	122	189	208	91
6-7	3,400	1,681	251.16	269	117	193	202	88
7-8	5,300	0	263.235	265	116	212	210	92
Average	6,982	= ft^3			120			99
	1,253,322	= gal/day						

Theoretical Flow Equalization Basin Results
Volume Determination

Current - Average Year



Flow Equalization Calcs_Future_Average Condition.xls

Time Period	Average Flow Rate During Time Period, ft^3/s	*Average BOD concentration during time period, mg/L	*Average TSS concentration during time period, mg/L	Cumulative volume flow at end of time period, ft^3	BOD Mass Loading during time period, lb/hr	TSS Mass Loading during time period, lb/hr
M				0		
M-1	1.65	260	174	5,945	96	65
1-2	1.27	252	166	10,525	72	47
2-3	0.94	245	157	13,911	52	33
3-4	0.90	237	154	17,137	48	31
4-5	0.77	228	151	19,892	39	26
5-6	0.78	221	155	22,695	39	27
6-7	1.04	215	158	26,435	50	37
7-8	1.62	225	174	32,265	82	63
8-9	2.54	235	190	41,403	134	108
9-10	2.92	237	196	51,903	155	128
10-11	3.10	239	201	63,056	166	140
11-N	3.40	234	187	75,293	179	143
N-1	2.93	230	172	85,848	151	113
1-2	2.70	226	171	95,554	137	103
2-3	2.54	223	169	104,708	127	97
3-4	2.39	224	165	113,307	120	88
4-5	2.40	226	160	121,942	121	86
5-6	2.30	221	198	130,231	114	102
6-7	2.45	216	236	139,064	119	130
7-8	2.60	226	225	148,438	132	131
8-9	2.87	237	214	158,786	153	138
9-10	2.61	252	197	168,176	148	116
10-11	2.47	268	181	177,069	149	100
11-M	2.01	264	178	184,312	119	80
Average	2.13	235	180		113	89
Ave MGD	1.38					
Minimum	0.77	215	151		39	26
Maximum	3.40	268	236		179	143

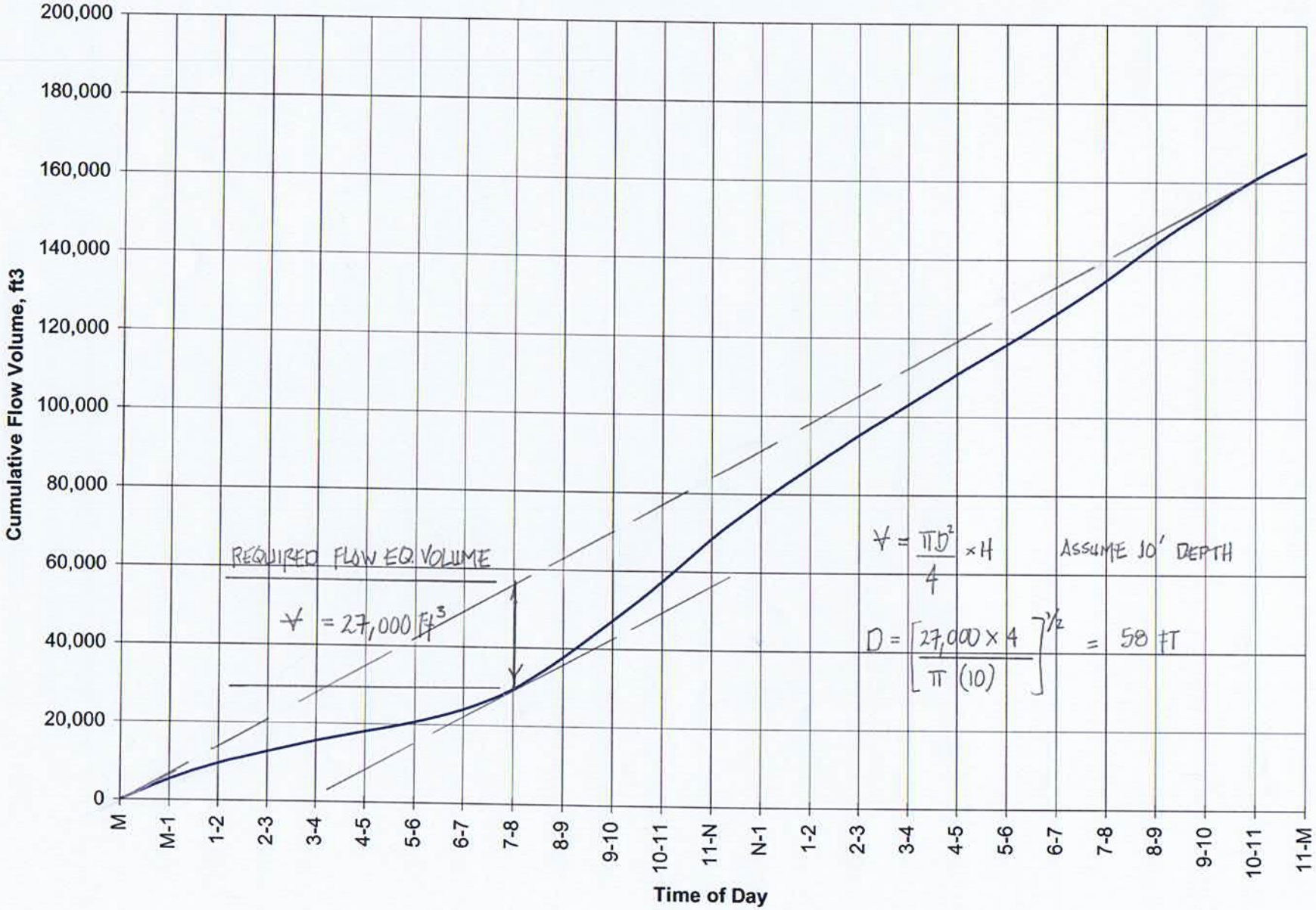
* Data was compiled for every other hour; intermediate hours are an average of the hours before and after; the BOD and TSS values are increased 37.5% and 42%, respectively, to account for increased wastewater strength during the peak seasonal periods and to normalize the data with past performance data.

Flow Equalization Calcs_Future_Average Condition.xls

Time Period	Volume of Flow during time period, ft3	Volume in Storage at end of time period, ft3	Average BOD concentration during time period, mg/l	Equalized BOD concentration during time period, mg/l	Equalized BOD mass loading during time period, lb/hr	Average TSS concentration during time period, mg/l	Equalized TSS concentration during time period, mg/l	Equalized TSS mass loading during time period, lb/hr
8-9	9,139	1,459	235.125	235	113	190	190	91
9-10	10,499	4,278	237.1875	237	114	196	195	94
10-11	11,153	7,752	239.25	239	115	201	199	96
11-N	12,237	12,309	234.4375	236	113	187	192	92
N-1	10,555	15,184	229.625	233	112	172	183	88
1-2	9,707	17,211	226.1875	230	111	171	178	86
2-3	9,154	18,685	222.75	228	109	169	175	84
3-4	8,599	19,605	224.125	227	109	165	172	83
4-5	8,635	20,560	225.5	226	109	160	168	81
5-6	8,289	21,169	220.6875	225	108	198	177	85
6-7	8,833	22,322	215.875	222	107	236	194	93
7-8	9,375	24,017	226.1875	223	107	225	203	98
8-9	10,347	26,685	236.5	227	109	214	206	99
9-10	9,390	28,396	252.3125	234	112	197	204	98
10-11	8,893	29,609	268.125	242	116	181	199	95
11-M	7,243	29,172	264	246	118	178	194	93
M-1	5,945	27,438	259.875	249	119	174	191	92
1-2	4,579	24,338	252.3125	249	120	166	187	90
2-3	3,387	20,045	244.75	249	119	157	184	88
3-4	3,226	15,591	236.5	247	119	154	180	86
4-5	2,755	10,666	228.25	244	117	151	175	84
5-6	2,803	5,789	221.375	239	115	155	171	82
6-7	3,740	1,849	214.5	230	110	158	166	80
7-8	5,830	0	224.8125	226	109	174	172	83
Average	7,680	= ft^3			113			89
	1,378,654	= gal/day						

Theoretical Flow Equalization Basin Results
Volume Determination

Current - Peak Seasonal



Flow Equalization Calcs_Future_Peak Season.xls

Time Period	Average Flow Rate During Time Period, ft^3/s	*Average BOD concentration during time period, mg/L	*Average TSS concentration during time period, mg/L	Cumulative volume flow at end of time period, ft^3	BOD Mass Loading during time period, lb/hr	TSS Mass Loading during time period, lb/hr
M				0		
M-1	1.80	304	212	6,486	123	86
1-2	1.39	295	202	11,481	92	63
2-3	1.03	287	191	15,176	66	44
3-4	0.98	277	188	18,695	61	41
4-5	0.83	267	185	21,700	50	35
5-6	0.85	259	189	24,758	49	36
6-7	1.13	251	193	28,838	64	49
7-8	1.77	263	212	35,198	104	84
8-9	2.77	275	232	45,167	171	144
9-10	3.18	278	238	56,621	198	170
10-11	3.38	280	245	68,788	213	186
11-N	3.71	275	227	82,137	229	189
N-1	3.20	269	210	93,652	193	151
1-2	2.94	265	208	104,241	175	138
2-3	2.77	261	206	114,227	162	129
3-4	2.61	262	201	123,608	154	117
4-5	2.62	264	195	133,028	155	115
5-6	2.51	258	241	142,070	146	136
6-7	2.68	253	287	151,706	152	173
7-8	2.84	265	274	161,933	169	175
8-9	3.14	277	261	173,221	195	184
9-10	2.85	295	240	183,465	189	154
10-11	2.69	314	220	193,166	190	133
11-M	2.19	309	216	201,068	152	107
Average	2.33	275	220		144	118
Ave MGD	1.50					
Minimum	0.83	251	185		49	35
Maximum	3.71	314	287		229	189

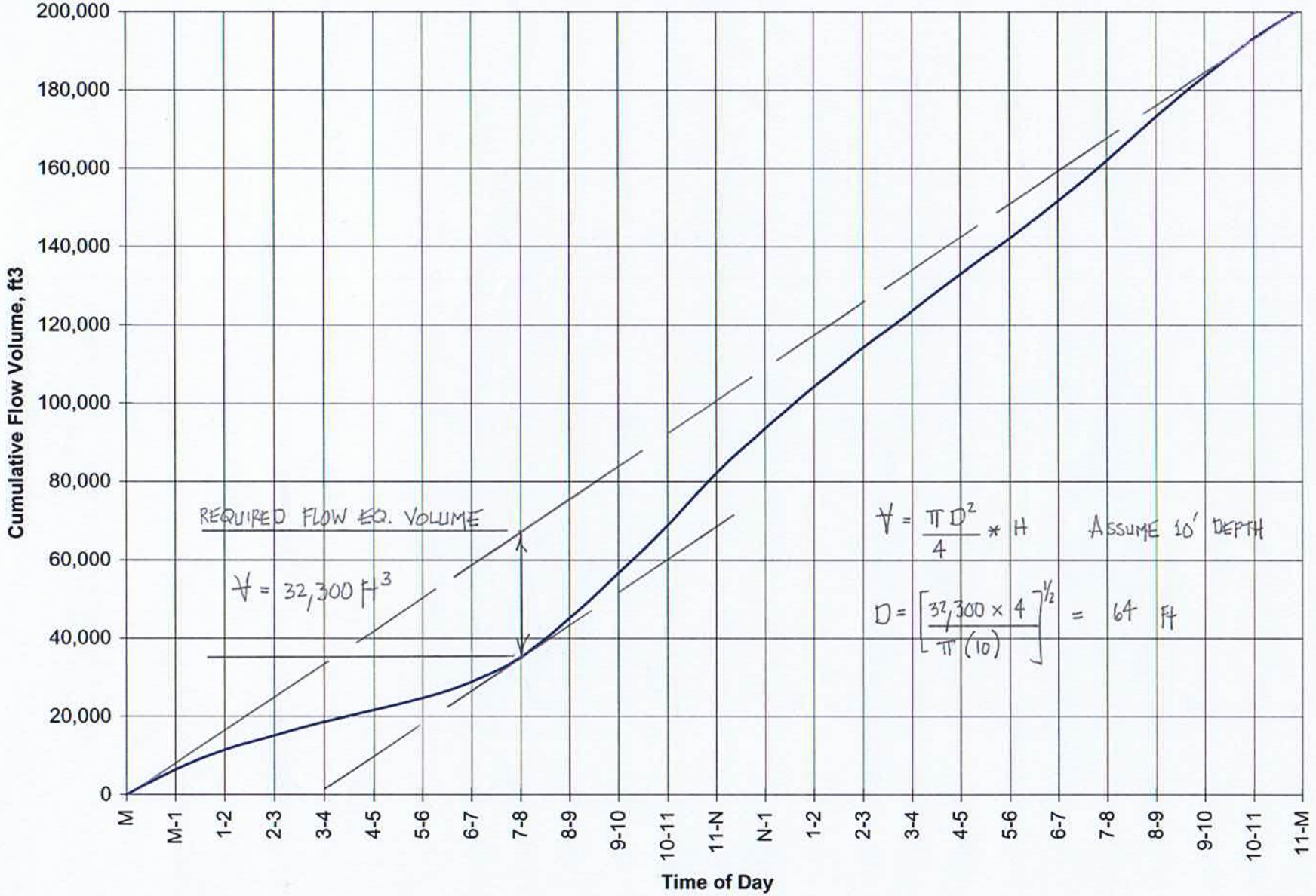
* Data was compiled for every other hour; intermediate hours are an average of the hours before and after; the BOD and TSS values are increased 61% and 73%, respectively, to account for increased wastewater strength during the peak seasonal periods and to normalize the data with past performance data.

Flow Equalization Calcs_Future_Peak Season.xls

Time Period	Volume of Flow during time period, ft3	Volume in Storage at end of time period, ft3	Average BOD concentration during time period, mg/l	Equalized BOD concentration during time period, mg/l	Equalized BOD mass loading during time period, lb/hr	Average TSS concentration during time period, mg/l	Equalized TSS concentration during time period, mg/l	Equalized TSS mass loading during time period, lb/hr
8-9	9,969	1,591	275.31	275	144	232	232	121
9-10	11,454	4,667	277.725	277	145	238	238	125
10-11	12,167	8,457	280.14	279	146	245	243	127
11-N	13,349	13,428	274.505	276	145	227	233	122
N-1	11,515	16,565	268.87	273	143	210	223	117
1-2	10,589	18,776	264.845	270	141	208	217	114
2-3	9,986	20,384	260.82	267	140	206	213	112
3-4	9,381	21,387	262.43	265	139	201	209	110
4-5	9,420	22,430	264.04	265	139	195	205	107
5-6	9,042	23,094	258.405	263	138	241	215	113
6-7	9,636	24,352	252.77	260	136	287	236	124
7-8	10,227	26,201	264.845	261	137	274	248	130
8-9	11,288	29,111	276.92	266	139	261	251	132
9-10	10,244	30,977	295.435	274	143	240	249	130
10-11	9,701	32,300	313.95	283	148	220	242	127
11-M	7,902	31,824	309.12	288	151	216	237	124
M-1	6,486	29,932	304.29	291	153	212	233	122
1-2	4,996	26,550	295.435	292	153	202	228	120
2-3	3,694	21,867	286.58	291	153	191	224	117
3-4	3,519	17,008	276.92	289	152	188	219	115
4-5	3,005	11,636	267.26	286	150	185	214	112
5-6	3,058	6,315	259.21	280	147	189	208	109
6-7	4,080	2,018	251.16	269	141	193	202	106
7-8	6,360	0	263.235	265	139	212	210	110
Average	8,378	= ft^3			144			119
	1,503,986	= gal/day						

Theoretical Flow Equalization Basin Results
Volume Determination

Future - Peak Seasonal



Appendix F

Manufacturer's Trickling Filter Media Modeling Results

PEAK TOURISM
FUTURE
AVE FLOW DRY WEATHER

Bio-Pac SF#30 Random Media Engineering Report For City of Morro Bay Morro Bay CA

Report Date: 11/20/2003

Existing System Data

No. of Filters:	1
Filter Diameter:	92.2
Media Depth	5.0

Wastewater Characteristics	Influent
Flow (gpd)	1,500,000
Total BOD (mg.l):	393
Total BOD (lbs/day)	4,915
BOD After Primary Clarifier (mg/l)	275
BOD After Primary Clarifier (lbs/day)	3,440
BOD Removal Efficiency Required (%)	Data Needed
Maximum Temp (°F)	68
Minimum Temp (°F)	68

Media Required for Total Rock Replacement **33,300** cubic feet

Total Media Replacement Evaluation (summer conditions)

Temp Correction Factor: 1.00
Biotower Loading: 104 lbs/1000 ft³/day
Predicted Removal Efficiency: 77 %

BOD concentration after clarification: 65 mg/l

Total Media Replacement Evaluation (winter conditions)

Temp Correction Factor: 1.00
Biotower Loading: 104 lbs/1000 ft³/day
Predicted Removal Efficiency: 77 %

BOD concentration after clarification: 65 mg/l

Based on the above analysis, the following recommendations are made:

Replace existing Rock Media with **33,300 cubic feet**
of SF#30 Bio-Pac Media and **7,340 ft³**
of N-Dec underdrain to achieve a minimum **77 %**
removal efficiency.

or

Replace the existing rock media with: **#VALUE!** **cubic feet**
of SF#30 Bio-Pac Media to achieve minimum required **Data Needed** **%**
removal efficiency.

The information presented is the confidential property of NSW, LLC. No part thereof shall be reproduced or disclosed to a third party without the prior written consent of NSW, LLC.

Bio-Pac SF#30 Random Media
Engineering Report For
City of Morro Bay

Report Date: 1/21/2004

Wastewater Characteristics	Influent	Effluent
Flow	1,500,000	
Total BOD (mg/l)	393	28
Total BOD (lbs/day)	4,915	350
BOD After Primary Clarification (mg/l)	275	
BOD After Primary Clarification (lbs/day)	3,440	
Tower BOD Removal Efficiency	90	
Sludge Production (lbs/day)	1,390	
Maximum Temp (°F)	68	
Minimum Temp (°F)	68	

→ AFTER 2ND CLARIFIER.

Media Required for BOD Reduction (summer conditions)

Temp Correction Factor: 1.00

Biotower Loading (lbs/1,000 ft^3/day): 43

Biotower Volume for BOD Reduction: 80,600 cubic feet

Media Required for BOD Reduction (winter conditions)

Temp Correction Factor: 1.00

Biotower Loading (lbs/1,000 ft^3/day): 43

Biotower Volume for BOD Reduction: 80,600 cubic feet

Based on the above analysis, the following recommendations are made:

Media Volume Recommended: 80,600 cubic feet

Number of Towers:	1	
Unit Diameter:	#DIV/0!	ft
Bed Depth:	#DIV/0!	ft
Maximum Sidewall Load is approximately:	#DIV/0!	lbs/ft^2
N-Dec Underdrain Required:	#DIV/0!	ft^2
Suggested Recycle Rate per Tower:	#DIV/0!	gpm
Wetting Rate:	#DIV/0!	gpm/ft^2

The information presented is the confidential property of NSW, LLC. No part thereof shall be reproduced or disclosed to a third party without the prior written consent of NSW, LLC.

ONE (1) 60' DIAMETER TRICKLING FILTER 12' MEDIA DEPTH
ONE (1) 70' DIAMETER TRICKLING FILTER 12' MEDIA DEPTH

BRENTWOOD INDUSTRIES

12/2/03 WATER TECHNOLOGY GROUP

Technical Data Summary [1]

PROJECT: Morro Bay

ENGINEERING FIRM: Cannon Assoc.

ENGINEERS NAME Larry Kraemer

REPRESENTATIVE FIRM: MISCO, Pleasanton, CA ph: 925-225-1900

REPRESENTATIVE: Mike Tooley fax: 925-225-9200

PROJECT DESCRIPTION: Two new trickling filter towers operating in parallel are proposed, using 12 feet of CF/S-3000CF/S-3000 media, for CBOD Removal Only; Tower Height (ft) = 12, the Tower Diameter (ft) is 60; ; Design Influent Flow Rate = 1.5 mgd, CBOD Load is 43.4 lbs/10^3 ft3, Wetting Rate 0.31 gpm/ft2, design temperature was 11 deg. C., Clarified Effluent Result.

Wastewater Influent & Effluent Data:	Raw Influent	Primary Effluent	Required Effluent	Clarified Eff. (est.)
Wastewater Type:		Domestic Wastewater		
Flow, mgd (m3/min.):	1.5 (3.94)			
CBOD5 (mg/l):	425	276	30	21.0

Filter System Configuration:	T-F Stage #1
Treatment Objective	BOD Removal
No. of Trains	2
No.of Filters	2

System Configuration

2 Trains of 1 Filter Each	Tower Diameter	Depth	Media Type	A_s	Volume
Tower #1, ft (m):	60 (18.3)	12 (3.7)	CF/S-3000	31 ft2/ft3 (102) m2/m3	33737 ft3 (955 m3)
Tower #2, ft (m):	70 (21.3)	12 (3.7)	CF/S-3000	31 ft2/ft3 (102) m2/m3	45989 ft3 (1302 m3)
			Total Volume of Tower(s):		79,727 ft3

Process Information:	T-F Stage 1
Purpose of Biological Filter:	CBOD,R
Design Temp., deg. F (deg. C):	51.8 (11)
Recycle Ratio Stage #1	1.00
Hydraullic Load, (Q_t, gpm/ft2 (m3/m2-hr)):	0.31 (0.77)
Org. Load, lbs/10^3ft3-day (kg/m3-day):	43.4 (0.7)
NH3-N, T-F Eff, lbs/day:	195 (142)
Vent Rate Each Filter, cfm (m3/min) =	1209 (34)
% O2 in Vented Air =	19.9%

Process Load Data:	T-F Stage 1
Raw WW Load/system, lbs/day (kg/day):	5320 (2418)
CBOD Load/Trickling Filter, lbs/day (kg/day):	3458 (1572)
CBOD Removed, T-F, lbs/day (kg/day):	3196 (1453)

(1) These calculations are completed as a courtesy. Brentwood Industries does not provide nor accept any responsibility for performance or process warranties as part of this offering, whether expressed or implied. We recommend that a professional engineer provide detailed structural and process designs.

Appendix G

Trickling Filter Upgrades – Cost Estimate

NSW
ENVIRONMENTAL SYSTEMS

NSW, LLC
530 Gregory Avenue
Roanoke, Virginia 24016

800-368-3610
540-981-0362
540-345-8421

January 21, 2004

Pages: 3

Fax: 805-544-3836

To: Larry Kraemer

Firm: Cannon Associates

Subject: City of Morro Bay CA Trickling Filter Project

I have included the process designs based on 1.5 MGD flow and 2.4 MGD flow with 275 mg/l TBOD influent to the two trickling filters running in parallel. Also, 28 mg/l TBOD was used as the effluent from the secondary clarifiers.

Total Estimated Budget Costs

1.5 MGD	
36,699 ft³ Rock Media and Shale Underdrain removal	$20,400
80,070 ft³ Bio-Pac SF#30 Media	$240,210
Media Installation	$16,815
7,007 ft² N-DEC Underdrain	$20,671
Underdrain Installation	$2,000
12 PVC 8" Thin Wall Vent Pipes and Installation	$1,200
Total Estimated costs including freight	**$301,896**

2.4 MGD	
36,699 ft³ Rock Media and Shale Underdrain removal	$20,400
128,446 ft³ Bio-Pac SF#30 Media	$385,338
Media Installation	$26,974
7,007 ft² N-DEC Underdrain	$20,671
Underdrain Installation	$2,000
12 PVC 8" Thin Wall Vent Pipes and Installation	$1,800
Total Estimated costs including freight	**$457,183**

I would suggest bolted steel tank be used to increase the tank height. I will copy Tarn Victor with JBI this Fax for his assistance in obtaining budgetary costs.

If you should have further questions feel free to contact me.

Very Truly Yours,

Edward Sagedy

Edward Sagedy-National Sales Manager

C/c: Ken Krupa-NSW
Tarn Victor-JBI

NSW, LLC.-Charlotte Office, 10194 Willow Rock Drive, Charlotte, NC 28277
Telephone 704-540-7573 Fax 704-540-7597 Email: sagedychar@earthlink.net

Appendix H

NPDES Requirements Comparison

This section provides a brief summary of the different NPDES permit designations and requirements and compares them to recent performance data from the Morro Bay-Cayucos WWTP.

The WWTP has a 301(h) modified National Pollutant Discharge Elimination System (NPDES) permit for discharge of blended primary and secondary effluent. Compared to the standard full secondary treatment permit, two major secondary effluent limitations, Biochemical Oxygen Demand (BOD_5) and Total Suspended Solids (TSS), are eased under this permit, whereas, all other secondary discharge limitations remain in effect. An (*) is placed next to the category where, on rare occasion, the limitation was exceeded. See Appendix I for a summary of these occurances.

Table A-1 Effluent Quality Comparison

Constituent	Limitation	Units	Effluent Limitation Value for Different Levels of Treatment			MBCWWTP Effluent Quality (Average of Past 3-years)	
			Current Modified Discharge Permit	Equivalent to Secondary	Full Secondary	Peak Seasonal	Average Annual
BOD_5	30-day average	Mg/l	120	45	30	68	62
	7-day average	Mg/l	---	65	45	---	---
	Maximum at any time	Mg/l	180	---	---	---	114
	30-day removal efficiency	%	30	65	85	82.5	82.1
TSS	30-day average	Mg/l	70	45	30	49	42*
	7-day average	Mg/l	---	65	45	---	---
	Maximum at any time	Mg/l	105	---	---	---	105*
	30-day removal efficiency	%	75	65	85	87.2	87.4
pH	Minimum		6.0	6.0	6.0	---	7.1
	Maximum		9.0	9.0	9.0	---	7.7

Appendix I

Response to Regional Water Quality Control Board Letter Regarding Plant Permit Violations

City of Morro Bay

Morro Bay, CA 93442 • 805-772-6200
www.morro-bay.ca.us

November 6, 2003

Mr. Roger Briggs
Executive Summary
California Regional Water Quality Control Board
895 Aerovista Place, Suite 101
San Luis Obispo, CA 93401

Subject: Complaint for Mandatory Penalty - Morro Bay/ Cayucos Wastewater Treatment Plant

Dear Mr. Briggs,

This letter is written in response to your letter dated October 7, 2003 concerning the complaint for mandatory penalty for the Morro Bay/Cayucos Wastewater Treatment Plant.

The City of Morro Bay and the Cayucos Sanitary District (MBCSD) have always been sensitive to the quality of the local marine environment within Estero Bay and have operated the WWTP with a commitment to protection of the local ecology and coastal resources. We are committed to maintaining the high levels of treatment at the WWTP with the goal of preserving the beneficial uses of the marine environment within Estero Bay. The City and District are proud of the exceptional operational performance that the MBCSD WWTP has consistently achieved since its reconstruction in 1984.

Unfortunately, there have been rare occasions where the WWTP has unavoidably been unable to meet the Waste Discharge Requirements specified in WDR 98-15. Along with reporting these unusual occurrences, the WWTP has documented the extraordinary efforts expended by City and District staff to investigate and implement the appropriate corrective actions necessary to maintain the normally high operational performance of the WWTP. In keeping with that policy, we submit the following explanations of the incidents which summarize the actions taken at the WWTP to avoid future violations of the sort listed on page 3, of Complaint No. R3-2003-0052 dated October 7, 2003. More timely and detailed submissions of the explanations which follow were also presented in: Monthly Operations Summaries, the July 2002 Semi Annual Effluent Sampling Report, the 2002 Annual Report, and the Supplement to the 2003 Renewal Application for Ocean Discharge Under NPDES Permit No.CA0047881.

Notwithstanding these discussions of the incidents, the City of Morro Bay and the Cayucos Sanitary District hereby waive the rights to a hearing, which had been scheduled for December 5, 2003. A signed waiver form is enclosed. In accordance with item No.4 on page one of Complaint No. R3-2003-0052, the MBCSD propose that the fines be applied to two worthy Supplemental Environmental Projects (SEP) that were proposed in a letter to your staff, dated September 12, 2003. MBCSD anticipate approval of funding for both

the Cayucos Lioness Mutt Mitt Program and Inactive Metal Mine Investigation. As stated to Mr. Matt Thompson of the RWQCB staff during a recent phone conversation, MBCSD authorized Marine Research Specialists to initiate collection of offshore porewater samples for the Inactive Metal Mine SEP during the recent, regularly scheduled, benthic sampling survey. Had these samples not been collected during that survey, sampling would have been postponed until the next annual offshore survey in October 2004. Projected funding for this SEP would not cover the costly and unnecessary additional expense of mobilizing for a separate offshore benthic survey to collect porewater samples. The MBCSD felt that the implications of this SEP for regional water quality merited the initial outlay of funds for the timely collection of benthic porewater samples and their high-resolution analysis for bioavailable trace metals. Without additional funding for this SEP the results of these chemical analyses will not be reported and the onshore source-identification phase will not occur.

Each incident leading up to the six violations identified in Complaint No. R3-2003-0052 follows below. None of the incidents were related to degradation in plant performance. Instead, they were all isolated incidents caused by either a brief unavoidable mechanical failure of a treatment-system component, the external introduction of a contaminant within the influent stream, or laboratory error.

Exceedence of TCDD Equivalents (Dioxin) on 10 July 2002:

The dioxin concentration measured in the July-2002 effluent sample (0.563 pg/L) exceeded the permit limit (0.52 pg/L) by an extremely small concentration (0.047 pg/L). Concentration differences as small as 0.047 parts-per-trillion are not reliably determined because protocols only require that the instrument calibrate to within 20% when running standards. This introduces instrumental uncertainty that is much larger than the 9% difference between the measured concentration and the permitted limit. The determination of the extremely low dioxin concentrations is fraught with difficulty and tests are expensive to conduct (more than $1000 per sample). The California State Water Resources Control Board acknowledges the difficulties associated with dioxin surveys and "*...recognizes that there is a very limited number of laboratories equipped to perform these analyses at the very low levels of detection necessary to determine if these compounds are present at concentrations of potential health significance. A credible survey would be expensive*" (SWRCB, 1999).

Laboratory contamination at these extremely small concentrations adds significant additional uncertainty to the reported results. For example, initial analysis of the July-2002 effluent sample erroneously reported a much higher dioxin level (0.800 pg/L). This reported concentration was subsequently found to be due to laboratory contamination. The contamination was discovered during the concurrent analysis of the "blank" sample which found that the certified "pure" water used to dilute the effluent sample contained a comparatively high level of dioxin contamination, yet it was still certified "pure." Subsequent use of an internal standard from a second manufacturer reduced the levels in the blank and resulted in the erroneously high concentration reported for dioxin.

Even if dioxin were present within the effluent, the July-2002 measurement does not necessarily reflect a significant new source of dioxin and a costly source-identification investigation is unlikely to isolate a mitigable contaminant supply. Dioxins are ubiquitous in the environment and do not easily break down into other compounds. Therefore, once produced, dioxins persist in the environment and are found almost everywhere. Only four of the 210 dioxin/furan congeners were found in the July-2002 effluent sample. These four congeners are the least toxic and most

common of the dioxin congeners. The four dioxin/furan congers found in the July-2002 sample are 100-times to 1000-times less toxic than the TCDD congener. Additionally, this was the first instance of an exceedence of the dioxin limitation in the NPDES permit. It does not portend lack of compliance with dioxin standards in the future.

Exceedence of Suspended Solids on 26 August 2002, 11 September 2002, and September 2002:

One of the major challenges that faced the WWTP staff during 2002 was a bloom of adverse bacteria that began in late August, and compromised the secondary treatment process for much of September. Repercussions from this event lasted into December. WWTP staff expended immense effort attempting to remediate the upset in the secondary clarifier and to discover its cause. They consulted a wide variety of experts in treatment processes and microbiology. Throughout the effort, RWQCB staff were kept apprised by telephone of the status of the upset. The largest exception to the TSS limit was caused by the application of a remedial measure (chlorination) that temporarily exacerbated TSS emission.

The cause of the upset was eventually identified as external to the plant and, as described below, corrective actions have been implemented to limit future incidents. As an additional preventative measure, WWTP staff received advanced training in microbiological techniques that will serve to monitor the health of the bacteria in the secondary treatment process. This expertise will help to provide early warning of any potential future problems similar to the one experienced in 2002.

The upset resulted in three exceedences of the permitted discharge limit on TSS. The instantaneous maximum limitation on suspended solids (105 mg/L) was slightly exceeded in the 24-hour composite sampled collected on 26 August. It contained a TSS concentration of 107 mg/L. The 2-mg/L exceedence of the permit limit only amounted to an additional 7.6 kg of discharged solids. Over the next two weeks, various operational changes failed to ameliorate the problem, and on 10 September the mixed liquor was chlorinated in an attempt to eliminate undesirable bacteria. The resulting widespread decimation of bacteria generated a TSS concentration of 147 mg/L that was measured in the effluent sample collected on 11 September. Although short-lived, this was the highest effluent concentration recorded since 1995 when the WWTP was flooded during intense rainstorms. These and some other relatively high TSS concentrations measured in September generated an average TSS concentration of 79 mg/L, which exceeded the monthly average of 70 mg/L specified in the NPDES discharge permit.

All of these incidents were related to the bloom of adverse bacteria in the secondary treatment process. The original upset began in late August when a bloom of deflocculating bacteria was noted in the secondary clarifier. Bacteria normally aid flocculation of activated sludge and enhance the effectiveness of clarifiers. However, excessive growth of certain types of bacteria can create a settling problem within the secondary clarifier, thereby limiting the return of settled sludge to the solids contact chamber. This increases the discharge of suspended solids from the secondary treatment system. When such an increase occurred in August, WWTP staff disassembled and inspected the return-activated-sludge (RAS) and waste-activated-sludge (WAS) pumping systems and found them to be operating as designed. They collected dissolved oxygen measurements within the solids contact channel and secondary clarifier and determined that the aeration system was also functioning properly. They consulted numerous experts including: staff

at the City of San Luis Obispo Water Reclamation Facility, Mr. Darrel Richards (retired manager of the MBCSD WWTP), Mr. Ron Sherer (retired RWQCB staff), Dr. Kris Kitts (Cal Poly microbiologist), RWQCB staff, and Mr. Ron Schuyler (prominent wastewater microbiologist). Microscopic examination of the mixed-liquor suspended solids and zoogleal mass from the trickling filters identified excessive deflocculation as the problem causing the bulking sludge and high effluent TSS.

Evidence of excessive bacterial deflocculation and visible changes in composition of the dominant bacteria adhering to the trickling filter media strongly suggested that the upset was caused by harmful influent contaminants rather than mechanical disfunction of the treatment process within the plant. Accordingly, WWTP personnel embarked on a major source-identification effort to locate potential contaminants entering the sewer system that could have caused the upset. WWTP personnel consulted with industrial waste inspectors and operations staff from numerous agencies including: Los Angeles County Sanitation, Orange County Sanitation District, City of San Diego, RWQCB staff, City of Simi Valley, and the City of Santa Barbara. Staff discussed the possible pollutants of concern and their effects on the biological treatment process. Of particular interest were Methylene Blue Active Substances (MBAS), sulfides, phosphates, and nitrogen compounds.

Excessive sulfate levels were found to be entering the collection system from Mission Linen. In response to elevated influent pH, Mission Linen had been directed to install a pH neutralization system in June 2002. However, the sulfuric acid used to reduce high-pH discharge from Mission Linen produced a sodium sulfate byproduct that may have caused an adverse impact on the biological processes at the plant. On 23 October, Mission Linen discontinued use of the pH neutralization system until phosphoric acid could be used as a replacement pH neutralizer. Although there was another minor instability in the secondary treatment process in early December, plant efficiency steadily improved after the use of sulfuric acid was discontinued.

After the treatment process returned to its normal level of efficiency in mid-December, WWTP staff instituted routine monitoring of microbiological characteristics of the suspended solids and settled sludge. WWTP personnel have continued to receive training in microbiological analysis from staff at the City of San Luis Obispo Water Reclamation Facility, and from wastewater biology classes sponsored by Pennsylvania State University. Continued monitoring of microscopic processes within the wastestream can identify precursors to the kinds of bacterial problems that occurred in the secondary treatment system in August 2002.

Excessive Total Chlorine Residual on 29 December 2002 and 16 January 2003:

The two violations listed in the complaint were the direct result of brief unavoidable mechanical failure of a single treatment plant component. Both incidents occurred because of malfunction in the chlorine analyzer used to pace the dose of hypochlorite and bisulfite. As described below, steps have been taken to reduce the likelihood of future incidents of high chlorine residual resulting from mechanical malfunction. Overall, the historical record of chlorine residual concentrations amply demonstrates the treatment plant's ability to consistently meet the applicable receiving water standards. There is no evidence of a decline in the plant's ability to eliminate residual chlorine from the effluent. In fact, the presence of detectable chlorine in effluent samples has become increasingly rare, which suggests that the effluent will continue to comply with receiving-water standards in the future.

The large chlorine-residual concentrations (10 mg/L) measured on 29 December 2002 and 16 January 2003, exceeded both the daily and instantaneous standards. Both were caused by a malfunction in the chlorine analyzer, which continuously evaluates concentrations within the chlorine contact tank and paces the sodium hypochlorite dose to ensure adequate disinfection. On one occasion, an internal filter screen clogged, and on the other, internal tubing ruptured. In both cases, the analyzer interpreted the absence of sample to mean that chlorine levels within the contact tank were too low. It increased the sodium hypochlorite dose accordingly. The sodium bisulfite dose, which is used for dechlorination, is also partially based on the residual chlorine measurement provided by the analyzer. With an erroneous zero reading, the sodium bisulfite dose was incorrectly minimized. This compounded the problem of the excess hypochlorite dose, and chlorine levels were not adequately neutralized.

Because of their complexity, the chlorine analyzers have historically had an intensive maintenance program. A new analyzer was installed in February 2002 to complement the existing one. In May, the sample pump motor was replaced on the existing analyzer, and the sample cell was rebuilt in October 2002. After the filter screen clogged on the intake tube in December, WWTP staff redesigned the sample-supply line to improve flow, and increased the frequency at which the filter screens are cleaned. To further limit the risk of an excess disinfection/dechlorination incident after normal operating hours, WWTP staff installed a dial-out alarm system to notify offsite personnel when TRC concentrations extend above or below a nominal range. The alarm system became operational in February 2003, after a second malfunction in the analyzer caused elevated chlorine concentrations in the effluent on 16 January 2003.

If you have any questions or comments please contact Bruce Keogh at (805) 772-6272.

Sincerely,

Bruce Ambo
Public Services Director

cc
Mr. Rob Schultz
City Attorney

Ms. Bonnie Connelly
Cayucos Sanitary District

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Mr. Bruce Keogh
Wastewater Division Manager

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Manager/C/Bkeogh/Letters1/RWQCBWaiveHearing102903

Appendix J

Efficiency Curves for Singe-Stage Stone-Media Trickling Filters

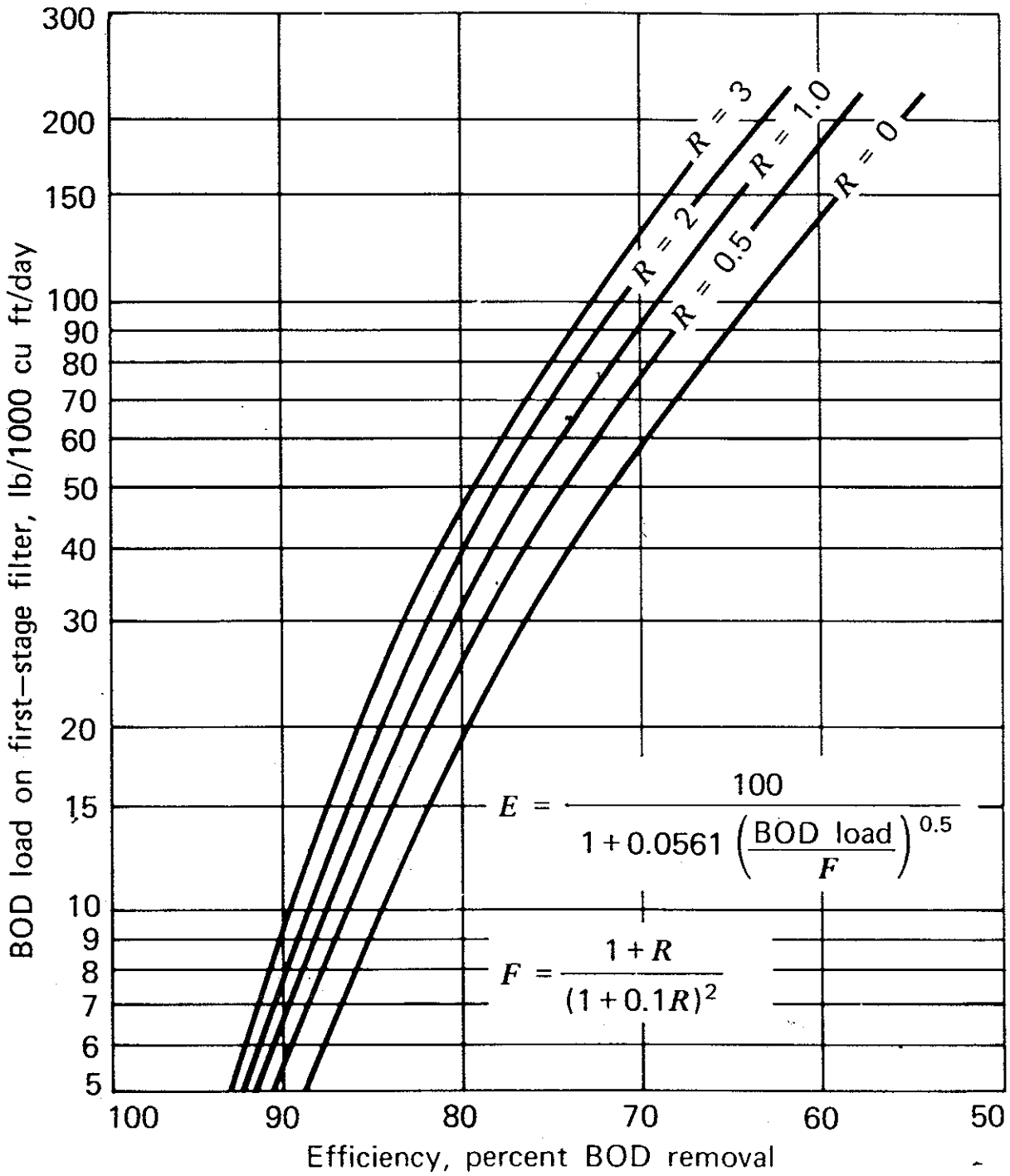

Figure 11-20 Efficiency curves for single-stage stone-media trickling filters treating domestic wastewater at 20° C, based on the National Research Council data.